Structural Design

~~Of~~ of

Multi-Storeyed Buildings

~~Second~~ Third Edition

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1 Introduction

1.1. Need ~~Of~~ of Multi-Storeyed Buildings

Most of the existing buildings are multi-storeyed, in the sense that “multi-storey” means more than one storey. Single storey buildings may comprise of workshops, factory sheds, cinema halls, auditorium, etc. Even ordinary residences can be thought of as multi-storeyed buildings as presently ordinary residences are allowed to be built of two and a half storeys. With increasing pressure of population, commerce and trade, land costs in cities have risen very high. Often, there is no other alternative except building high on a given plot of land. Educational and commercial buildings are, in general, eight storeys high with lifts provided. Without lifts, a building has to be less than or equal to four storeys in Delhi, but not more than five storeys in Bombay. b/ These restrictions are imposed by the city bye-laws, keeping in view the pressure on the services like roads, water, electricity and sewage disposal. Some buildings are made extra ordinarily tall out of prestige. For example, Delhi Development Authorit~~ys~~'s Vikas Minar is a 23-storey tower, while Hansalaya at Barakhamba Road in New Delhi, is a 22-storey building. ?Urban Arts Commission is one of the agencies which control the building height or the number of storeys of buildings in Delhi and New Delhi. Similar controls are exercised in all major cities in India.

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1.2. Structural Materials For Buildings

Traditionally, buildings have been built in brick (or stone) and mortar and also in timber in certain areas, where wood was available in plenty. Presently, wood has become scarce and prohibitively costly. Small buildings, particularly residences, are ideally built with load bearing brick walls. The horizontal members like slabs are built in reinforced concrete. In certain areas, brick walls may be replaced by random stone masonry walls, but these occupy more space in plan. Brick walls are, in general, 230 mm thick in plan, while stone walls may be 300 mm to 450 mm thick in plan. Concrete blocks, hollow or solid, are also available in the market, but these have not caught on for certain practical reasons, like difficulty in making notches for electrical conduits, etc. In framed buildings, structural steel and reinforced concrete are the two alternative materials. Structural steel frames were previously used in Calcutta area for multi-storeyed buildings. But now, for reasons of economy, reinforced concrete has replaced structural steel in the construction of multi-storeyed buildings of medium height. With the introduction of shear walls and shear cores, reinforced concrete systems are working out well for tall buildings as well. Further, a reinforced concrete structure has better fire resistance than that of a steel structure. So, in the present scene, brick and reinforced concrete are the two most prevalent structural materials in building construction.

pre-stressed?

1.3. Load

1.3.1. Vertical Loads

Structurally speaking, buildings are built to support loads. The load, which is ever present and ever acting on a building, is the dead load which consists of the self-weight of members, finishes, plaster, etc. Dead load should be calculated very accurately, as it

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comprises most of the building load. IS: 1911 − ~~1967~~[1*] gives a schedule of unit weights of building materials and it is used extensively to calculate the dead load.

imposed

Next in importance to dead load, is the live load, which is caused by the use of building. Live loads are given in IS:875. part 2 1987 Amendment 1 2 Live loads are generally high (150 Kg/$m^2$to 1000 Kg/m^2) on floors depending on the activity that is carried on there, while it is of a low value (75 Kg/$m^2$to 150 Kg/m^2) on a roof, which may or may not be accessible. Snow loads on roofs in hilly areas are also specified in IS:875. part 4 1987 In snow-incident areas, roofs are to be made sloping so that snow cannot get accumulated to a great height. IS:875 gives the loading due to snow at 2.5 Kg/m^2per cm depth of snow. With 30 cm 300 mm snow depth, the snow loading will work out to be 75 Kg/m^2, which may be reasonable for sloping roofs.

kN/m^2

Partition loads are also important to be considered. Wooden or similar light-weight partitions anywhere on a floor give a general loading of 100 Kg/m^2 foot? of floor area. But in most buildings, 115 mm thick brick walls are arranged to divide space, this gives a heavier loading on the floor. IS:875 /b -part 1 or 2? 2 gives the partition walls loading at one-third the weight of 1.0 m run of the partition wall. For 115 mm thick brick walls of 3.0 m height anywhere, the equivalent loading works out to be (0.115 ×1.0 ×3.0 ×1900)/3 = 218.5 Kg/$m^2$of the floor area. For 230 mm thick brick walls, we generally take care to provide beams to support directly such walls. In multi-storeyed buildings, 115 mm t hick brick walls anywhere add substantially to the load of the building and it affects the design of slabs, beams, columns and footings too. But in the present practice, for flexibility in the use of the building, this provision is made in most buildings and wherever possible, 115 mm thick brick walls should be replaced by wooden partitions to achieve lighter partition loading, which finally leads to economy in structural design. In practice, wooden partitions are provided in office buildings, while in hospitals and institutional buildings , 115mm thick brick walls arc used as partition walls.

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1.3.2. Temperature and Shrinkage Loading

degree in LaTeX?

reference 3

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Temperature and shrinkage also act on a building and these can also be regarded as a load on it. Shrinkage is equivalent to -15^0C, where negative stands for fall of temperature.3 The temperature differential is taken a $\pm\frac{2}{3}(t_1 - t_2)$, where t_1 and t_2 are the maximum and the minimum temperatures observed in a day (24 hours) for a given place or locality.(4) Fall of temperature together with shrinkage will govern the design, while the rise of temperature will be substantially reduced in effect by the action of shrinkage. The design temperature differential is given by the Indian Road Congress (5) at $\pm 17^0C$ for moderate climates and at $\pm 25^0C$ for extreme climates. The combined effect of temperature and shrinkage is given below:

$$\text{For moderate climates : } \pm\ 17\ \text{-}\ 15 = +2\ \text{-}\ 32\ (^0C)$$
$$\text{For extreme climates : } \pm\ 17\ \text{-}\ 15 = +2\ \text{-}\ 32\ (^0C)$$

-32 to 2
-40 to 10

2000 (6)

(8)

IS:456 1978 (hereafter called simply the Code) states in its clause number 17.5.1 that "in ordinary buildings, effect due to temperature fluctuations and shrinkage and creep can be ignored in the design calculations". It is, however, not explained, what is meant by an ordinary building.(7?) It is, of course, clear that temperature and shrinkage loading has an effect on the design of long concrete buildings, H which can be neglected if the length of building is restricted to 45 m (clause 20.3 of the Code). Thus, it can be surmised that temperature and shrinkage effect can be neglected in short length buildings. It is also seen that by providing minimum specified steel percentages in concrete members, temperature and shrinkage effects can be absorbed in short length buildings, while in long concrete buildings, these members have to be designed for this extra loading or a long building has to be cut up in two or more short length buildings. Further, this loading can be made use of in the evaluation of the gap of an expansion joint.

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1.3.3. Wind Loading

Dead and imposed live loads are vertical or gravity loads, while wind and earthquake cause horizontal loads on a building. Temperature and shrinkage also result in a horizontal load on a building. Blast effects, earth and water pressures also cause horizontal loads on a structure. IS:875 /b part 3 gives values of wind pressures varying from 100 Kg/m^2 to 200 Kg/m^2 acting on building upto a height of 30m above the mean retarding surface, i.e. the mean level of the adjoining ground. For buildings of height upto 10.0 m, these wind pressure values can be reduced by 25%.

1.3.4. Earthquake Loading

Details of earthquake loading are given in IS:1893 year?. India has been divided into five zones with basic horizontal seismic coefficient (α_0) varying from 0.01 to 0.08. The base shear (kb) V_B is given by

$$V_B = C\alpha_h KW \tag{1.1}$$

C = efficient defining the flexibility of structure, which depends on the time period of the building, which in turn, is a function of number of storeys

α_h = design seismic cofficent $= \dfrac{\beta}{\alpha_0}$

K = performance factor which is equal to unity for reinforced concrete framed building with detailing taking into account the requirements of ductility

W = total dead load g + appropriate amount of live load (25% to 50% LL)

P = coefficient depending on the soil-foundation system (varying from 1.0 to 1.5)

I = coefficient depending upon the importance of structure varying from 1.0 to 1.5 for buildings.

α_0 = basic horizontal seismic coefficient depending upon the zone in which the locality in question falls.

$$= \frac{a}{g} = \frac{\text{design acceleration in horizontal direction due to earthquake}}{\text{accleration due to gravity}}$$

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The vertical basic seismic coefficient is half the value of α_0 It has, in general,no effect on the structural design of buildings. All buildings, whether short ortall, shall be checked for horizontal effect of earthquake loading.

1.3.5. Blast Loading

In certain coal mine areas, where open cast methods of mining are used, blast is of regular occurrence. Buildings in these areas must be designed for blast effects. IS:6922 underground -1973 gives the blast effects

$$\frac{a}{g} = \frac{\text{design acceleration in horizontal direction due to earthquake}}{\text{acceleration due to gravity}}$$

$$= K_2 \frac{Q^{0.83}}{R^2} \qquad \textbf{(1.2)}$$

where, $K_2 = 4$

Q = charge per delay in Kg.

R = distance of structure from blast point in meters.

α_0

The calculated a/g value for blast may be taken as a fraction of earthquake effect given by Uo. The building can be analysed for, say, earthquake and the the blast effect can he evaluated by applying this fraction on the values obtained from the earthquake analysis. For a locality with $\alpha_0 = 0.04$, Q = 20,000 kg, R=1200 m. (Eq.3.1) gives,

$$\left(\frac{a}{g}\right)_{blast} = 4 \times \frac{(20,000)^{0.85}}{(1200^2)} = 0.01$$

This indicates that the blust effect is equivalent to (.01)/(.04) = .25, i.e. one-fourth the earthquake effect, in this particular case.

1.3.6. Impact

Live loads given in IS:K75 /b8 include the effect of impact and this is clearly mentioned in clause 3.1.1 of this code. But for a moving machinery, these loads should be increased

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by an allowance varying from 20% to 100%, as per clause 3.4. In a lift machine room above the roof of a multi-storeyed building, we generally consider a live loud of 1000 Kg/m^2together with an impact allowance of 100%, giving a loud of 2000 Kg/m^2.

/b

1.3.7. Earth and Water Pressures and Surcharge Load

Fig.

These loads act on a retaining wall. In 1.1, a basement retaining wall is shown under the action of these loads. The base pressures acting on a retaining wall are given as under

$$\begin{aligned}
p_e &= \text{earth pressure} = \frac{1-\sin\phi}{1-\sin\phi}\gamma h && \textbf{(1.3)}\\
p_w &= \text{water pressure} = wh^{'} && \textbf{(1.4)}\\
p_s &= \text{surcharge pressure} = \frac{1-\sin\phi}{1-\sin\phi}w_s && \textbf{(1.5)}\\
where, \phi &= \text{angle of repose of earth}\\
\gamma &= \text{unit weight of earth}\\
w &= \text{unit weight of water}\\
w_s &= \text{surcharge loading} = 500kg/m^2\text{for pedestrain traffic}\\
&= 1000kg/m^2\text{for vehicular traffic}\\
h &= \text{height of the earth retained}\\
h^{'} &= \text{height of the subsoil water retained}
\end{aligned}$$

/b ian

1.3.8. Load Combinations

The load which is ever acting on a structure is the dead load which includes the load of partitions also. Then the effect of live load, which may vary in intensity from 0 to 100% of its value, is additive to the effect of the dead load, as both these loads are gravity or

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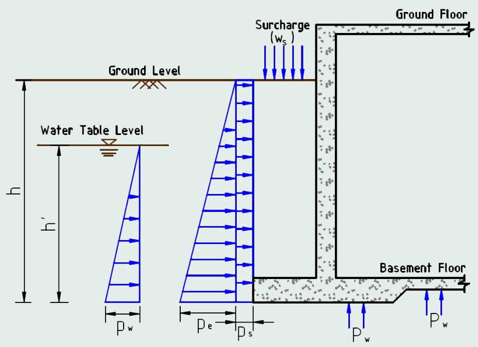

Figure 1.1: Pressures on a basement retaining wall

vertical loads. So, the structures should be designed by the limit state method for the dead and live load combination and it is given by the Code as

$$U \;=\; 1.5\,(D + L) \tag{1.6}$$

Then, horizontal loads of wind, earthquake and temperature will act on the building. It is assumed that the worst effects of the above three loadings will not coexist at the same time. Further, it is assumed that the worst effects of these loadings will act only for a short while on a building so that its load factor can be reduced by 25% (or allowable stresses in materials are increased by 33.33% in the working stress method, resulting in the same effect). Hence, the next load combination for checking the structure is given by the Code,

1.2

$$U \;=\; 1.5\,(D + L \pm W or E or T) \tag{1.7}$$

where the value of 1.2 is derived from 0.75 ×1.5 = 1.125 or 1.2 and ± sign is added to take care of the reversal of direction of horizontal loads. For checking overturning of certain isolated tall structures, live load in equation 1.7 may be made zero and 10% reduction

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may be made in the dead load to account for any inaccuracy in the calculation of the dead load. This will give the following load combination lot for isolated tall structures,

$$U = 1.2\,(0.9D + WorEorT) \tag{1.8}$$

/b

1.5

For inclusion of the blast effect in structural design, we consider a load factor of 1 5 for blast loading, as it occurs frequently in the open-cast mining areas.So, for such areas, the governing load combination, in addition to those given above, is also,

$$U = 1.5\,(D + LorB) \tag{1.9}$$

it

Also, ii is assumed that the worst effect of blast will not take place together with the worst effects of earthquake or wind or temperature. The notation in Eq: 1.7 to 1.9 is explained below

$$\begin{aligned} U &= unlimitedload \\ D &= deadlove \\ L &= liveload \\ W &= windload \\ E &= earthquakeload \\ T &= temperatureandshrinkageload \\ B &= blastload \end{aligned}$$

The above write up is our explanation of Table 12 of the Code, which gives the relevant load combinations for structural design. The reader may take note of the changes called for in Table 12 of the Code, in the light of above explanation.

1.4. Building - A Result Of Combined Efforts Of Several Professionals

A client is one who has the necessary motivation and capital to make a building either for his own use or for sale. He engages an architect to prepare an architectural scheme and drawings and get these approved from the local municipal authority and these drawings

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form the basis on which the other professionals like structural, estimating, plumbing, electrical and air conditioning engineers work to prepare their own drawings and estimates. An architect is primarily concerned with the function and the area utilization of building. It takes care of the visual look of the building, its orientation and its position in the promulgation area. The structural engineer is to take care of the safety of the structure and he is to devise ways to support all the loads coming on the building in the most economical way. He is concerned with the design of both the superstructure and the sub-structure (i.e. foundations). A structural engineer is called upon, at an early stage, to work in close collaboration with the architect, to decide the structural grid and the sizes of structural members. An estimating civil engineer is also associated to prepare estimates and tender documents. After the tenders are called, these are compared and the client awards the work of construction of building to a contractor selected from the tenderer.

The role of the contractor and his engineers is of paramount importance in the construction of building. They have to ensure that the work at the site is carried out in accordance with the working drawings of the architect, the structural and other professional engineers. To supervise the work of the contractor, a separate agency or an individual (called clerk of works) is appointed, who coordinates the work of the. several agencies at the site and ensures quality and timely completion of work at the site. The author has the highest regard for the work of contractors site engineer, as he has to construct the building from both the architectural and the structural drawings. In cast-in-situ construction, there may remain, in practice, some gaps in these two sets of drawings and the site engineer, with the help of the clerk of works, closes up these gaps, reconciles these differences and gets the building constructed. The best site engineer is one who consults the architectural and the structural staff and with proper coordination, completes the building in accordance with the drawings. There are some site engineers who arrogate to themselves the powers of the design staff and these engineers, sooner or later, come to grief.

In some unusual structures, specialists or academic staff may be required to be associated with the structural design for consultation and/or checking as per the need.

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Further, the architect plays an initiatory, coordinating and controlling role and he is, thus, rightly called as the leader of the team of professionals involved in the design and construction of a building. Finally, it should be rightly understood that a building is the net result of the combined efforts of several professionals involved in the building industry.

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2 Structural Systems for Buildings

2.1. Introduction

Choice of appropriate structural system for a given building is vital for its economy and salary. It is an important decision which is to be taken by a senior structural engineering.Small buildings like houses, etc. generally use bearing brick walls will reinforced concrete floor slabs. For taller buildings, reinforced concrete frames in both the principal directions are provided with buck walls used as only filler walls. For still taller buildings, frames with shear walls will be provided to resist both the vertical and the horizontal loads. Like wise, more intricate and innovative structural systems may bethought of, in the case of unusual buildings. We describe below the various structural systems commonly used in buildings of different types.

2.2. Load bearing Masonry Buildings

Houses, hostels and similar small buildings are built with load bearing brick walls with floor slabs being cast in reinforced concrete. This system is suit able for buildings upto four storeys or less in height (Fig: 2.2). It is quick in construction and economical in cost. However, care shall be taken to arrange walls over walls in plan and openings in walls shall be restricted. Bricks shall be of a crushing strength of 100 Kg/cm^2minimum

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for four storeys, but this value can be 75 $Kg/cm^2$for two storeys or less. Brick walls with reinforced concrete floor slabs are adequate for vertical loads. This system also serves to resist horizontal loads like wind, earthquakes or blast, by way of box-action in plan. Further, to ensure its action against earthquake, it is necessary to provide horizontal bands and vertical reinforcement in brick walls as per IS: 4326. In some buildings, 115 mm thick internal walls in brick are provided. As 115 mm thick walls are incapable of supporting vertical loads, beams have to be provided along their lengths in order to support the adjoining slab panels and the weight of 115 mm thick brick walls. These beams are to rest on 230 mm thick walls or reinforced concrete columns if required, resulting in a mixed system of load bearing brick walls with reinforced concrete columns wherever system IS: 1905 is the code governing the design of brick wall structures.

2.3. Twin System of Brick Wall and Reinforced Concrete Columns

In this system, vertical load is to be resisted by beam-column system and the horizontal loads are to be resisted by brick walls by way of box-action in plan (Fig 2.2). In this system, column sizes are restricted to, say, 230 mm x 230 mm. Beams are designed as continuous beams on knife-edge supports, i.e.columns being flexible, carry only vertical load and no moment. This system is suitable for four storeys for apartment buildings, the limitations being, firstly, 230 mm x 230 mm column capacity may be fully utilized for the storeys indicated and secondly, the box-action of brick walls under horizontal loads may be fully utilized for the height of four storeys. Beyond four storeys, undesirable stresses may develop in masonry walls due to horizontal loads.This system is suitable where 115 mm thick walls may be required to afford greater use of the covered area and where rooms are large, in which case, system of (Fig: 2.1) of load bearing brick walls will require excessively thick walls, ic.im i mr optimum use of the covered area. This is an innovative structural system. leading to great economy, where applicable

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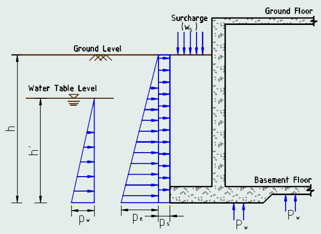
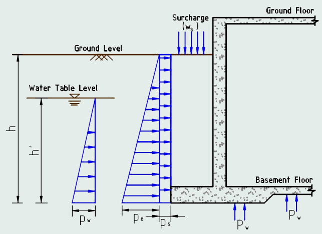


(a) Pressures on a basement retaining wall



(b) Pressures on a basement retaining wall

Figure 2.1: Structural system consisting of load bearing brick walls, (a) Plan of building; (b) Section A-A.

2.4. Framed Buildings

Reinforced concrete frames, provided in both the principal directions, are effective in resisting both the vertical and the horizontal loads (Fig: 2.3). The brick walls are to be regarded as non-load bearing filler walls only. The spacing of frames varying from 4.0 m to 7.0 m or more (7.0 m is relevant for hospital buildings) is closely related to the function of building. The slab thickness should be as close to 100 mm as possible. This can be achieved by providing subsidiary beams in addition to the frame beams.

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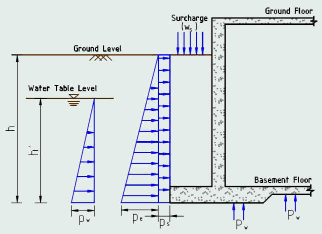
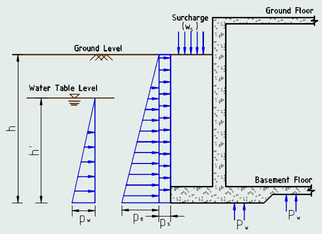


(a) Pressures on a basement retaining wall



(b) Pressures on a basement retaining wall

Figure 2.2: Twin system of brick walls and RC columns, (a)Plan of building; (b) Section A-A.

The finishes and the partition wall should be kept light in weight. This is important for multi-storeyed buildings in order to reduce the dead load. This system is suitable for buildings of more than four storeys. But, in certain blast-or earthquake prone areas, even single or double storey buildings are made framed structures for reasons of safety. Single storey buildings of large storey heights (5.0 m or more), like electric sub-stations, etc. are also made framed structures, as brick wall of large height are slender and these may not be relied on to support vertical loads.
When the lifts are provided, the optimum number of storeys is eight, in order to make full use of lifts. For eight to twelve storeys, it is advisable to avoid all reinforced concrete

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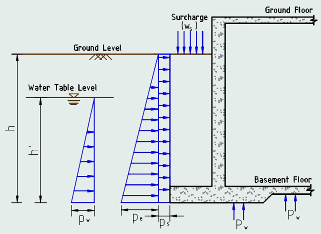
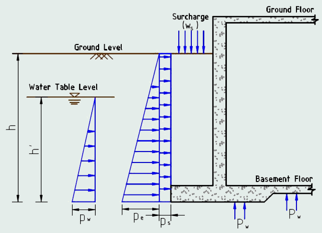


(a) Pressures on a basement retaining wall



(b) Pressures on a basement retaining wall

Figure 2.3: Structural system of RC frames. (a) Plan of building (b) Section A-A.

walls, even for lift-wells, in order to avoid undesirable centres of rigidity, which will interfere with the distribution of the horizontal load to various frames. In earthquake prone areas, columns should be made square in size, as earthquake is to be checked in either principal direction. Rectangular columns should be provided in wind-dominated areas, with the long side of the columns being kept parallel to the short side of buildings. Proper sizing of columns and beams and spacing of frames in either principal direction are crucial aspects affecting economy.

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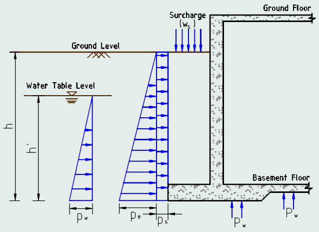

Figure 2.4: Pressures on a basement retaining wall

2.5. Framed Buildings With Shear Walls

When a building exceeds ten to twelve storeys, column and beam sizes workout quite large and there is congestion of steel bars at beam- column junctions. In order to avoid these practical difficulties, shear walls or shear boxes or cores can be introduced in the structural planning of multi-storeyed buildings. Shear walls consume a great quantity of concrete but these relieve columns of most of the horizontal load due to earthquake or wind and in the net result, lead to an efficient and economical structural design.

Straight deep reinforced concrete walls may be provided at the ends of a building with lift walls and/or stair wells in the interior (Fig: 2.4). In general, shear walls shall be so arranged in plan as to attract as much vertical load as possible, for which, the nearby columns are to be omitted and the loads brought to shear walls by means of long-span beams. This system is suitable for 10 to 20 storeys.

A shear core (or box) housing lifts, toilets and other services may be provided preferably placed at the center of the building. This system is suitable for 15 to 40 storeys (Fig: 2.5). In the upper range of storeys (say, 25 to 40 storeys), the core may require to be assisted by other shear walls either in the interior or on the periphery of the building - like four angular walls at corners or a system of closely spaced fins or grid op the

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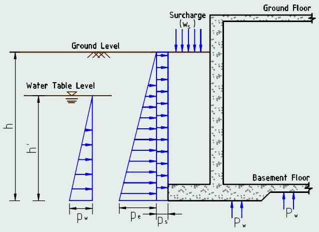

Figure 2.5: Pressures on a basement retaining wall

periphery to act as a core-within-core system (Fig: 2.6). For buildings higher than 40 storeys, multicored systems may have to be devised.

2.6. Selection Of Structural System

For substantial economy to be achieved in the structural design of a building, a correct choice of structural system is more important than designing accurately only the critical sections of members forming a building. A structural system suitable for a low-rise building is not adequate for a high-rise structures.
In the housing sector, load bearing brick walls are widely used. This system is quick in construction and economical in cost. A mixed system of brick walls with reinforced concrete columns is also widely used. Reinforced concrete beam-column system for vertical loads and brick walls box-shaped in plan for horizontal loads are used upto four storeys. For buildings beyond four storeys, reinforced concrete framed structure is economically sound to resist both the vertical and the horizontal loads. With tall buildings, reinforced concrete frames are to be combined with shear walls and/or shear cores in order to get reasonable beam sizes and get reasonable steel consumption in buildings.

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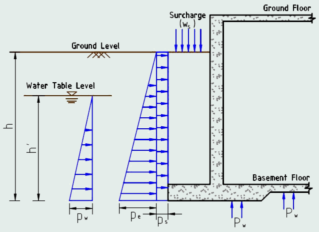

Figure 2.6: Pressures on a basement retaining wall

Inspite of the above general rules, striking variations may occur in practice. In the planning of a six storeyed building with a basement, the frames were put at a spacing of 12.0 m center to center. The number of columns was considerably less. So, shear walls were provided extensively to resist earthquake loading in each principal direction. This added considerably to the cost but it suited the function of the building. In the Srinagar Secretariat building, deep straight shear walls have been provided at the ends of building, although it is a six-storeyed building. The shear walls resist earthquake in the transverse direction, while, in the longitudinal direction, earthquake is resisted by the longitudinal frames.

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3 Types Of Floors

3.1. Introduction

Floors or roofs are structures in horizontal planes, supported on vertical elements like walls and/or columns. These are required to support vertical loads acting at these levels. Vertical loads consist of the dead load of slabs, beams, partition walls, finishes and plaster, etc. together with live loads due to the usage of the floor. Further, a floor is also to act as a diaphragm in its own horizontal plane to hold together all the vertical elements like brick walls, columns and/or shear walls (Fig: 3.1)

This action is similar to the action of bracings at the eaves level in steel structures. This diaphragm action of a floor is, often, not appreciated. But it is basic to the integrity

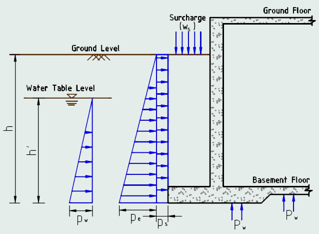

Figure 3.1: Diaphragm action of floor

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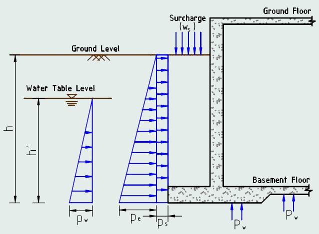

Figure 3.2: Arrangement of one-way siaD panels in a floor plan

of the building as a whole, under horizontal loads like wind or earthquake, which are erratic in magnitude and direction.16 The floor diaphragms are infinitely stiff in their own planes due to the large tn-plan dimensions of buildings. The floors are, however, flexible perpendicular to their own planes and under vertical loads, floors come under bending and shear and undergo deflection.

3.2. One Way Slab Systems

Studies have shown that one-way slab systems resting on beams lead to economy, although the material is stressed in one direction only, the other direction being unstressed. Slab panels with $l_y/l_x \ll 2$, will be designed as one-way slab, in that, the load goes fully in the short direction (Fig: 3.2). Slab thickness is governed by deflection, while the steel area is governed by the bending moment at the center of the short span (l_x). For general economy of structural design, slab thickness shall be kept as small as practicable, i.e. about 100 mm to 150 mm. Minimum slab thickness can be kept as given by IS: 456-1964. For,

$$\text{Single span one way slab : D} = 1/30$$

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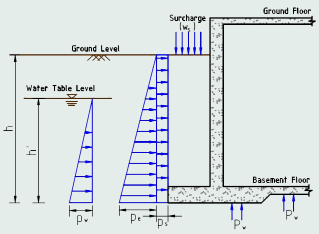

Figure 3.3: Ribbed slab cast on forms removable after the concrete has set(this looks ribbed from below)

$$\text{Continuous span one way slab : D} = \text{l}/35$$

$$\begin{aligned} where,\ l &= \text{span of one way slab} \\ D &= \text{over all depth of concrete slab} \\ l_x &= \text{span in short direction} \\ l_y &= \text{span in long direction} \end{aligned}$$

More steel area may be provided at mid-span to make these slab thicknesses adequate for deflection. One-way solid slabs are suitable for spans 3.0 m to 6.0 m in the short direction. When the span is large or slab thickness works out larger than 200mm (say), it is economical to go in for one-way ribbed slab. The ribbed slab can be constructed in the following ways (in the words of the Code), as given by clause 29.1 of the Code.

(1) As a series of concrete ribs with topping cast on forms which may be removed after the concrete has set (Fig: 3.3).

(2) As a series of concrete ribs between precast blocks which remain part ofthe completed structures, the top of the ribs may be connected by a topping of concrete of the same strength as that used in the ribs (Fig: 3.4).

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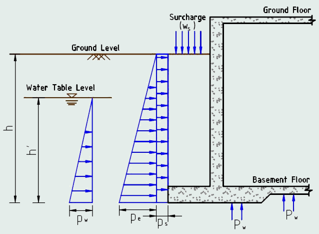

Figure 3.4: Ribbed slab cast on forms removable after the concrete has set(this looks ribbed from below)

(3) With a continuous top and bottom face but containing voids of rectan-gtilai, oval or other shapes (Fig: 3.5).

The main idea of a ribbed slab is to reduce dead load of the slab panel. The self-weight (w_s) per unit m^2 of area of one-way ribbed slab (Fig: 3.3) is given by,

$$\begin{aligned} w_s &= \rho_c \left[D_f + \frac{b_w}{b_f\,(D_f)} \right] \qquad \textbf{(3.1)} \\ where\ \rho c &= \text{unit of weight of concrete.]} \\ D_f &= \text{thickness of topping slab} \\ b_w &= \text{width of ribs} \\ b_r &= \text{center to center distance of ribs} \\ D &= \text{overall depth of ribbed slab} \end{aligned}$$

This value of w_s should be less than the self-weight of the equivalent solid slab for the same span. This can be made clear by an example. Let l = 6.0 m. The solid slab thickness D= l/30 = 600/30 = 20 cm. The self-weight of20 cm thick slab = 0.20 x 2500

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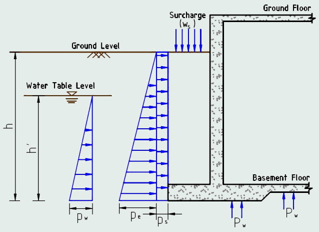

Figure 3.5: Ribbed slab with continuous top and bottom slab (this looks flatfrom bottom).

= 500 Kg/m^2. With b_w =0.15 m, b_f = 0.75 m, D_t = 0.05 m, D = 0.30 m and p_c = 2500 kg/m^3, the self-weight of the ribbed slab is given by (Eq: 3.1),

$$w_s = 2500 \left[0.05 + \frac{0.15}{0.75} (0.3 - 0.05) \right] = 2500 \, (0.05 + 0.05) = 250 \text{ Kg}/m^2$$

The proposed ribbed slab has a self-weight of only 250 Kg/m^2, while the solid.lab will have a weight of 500 Kg/m^2. This is the gain of adopting a ribbed slab m place of the slab. Clause 29.5 of the Code gives that $b_w \nless 65mm$ and $b_r = 1.5m$. Also, it specifics that $(D - D_f \ngeq 4b_w)$. It is seen that for adequancy m deflection, depth of the ribbed slab shall be 25% to 50% more than the depth of tin solid slab for the same span.

Two structural arrangements for a floor with one-way slab panels are given in (Fig: 3.6 and 3.7), in order to illustrate the efficiency of a floor system. The arrangement shown in (Fig: 3.6) is structurally efficient, as long-span beams like BI, B2 and B3 have less loading, while the short-span beam B4 has mote load, leading to a uniform concrete size for all beams, giving an elegant look to the ceiling and also resulting in less consumption of steel reinforcement. In (Fig: 3.7) the floor arrangement has long beams with heavy loading. Resulting in more consumption of steel. Further, with unequal sizes of beam

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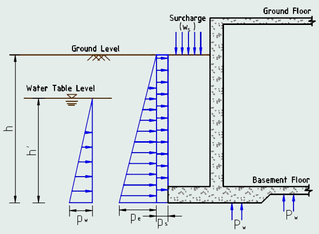

Figure 3.6: Efficiency arrangement of one-way slab panels in floor plan.

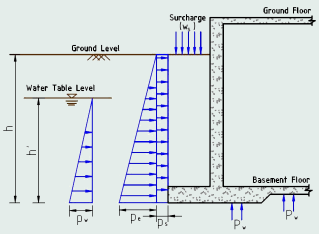

Figure 3.7: Inefficient arrangement of one-way slab panels in a floor plan.

the ceiling may look ugly. It is however, seen that the concrete quantum is nearly the same in these two arrangements.

3.3. Two Way Slab System

Two way slab panels are efficient as the material gets stressed in the two perpendicular directions. This system is suitable for square or squarish panel with $l_x and l_y$. (Fig: 3.8). The slab panel bends in the two cross directions and it assumes a saucer-like deflected

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shape. The load gets divided in the two directions, depending on the ratio of the sides. When $l_x/l_y > 2$, a two-way slab panel tends to behave like a one-way panel, then most of the load goes in the short direction. The two-way slab system is suitable for a panel size upto 6.0 m ×6.0 m, which may give a slab thickness of 150 mm to 200 mm. Minimum slab thickness for two-way solid slabs can be kept as given by IS: 456-1964.

For a panel simple supported on four sides D = $l_x/35$
For a panel with continuous four sides D = $l_x/40$

where l_x = short span of slab panel; l_y long span of slab panel

By putting more steel at midspan in either direction, we may check the slab panels to be adequate for deflection. For an overall economy of the buildings as a whole, we would like to have the minimum slab thickness in order to reduce dead load.

When the panel size is large or slab thickness works out large, two-way added slab may be used to save on the self-weight of the slab-panel(Fig: 3.9). The construction system as given in (Fig: 3.3 to 3.5) for one way ribbed slab panels are valid for the two way ribbed slab panels also. Restrictions on the rib size and spacing as given in clause 29.5 of the Code are valid for two-way ribbed slab panels also. Further ribbed slab panels can be analysed as two-way solid panels, provided the rib spacing (b_f) is not more than twelve times the flange thickness(D_f),in accordance with the clause 23.4 of the Code. Large two-way ribbed panels are regarded as simply supported on all four sides(clause 29.2 of the Code), as it is expensive to design thin ribs for negative support moments. The rib-sections work out well at the mid-span section due to the T-action of the topping slab.The self-weight of a two-way ribbed slab shall be less than that of an equivalent solid slab. As an example, 6.0 m ×6.0 m panel is considered. For a solid slab panel with discontinuous four sides,

$$\mathrm{D} = \frac{600}{35} = 17\text{cm, w} = 0.17 \times 2500 = 425\ \mathrm{Kg}/m^2$$

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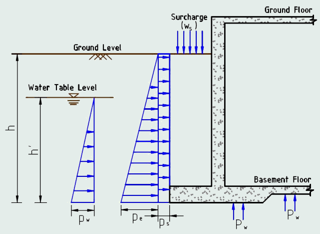

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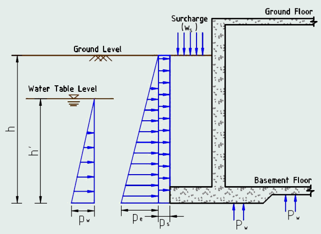

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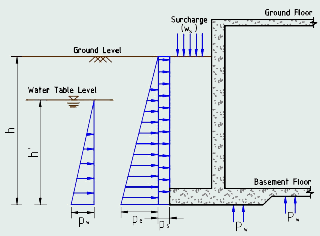

(c) c

Figure 3.8: Two-way slab panel in plan, (a) Plan of slab panel, (b) Section X-X; (c) Section Y-Y

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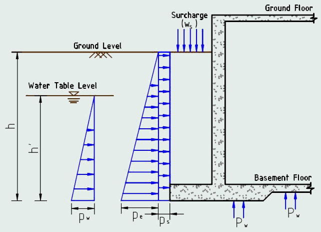

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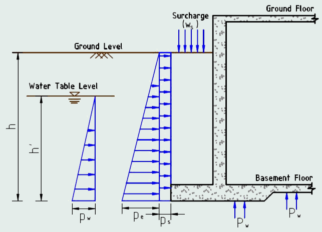

(b) b

Figure 3.9: Two-way ribbed slab panel in plan, (a) Plan; (b) Section X-X

For two-way ribbed square slab with square voids, the self-weight of theslab is given by (Fig: 3.3).

$$w_s = \rho_c \left[D_f + \frac{b_w}{b_f} \left(2 - \frac{b_w}{b_f} \right) (D - D_f) \right] \tag{3.2}$$

with $\rho_c = 2500kg/m^3$, $D_f = 0.08m$, D = 030m, $b_w = 0.15m$, $b_r = 1.0m$ (Eq: 1.5) gives

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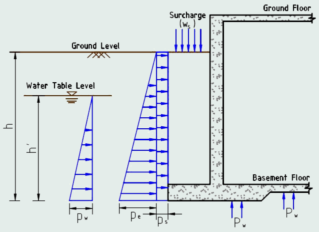

Figure 3.10: Flat slab with drop (without capital),

$$
\begin{aligned}
w_s &= 2500\left[0.08 + \frac{0.15}{1.0}\left(2 - \frac{0.15}{1.0}\right)(0.3 - 0.08)\right] \\
&= 2500\,[0.08 + 0.06105] = 353kg/m^2
\end{aligned}
$$

This value, being less than 425 Kg/m^2, two-way- ribbed slab panel will be beneficial in reducing the dead load. 6.0 m ×6.0 m panel is a border case, where the solid slab panel will be more economical, as it will Save labour in construction. For larger panels like 9.0 m x 9.0 m, solid slab will not do, two way ribbed slab or one-way slab systems have to be thought of.

Whether a slab panel will behave as a one-way or a two-way panel, solely depends on the panel dimensions in plan. When $l_y/l_x < 2$, the panel is a two-way slab. But, when $l_y/l_x > 2$, the panel may be regarded as one-way slab. This is how the slab panel behaves under the vertical loading. There are some engineers, who believe, that by putting steel in a selected direction, they can change the behaviour of the slab panel. This will lead to cracking in the direction, where the normal bending of the slab panel has been disregarded. It is rightly said that the structure will behave the way it is built, rather than the way we will like it to behave. It is, therefore, prudent to respect the natural propensity of the behaviour of a structure, rather than to foist our own ideas or it

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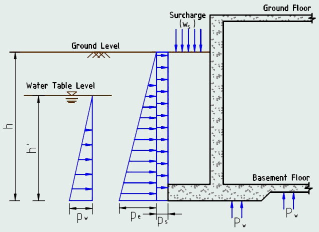

Figure 3.11: Flat slab with drop with column capital

3.4. Flat Slab Systems

Flat slab systems are two-way systems in which the load is resisted in the two principal directions. These are beamless structures in which the slab rests directly on columns. The column supporting a flat slab floor has a tendency toiiiimcIithrough the slab. In order to protect the structure against the punching effect, slab portions around columns are made 25% to 50% thicker than slab (called drops) and/or columns are given brackets all around at their junctions with slab (called capitals). Further, marginal beams are provided at the periphery of the floor in order to stiffen the free edges and also to support the load of the external brick walls. The Code gives rules for the dimensions of drops and capitals. The solid slab with drops with or without column capital (Figs: 3.10 and 3.11) is adequate for slab panels of 6.0 m $\times$6.0 m to 9.0 m $\times$9.0 m, the slab thickness D = l/28, where l is the average of spans in the two directions. An artistic shape which combines the drop and the capital into a common shape (Fig: 3.12) has been used in the Departmental Buildings of Indian Institute of Technology, Delhi.

The Code gives two methods of design of flat slab structures

(1) Emperical method.

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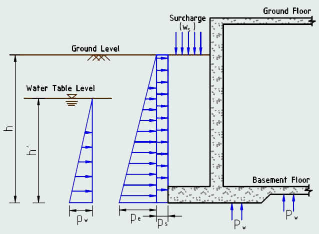

Figure 3.12: Flat slab with an artistic shape of drop and capital.

(2) Equivalent frame method.

The equivalent frame method has a wider application and it can be used to situations of unequal spans as well. The flat slab Systems are very efficient in resisting vertical loads but are weak in resisting horizontal loads. Thus, these systems are best used in short buildings, say, upto four storeys, where the horizontal loads can be resisted by brick-walls enclosing the building. In taller buildings (say, 4 to 8 storeys), the moments due to horizontal loads are to be resisted by frames formed by column strips with the supporting columns. As the depth of the drop slab and the flat slab are less in values, more steel is consumed in resisting horizontal loads. In buidlings of more than eight storeys, shear walls or shear cores are provided to resist the entire horizontals hear so that flat slab system with columns will resist only the vertical load.This arrangement gives an efficient structural system for both the vertical and the horizontal loads.

When the spans are larger than 9.0 m or so, solid flat slabs work out extra thick. So, in order to reduce the dead load, it is better to use two-way ribbed flat slabs with or without filler blocks. The overall depth of ribbed flat slab will work out about 50% to 100% more than that of an equivalent solid flat slab. Further, the rib spacing (b_f) shall be kept at or closer than twelve times the thickness of the topping slab. In the drop areas around the supporting columns, the ribbed slab is made solid to provide resistance to the punching tendency of columns (Fig: 3.13). The column may be provided a capital

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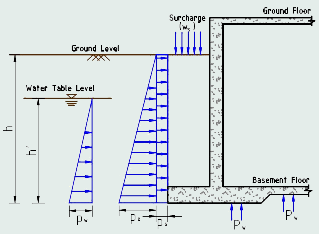

Figure 3.13: Two-way ribbed flat slab with solid drop (without column capital).

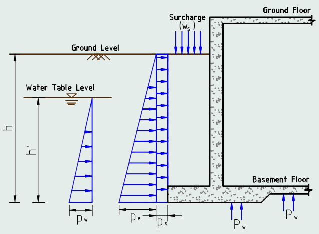

Figure 3.14: Two-way ribbed flat slab with solid drop (with column capital)

to provide additional safety in this regard (Fig: 3.14). These systems are efficient for vertical loads with large dimensions 9.0 m to 12.0 m, the overall depth (D)of panel, being given by (for l ¿ 10.0 m).

$$\begin{aligned} where\ l &= \frac{l_x + l_y}{2} \\ l_x &= \text{span in short direction} \\ l_y &= \text{span in long direction} \end{aligned}$$

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The deflection of flat slab panels has to be checked at the mid-span section of the long span (l_y). This is in contrast to a two-way slab panel, where the critical section for checking deflection is the mid-span of the short side(l_x).

3.5. Flat Plate Systems

A flat plate floor has an elegant looking ceiling, as it consists of a solid floor slab resting directly on columns. Marginal beams are provided on the periphery, to support the external brick walls. The punching tendency of columns through the flat plate is to be resisted by providing steel shear heads around columns, which may be formed by either structural steel I-beams (Fig: 3.15) or reinforcing steel bars (Fig: 3.16). The depth (D) of a flat plate floor, may be given as,

$$\text{where } \ \mathrm{l} = \frac{l_x + l_y}{2}$$

This system may be usefully provided for small spans in either direction, such as 6.0 m $\times$6.0 m or even less. The columns may be with or without capitals. In the former case (Fig: 3.17), shear heads may not be required or the shear heads may work out lighter, leading to economy. For large spans, two way ribbed floor plate system with solid drops will lead to the systems given in (Fig: 3.12 and 3.13). This system is efficient for vertical loads only. For horizontal loads, its behaviour is the same as that of the flat slab systems.

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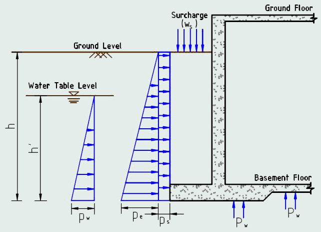

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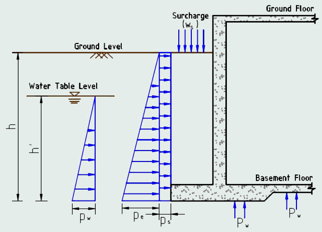

(b) b

Figure 3.15: Flat plate system with structural steel shear head (withoutcloumn capital), (a) Plan; (b) Section X-X

3.6. Grids

When may a two-way ribbed slab be regarded as a grid? When the hollow spaces are less prominent in a voided slab, it is close to a solid slab and it is regarded as a ribbed slab and it can be analysed in the same way as a solids lab provided that the rib spacing (b_f) is less than or equal to twelve times the thickness of the topping slab. Ribbed slabs are suitable for short spans, say, less than 10.0 m. But, when the hollow spaces are prominent, the voided slab is regarded as a grid and its behaviour is different from a

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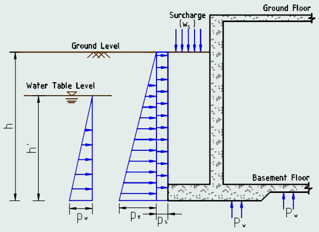

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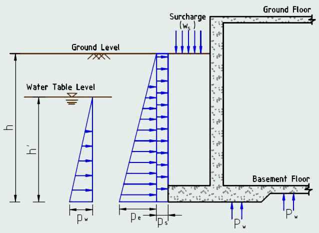

(b) b

Figure 3.16: Flat plate with shear head of steel bars (without column capital). (a) Plan; (b) SectionX-X

solid or a ribbed slab, in that the torsional rigidity is negligible in grids. Grids are suitable for large panels with spans greater than 10.0 m. Also, the rib spacing (bf) can be more than 12 D_f.A grid panel can be analysed as a two way slab panel or as a flat slab with solid drops around columns. The overall depth (D) of a grid panel is given as

$$D = \frac{l}{20}$$

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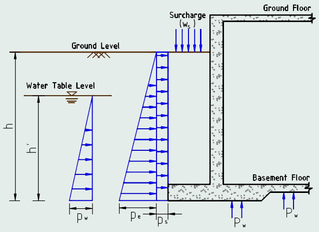

Figure 3.17: Flat plate with column capital

where, l = $(l_x + l_y)/2$ and l is more than 10.0 m. Due to large spans involved.M20 concrete mix is preferred to M15 in the grids. The detailed analysis of grids is given in the Manual. Grids with large values of b_f, are easy to construct at site, as the shuttering in such cases is economical.

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4 Types Of Stairs

4.1. Introduction

Stairs are inclined slabs connecting any two successive floors. The riser (R) is kept at 150 mm and the tread (7) at 300 mm. These values give an easy and comfor table access to a floor level. When the space available is tight, the riser may go upto a maximum value of 200 mm and the tread to a minimum value of 250 mm. The width (W) of stairs, as also that of landing, generally varies from 900 mm to 1800 mm. In general, the architect plans a stairs in a given area, while the str uctural engineer is to fix its waist slab thickness, steel bars and designs the supporting elements like beams and columns.

4.2. COMMON TYPES OF STAIR

A two-flight stairs with an intermediate landing is the most common type of stairs (Fig: 4.1). Structurally, each flight is designed as a one-way slab, sup ported on the floor beam on one side and on a landing beam on the other side. Due to the inclination of the flight slab, the dead load, though acting vertically down, gets increased to a value

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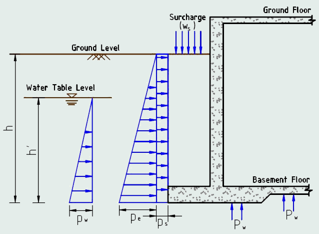

Figure 4.1: Plan of two-flight stairs

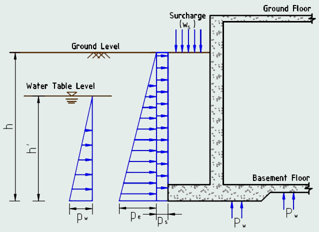

Figure 4.2: Plan of two-flight stairs with central flight beams

$\dfrac{W_d}{\cos\theta}$, where W_d = dead load intensity, θ = angle of inclination of the flight slab to the horizontal, $tan\theta = \dfrac{R}{T}$.

The waist slab works out, in most cases, to be 150mm to 200mm thick.In certain cases, waist slab works out to be about 300mm thick, which doesnot appear elegant to the view. In such cases, a small amount of compressionsteel may be permitted at the mid-span section and the waist slab thicknesscan then be brought down to a value of 200mm. This arrangement is valid forspans 4.0m to 6.0m. When the spans are longer than 6.0m, central beamsmay be put to support flight slabs, which may be cantilevered out on

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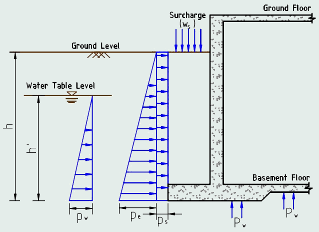

Figure 4.3: Plan of wide flights stairs with two central beams in each flight.

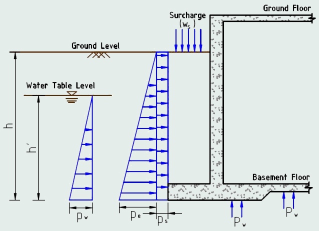

Figure 4.4: A three-flight stairs.

eitherside (Fig: 4.2). For extra wide flights, two central beams may also be providedto get an economical structural design (Fig: 4.3).The flights with centralbeams do not look good to the view, where the space is restricted. In suchsituations, the floor and the mid-landings are made to span cross-wise, so that the flight span is reduced, giving a reasonable value for waist slab thickness. The flight span in such cases (clause32.1 of the Code) works out to be equalto the going plus 1.0 m or half the landing width (whichever is less) on eitherside. Generally, a staircase needs four columns in plan. Of these, two columnsare a must for supporting the landing beam.

Three or four-flight stairs are also planned with an open well in the central area (Fig: 4.4 and Fig: 4.5). In Fig: 4.4, the two long flights of span ly maybe considered as main

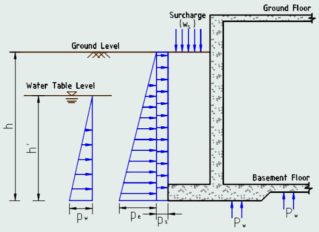

Figure 4.5: A four-flight stairs in a square room in plan.

flights supported on ends by floor beam on one sideand by the sloping landing beam on the other. The intermediate short flight ofspan $(L_x - W)$, may be regarded as a suspended flight, supported on the two main flights. In a square grid l x l,a four-flight stairs is planned in (Fig: 4.5), with sloping beams being provided along the profile of the flights. Each flight may be regarded as a simply supported one, with a span of l(m), with landingloading being taken half its value in each direction. This is explained in clause 12.2. of the Code. When the staircase room is rectangular in plan $(l_y > l_x)$, l_y-Ilights can be regarded as main flights, which support the short l_x-flights, being regarded as suspended flights (Fig: 4.6).
When the columns supporting the mid-landing are not preferred, themid-landings can be supported by hangers suspended from a floor beamabove, in order to get free space below the mid-landing. Thus, in practice, there are many ways of planning stairs and this planning aspect is taken care by the architect concerned.

4.3. CENTRAL-WALL TYPE STAIR

Central-wall type stairs have been used in Chandigarh Secretariat building and in many buildings of Indian Institute of Technology, Delhi. The steps are cantilevered out on

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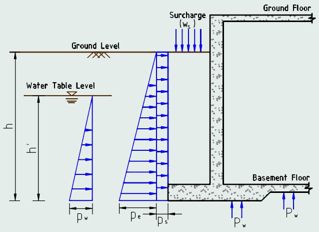

Figure 4.6: A four-flight stairs in a rectangular room in plan.

either side of the central wall and the landing slabs too arecantilevered out on either side of the central beams (B1 and B2), which are cantilever beams from the central wall (Fig: 4.7). The wall shall have to be reinforced concrete of 230 mm or 300 mm in thickness. Some openings of ovalor other artistic shapes can be made in the central wall, in order to save on thematerials and also it helps one to know of the movement of persons on the other side of the wall. For checking slenderness of the wall, it is assumed that stairs slab of width W holds the wall by a serpentine rope-like action so that effective height of the wall equals the storey height of the building. For this reason, we insist on joining the floor landing of the stairs to the main floorslab. It is, thus, not advisable to separate out the central-wall stairs from the main building by an expansion joint.

4.4. Central-column type stairs

In servants quarters, a central-column type stairs is commonly provided. It is also called a spiral stairs. It is a circular stairs with a width of minimum 750mm The steps are made precast with a central hole. These precast steps are joined with mortar, one on top of another, giving rise to a vertical circular hole in which the column steel shell is

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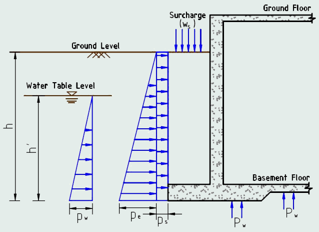

Figure 4.7: Central-wall type stairs.

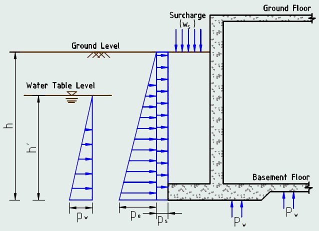

Figure 4.8: Spiral stairs, (a) Plan of precast step; (b) Section 1-1.

lowered and it is concreted to give the sprial stairs (Fig: 4.8). The structural design of this stairs is given by Jain and hrikrishna Vol. I.

In main buildings also, a central tube column or a lift-well of reinforced concrete walls may be used to support stairs on all sides, with cantilever stepscoming out of the lift walls of reinforced concrete (Fig: 4.9). For cantilever steps, it will be advisable to have full glazing on the outside, in order to reduce load. When outside walls may be required, we may think of having beams and columns on the outside, then the stairs slab will be spanning one-w ay between the outside, sloping beams, and the lift walls (Fig: 4.10). Theh lift walls will work out to be 150 mm or 230 mm thick in reinforced concreteand these systems given in Fig: 4.9 and Fig: 4.10 work out very cost effective.

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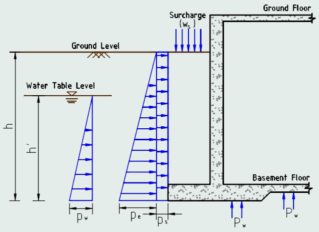

Figure 4.9: Central tube column stairs.

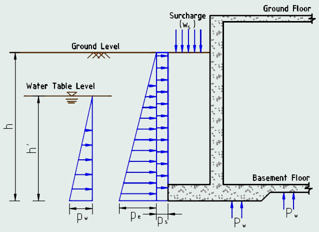

Figure 4.10: Central tube column stairs - another alternative.

4.5. Slabless stair

types given in Fig: 4.1, Fig: 4.4, Fig: 4.5, Fig: 4.6, Fig: 4.7, Fig: 4.9 and Fig: 4.10 can be madedablcss, in that, the waist slab can be omitted (Fig: 4.11a), and the steps cani a made in reinforced concrete with uniform thickness and with steel arrange-m, ni of closed stirrups in both the rise and the tread portions (Fig: 4.11b).I line are many methods given for the design of the slabless stairs.19-20 But the" i icsuit has been given,21 in that, it can be designed in the way of any com-......slairs with the depth and the steel calculated as for any solid slab. OnlyII a -.Icel arrangement has to be put in

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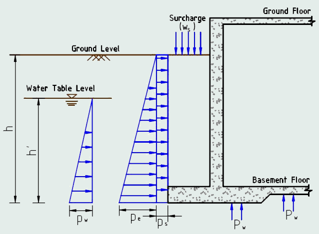

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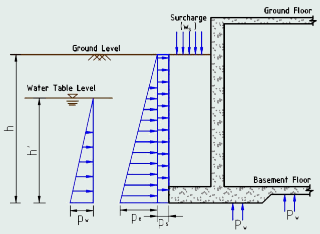

(b) b

Figure 4.11: Slabless stairs, (a) Common stairs (solid slab with steps work); (b) slabless stairs

loops in both the rise and the tread por-'i iir. and also in landing slabs. The slabless cantilever steps-are more effectivei. the depth available for resisting moment equals(R + D)(Fig: 4.11b),iln

4.5.1. Example of Slabless Stairs

This example is taken from reference 20, wherein it is taken as both ends fixed. With simply supported ends,

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$$M = +\frac{WL^2}{8}$$

Referring to Fig: 4.1lb

10/100 c/c closed stirrups shall be provided in all treads and,risers with3 ϕ 8 bars at each comer and 2 ϕ 8 bars extra at each mid-width of treads (Fig: 4.11b).
With both ends fixed, the moment in the reference 20 is given at 27600tb in/ft, which is equivalent to 10.4 tm/m. In practice, it is difficult to ensure fixity at ends. A fixed-end moment at $\frac{WI^2}{12}$ in our example is equal to 1.25x3.52/12 = 1.28 tm/n, which is not far from the value given in the paper. The author prefers to have simply supported ends for this type of stairs.

4.6. Helicoidal Stair

Helicoidal stairs are elegent access ways, generally provided in prestigious buildings and in posh bungalows. These can be circular or elliptical in plan and these can be a part of a circle or even a full circle or a full ellipse in plan (Fig: 4.12). The methods of analysis given in literature2 refer to helicoidal stairs, circular in plan. For elliptical plans, a circle of an average diameter may be used as an approximation. An approximate method which has been sugested in reference 22, is that the helicoidal stairs may be analysed as a beam circular in plan, for which the methods of Bergman and Salvador are eaisly available. These methods do not give horizontal support moments, which may be taken as twice the value of the vertical moments at supports. Each section of the e helicoidal stairs has to be designed for vertical moment, vertical shear, torque and a horizontal moment. It is a space structure and it is mandtory or reasons of equilibrium, to design this structure with both ends fixed. An example of helicoidal stairs is given below to illustrate the above ideas.

In Indian Institute of Technology, Delhi, these stairs have been used insome places. A full circular helicoidal stairs with the central beam supportingisolated steps has been

T= 0.25 m, R =0.15 m, D = 0.15 m, L =14 x 0.25 = 3.5 m

Selfweight of slabless stairs = $\frac{P}{T}$

P = 0.15(.15 + .15) ×1.0×2500 =112.5 kg/m

+ (.25 - .15) ×.15 ×1.0x2500 = 37.5 kg/m

= 150.0 kg/m

$\frac{P}{T} = \frac{150}{0.25}$ = 600Kg/m^2

W: Selfwight of slabless stairs = 600 Kg/m^2

Finish .05 ×2500 = 125Kg/m^2

Plaster = 25Kg/m^2

DL = 750Kg/m^2

LL = 550 Kg/m^2

W = 1250 Kg/m^2

Moment at centre = $\frac{WL^2}{8} = 1.25 \times \frac{(3.5)^2}{8}$ = 1.91tm

Withb =100 cm, D= 15 cm, d=13 cm

K = $\frac{Mu}{bd^2} = \frac{1.5 \times 1.91 \times 10^5}{100 \times 13^2 \times 10}$ = 1.7N/mm^2

For M15, Fe415, Design Aids58 give,

A_{st} = 0.558 ×13 = 7.25 cm^2/m

Provide by 10/100 c/c = 7.85 cm^2/m

Distribution steel = 1.2 ×15 = cm^2/m

Provide by 8/200 c/c = 2.51 cm^2/m

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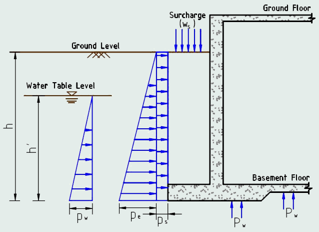
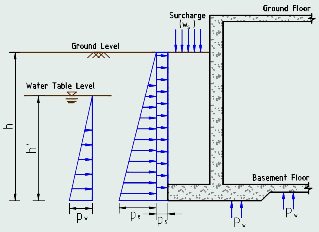


(a) a



(b) b

Figure 4.12: Helicoidal stairs, (a) Plan of helicoidal stairs; (b) Elevation ofm in, (c) Section X-X; (d) Alternative sections of stairs

given in Departmental Buildings, in IIT Delhi. Normally, a section of uniform depth of 150mm to 200mm is provided in practical designs of slab-type helicoidal stairs. In Ashok Hotel, New Delhi, asemi-circular helicoidal stairs has been provided with a cross-section of variable thickness, giving less thickness on the outside and more thickness in thecentre, for a better view (Fig: 4.12d). Because of the complex nature of forcesacting on sections of a helicoidal stairs, it is advisable not to use slabless stairsin these cases.

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4.6.1. Example of Helicoidal Stairs

A helicoidal stairs was planned in a building to connect the ground floor withthe first floor in a residential building, with the following data (Fig: 4.12):

(1) Data

b = 4' - 0" = 1.2 m

H = 10' - 6" = 3.2 m

R = 4' - 6" = 1.35 m

$2\beta = 360 - 120 = 240^0$

$$\text{arc length in Plan} = 2\pi R \frac{2\beta}{360} = 2\pi \times 135 \times \frac{240}{360} = 5.65\text{m}$$

$$\tan^{-1}\alpha = \frac{H}{\text{arc length in plan}} = \frac{3.2}{5.65} = 0.56637$$

$\alpha = 29.5^0$, $\cos\alpha = 0.8704$

Tread T = 0'-10" = 0.25m

Riser $= 0.25 \tan 29 \cdot 5^0 = 0.25 \times .56637 = 0.15$ m

Let the waist slab thickness be 0.15 m (i.e h =0.15 m).

(2) Loading

Selfweight 0.15 ×2500 = 1.2 Kg/m^2

4cm finish 0.04 ×2500 = 100 Kg/m^2

Plaster = 25 Kg/m^2

Railing = 25 Kg/m^2

$$\text{DL} = 525/\cos\alpha = \frac{525}{0.8704} = 600\ km/m^2$$

LL = 300 Kg/m^2

W = 900 Kg/m^2= 0.9 t/m^2 of plan area

W_1 = Wb = 0.9 ×1.2 = 1.08 t/m

$W_1 R^2$ = 1.08 ×$(1.35)^2$ = 2.0

(3) Analysis as a fixed curved beam by Bergman's method : From Table 10.1(Ref 24), for

$\frac{b}{h} = \frac{1.2}{0.15} = 8$

K = 0.64

From (Fig: 4.10 d) (Ref. 24), for

K = 0.64 $\beta = 120^0$, U = 1.28

Midspan $\phi = 0$

$Mr_c = W_1 R^2 (U - 1)$

$= 2.0(1.28 - 1.0) = 0.56$ tm

$Mt_c = 0$

$V_c = 0$

Support: $\phi = \beta = 120^0$ (2.094 radians)

$Mr_\phi = W_1 R^2 \left(U \cos\phi - 1\right)$

$= 2.0(1.28\,(-0.5) - 1.0) =$ -3.3tm

$Mt_\phi = W_1 R^2 \left(U \sin\phi - \phi\right)$

= 2.0(1.28×0.866 - 2.094) = -3.1t

Horz. moment at supports M_H = 2 × Vert. moment at supports
2×3.3 = 6.6tm

(4) Design:(Section 120×15) Support:

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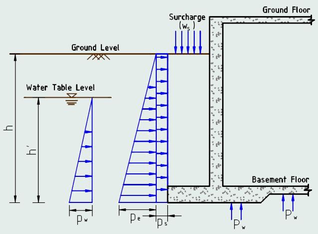
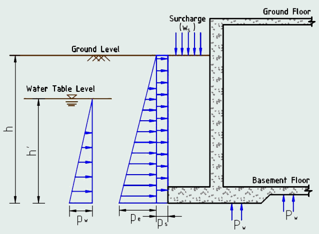
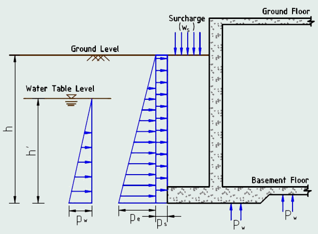


(a) a



(b) b



(c) b

Figure 4.13: Details of helicoidal stairs of Example 4.6. 1. (a) Plan of heli-coidal stairs); (b) Section X-X (midspan); (c) Section Y-Y (support TS).

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(a) Vertical moment and torque

Mt_ϕ = 2.0 tm which is equivalent to ,

$V_e = \dfrac{1.6 \times 2.0}{1.2} = 2.7t$

$M_e = +\dfrac{2.0}{1.7}\left(1 + \dfrac{0.15}{1.2}\right) = \pm\ 1.3$ tm

M_r = -3.3 tm

M design = -3.3±1.3 = -4.6tm, b=120cm, d=12cm

$\text{K} = \dfrac{1.5 \times M \times 10^5}{120 \times 12^2 \times 10} = 0.868M(tm) = 4.0Nmm^2$

$d^1/d = 3/12 = 0.25,\ A_{st} = \dfrac{120 \times 12}{100}pt = 14.4pt = 19.87cm^2$

$Asc = 14.4pc = 10.37cm^2$

Top bars 10 nos. 16 bars = 20.10 cm^2

Bott. bars nos. 16 bars = 10.05 cm^2

(b) Shear = 3.1 +2.7 = 5.8 t

$T_v = \dfrac{1.5 \times 5.8 \times 1000}{120 \times 12 \times 10} = 0.60Nmm^2$

For p_t = 1.4, T_c = 0.67Nmm^2, Safe

Min $\dfrac{V_{us}}{d}$ = 0.0348 b or d whichever is less

= 0.0348 ×12 = 0.42KN/cm

2 legged stirrups 8/150 THRO = 2.42 KN/cm

(c) Horz. momentMh = 6.6tm, b = 15 cm, d = 117 cm

$K = \dfrac{1.5 \times 6.6 \times 10^5}{15 \times 117^2 \times 10} = 0.48N/mm^2$

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$A_{st} = 0.144 \times 15 \times \dfrac{117}{100} = 2.5cm^2$

Min Ast $= 0.205 \times 15 \times \dfrac{117}{100} = 3.6cm^2$

2 ϕ 16bars(each face) = 4.02 cm^2, provided extra

Midspan: $M_t = 0.56$tm

$A_{st} = 0.144 \times 120 \times \dfrac{12}{100} = 2.07cm^2$

5 $\overline{\phi}$ 16 top and bottom provided for convenience in detailing.

On supports 5 $\overline{\phi}$ 16 extra bars shall be provided on top to give 10 $\overline{\phi}$ 16 bars at the top. Stirrups at 8/150 c/c are provided through. The bars at either support shall be fully anchored into the supports to provide fixity at the ends, which is so essential for the equilibrium safety of the structure. The reinforcement details are shown in Fig: 4.12A. It is seen that the approximate method suggested here may lead to a very conservative and expensive design in helicoidal stairs of large diameters. So, recourse should be taken to strict methods of Chatterjee and Cusens, for economy in design. It may be noted that Cusens curves save considerable time and labour of a designer.

4.7. FREE-STANDING STAIRS

Free-standing stairs provide also elegant access ways between floors, giving a free-floating feeling. In plan, it is similar to a two-flight common stairs, with its landing remaining completely unsupported (Fig: 4.14). It is a space structure and it is necessary to assume both its ends to be fixed for a safe analysis. The behaviour of the free-standing stairs is seen to be the cantilever action of boththe flights, with the landing slab connecting the flights, which makes it a triangular frame in space and this reduces the cantilever action of flights considerably. When the distance (a) between the flights in plan is

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small, plane frame analysis can be made to apply. There are excellent papers which give different approaches for the analysis of this neat structure. The approach given here is summarized below.

(1) We assume a fictitious support at the junction of flights of the landing slab and design the two identical parts of the structure for vertical loads (Fig: 4.15 a). The two parts of the structure are A-B-C and A'-B'-C' (Fig: 4.14). Wethen find the reaction R_b at the fictitious support.

(2) Now we remove the fictitious support and regard the reaction R_b as vertical load acting downwards, which causes compression in the lower flight and tension in the upper flight, which may be found by truss analogy by applying Sine Rule (Fig: 4.15 b).

(3) In the truss system of (ii), point B is located at one point. But it is actually located at two points in plan B and B', (w +a) distance away from each other (Fig: 4.14). This aspect causes horizontal moments in flights (Fig: 4.15 c).

This completes the analysis. All the three members the lower flight, landing slab and the upper flight are to be designed for the action of forces given by the analysis. The depth of slab can be kept uniform at 200 mm to 300 mm. The landing slab thickness can be reduced at its free edge (not less than 150 mm). The junction line B-B' is stiffened by providing extra top and bottom bars. The flight bars are to be well anchored into the supporting beams at the two floor levels and these beams are to be carefully designed to resist all incumbent loads from the flights. The lower flight is subjected to an axial compression with biaxial bending, while the upper flight has to resist axial tension with biaxial bending. The landing slab has to be stiff in its own plane in order to effectively connect the two out-of-plane flights. For this reason, the value ofa,the clear distance between the flights in plan is to be made as less as practicable, say 150 mm to 300 mm. When the distanceais required to be made substantial or a three-flight free-standing stairs are to be planned (Fig: 4.16), then its action can be simulated by an equivalent helicoidal stairs, circular in plane. An equivalent circle connecting points A, B, D", B' and A' (Fig: 4.16) with a central subtended angle 2β can be considered for analysis of

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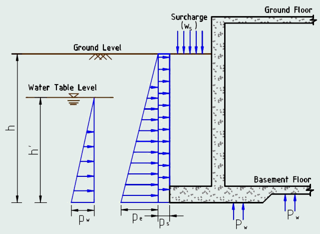
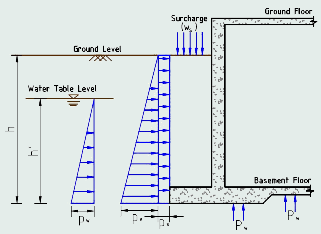


(a) a



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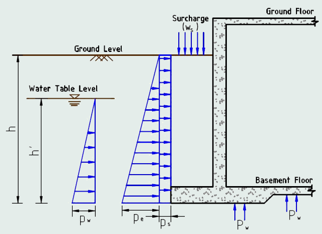

(c) b

Figure 4.14: Free-standing stairs, (a) Plan of free-standing stairs; (b) elevation of free-standing stairs

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the equivalent helocoidal stairs. This is expected to lead to a reasonably safe design of the three-flight free-standing stairs. An example of a two-flight freestanding stairs is given below.

Some authorities consider the effect of various positions of live loadon flights and the landing and they also consider the second order effects ofthe deflected shape of the structure. But both these effects are small in practical structures.

4.7.1. Example Of A Free-Standing Stair

A free-standing stairs is planned between the ground and the first floor of abuilding with the following data (Fig: 4.17).

(1) Data

$$R = 179\text{mm}$$
$$H = 4.6\text{m}$$
$$T = 275\text{mm}$$
$$\text{No. of risers} = \frac{4.6}{0.179} = 25.7 = 26$$
$$W = 1200\text{mm}$$
$$\text{No. of treads in each flight} = \frac{26}{2}+1=14$$
$$D = 200\text{mm, lenght of each flight } 14 \times 0.275 = 3.85\text{m}$$
$$LF = \text{Lowerflight}$$
$$UF = \text{Upperflight}$$
$$L = \text{Landing}$$

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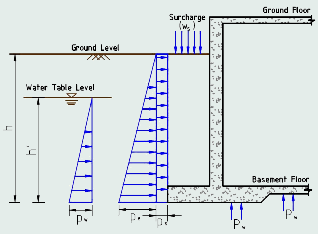
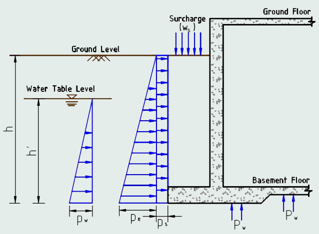


(a) a



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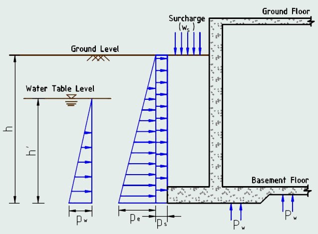

(c) b

Figure 4.15: Analysis of free-stadning stairs, (a) Fictitious support at line B-B'; (b) Truss action of flights; (c) Horizontal moments in flights.

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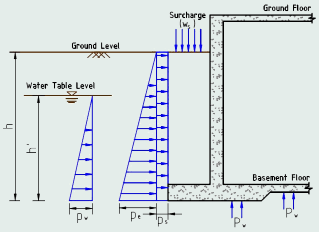

Figure 4.16: Three-flight free-standing stairs.

From Fig: 4.17b, $\tan\theta = \dfrac{2.3}{3.85} = 0.5974$

$\theta = 30.85^0$, $\cos\theta = 0.8585$, $\sin\theta = 0.5128$

(2) Loading

L: Selfweight 0.20 $\times$2500 = 500 Kg/m^2

4cm finish 0.04 $\times$2500 = 100 Kg/m^2

Plaster = 25 Kg/m^2

Railing(say) = 25 Kg/m^2

DL = 650 Kg/m^2

LL = 500 Kg/m^2

TL = 1150 Kg/m^2

LF, UF : Selfweight 0.20 $\times$2500 = 500 Kg/m^2

4cm finish = 100 Kg/m^2

Plaster = 100 Kg/m^2

Railing = 25 Kg/m^2

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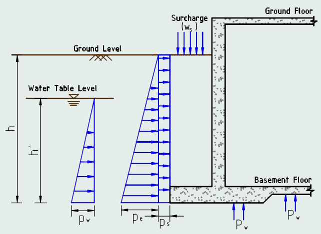
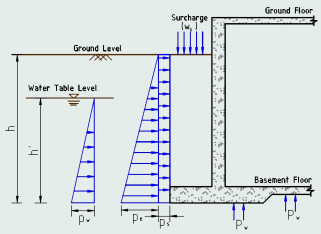


(a) a



(b) b

Figure 4.17: Planning details of a free-stairs of Example 4.7.1. (a) Plan ofstairs; (b) Skeletal elevation of stairs

$$\text{DL } \frac{650}{\cos\theta} = \frac{650}{0.8585} = 760 \text{ Kg}/m^2$$

$$\text{LL} = 500 \text{ Kg}/m^2$$

$$\text{TL} = 1260 \text{ Kg}/m^2$$

(3) Analysis

(a) Assume a fictitious support at B and analyse two continuous beams ABCand

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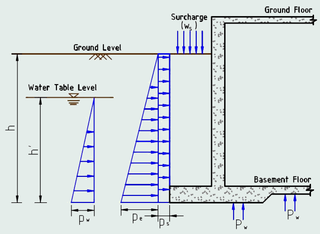
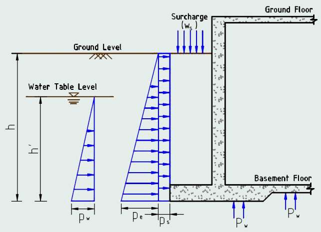
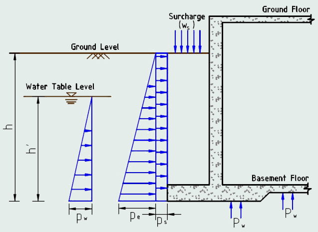


(a) a



(b) b



(c) b

Figure 4.18: Bending analysis of free-standing stairs by moment distribution. (a) Analysis of flight A-B-C; (b) Analysis of flight D-B-C.

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DBC by moment distribution as given in Fig: 4.18a, b

$$W_1 = 1.26 \times 1.2 = 1.512t/m$$
$$W_2 = 1.15\left(W + \frac{a}{2}\right) = 1.15\left(1.2 + \frac{0.3}{2}\right) = 1.5525t/m$$

In evaluating W_2, the portionaof the landing slab is considered as spanning crosswise.

Sum of reactions at B = $(2.62 + 1.86) = 4.48$t
Total load of one flight = $(3.2 + 4.48) = 7.68$t
Total load of both flights = 7.68 ×2 = 15.36t

(b) Remove the fictitious support at B and the reaction at B is applied at B tobe resisted by LF and UF by way of truss action.

R_B = 4.48 ×2 = 8.96t

$\theta = 30.85^0$

$2\theta = 61.7^0$

$\cos\theta = 0.8585$

$\sin 2\theta = 0.8805$

Fig: 4.19 gives by Sine Rule.

$$\frac{C}{\sin(90-\theta)} = \frac{T}{sin(\sin(90-\theta))} = \frac{R_B}{\sin 2\theta}) = \frac{8.96}{0.8805} = 10.181$$

C = 10.18 $\cos\theta$ = 8.74t

T = 10.18 $\cos\theta$ = 8.74t

Referring to Fig: 4.19, we get

At A : H' = C$\cos\theta$ = 8.74×0.8585 = 7.50t

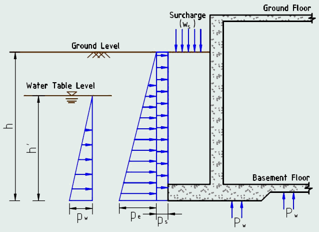

Figure 4.19: Truss analysis of free-standing stairs by Sine Rule.

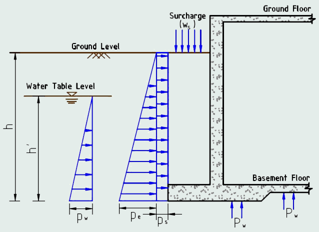

Figure 4.20: Overall equlibrium of the structure.

V $=$ Csin θ $=$ 8.74 $\times$0.5128 $=$ 4.48t

At D : H" $=$ Tcos θ $=$ 7.5t

V" $=$ Tsin θ $=$ 4.48t

Total V'+V" $=$ 4.48 +4.48 $=$ 8.96t

Total shear at supports $=$ 3.2 $+$ 3.2 $=$ 6.4t

Check: Total load $=$ 8.96 $+$ 6.4 $=$ 15.36t

Total load $=$ 5.82 $+$ 5.82 $+$ 3.72 $=$ 15.36 $=$ 2V

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or V = 7.68t

Overturning moment (OTM) about line AD = 5.82 ×2 ×$\dfrac{3.85}{2}$

$+3.72\left(3.85+\dfrac{1.2}{2}\right)$ - 2Mv = 22.4 + 16.6 = 39.0 - 4.48 = H ×4.6

or, H=$\dfrac{34.52}{4.6}$ = 7.5t

Horizontal moment in flights (LF or UF) = G$\dfrac{c}{2}$ = 7.5×$\dfrac{1.5}{2}$ = 5.62tm

(c) Design (M15, Fe415)

- Bending of flights and landing in vertical plane

A : $(LF in compression)$

M_v = 2.24 tm (Fig: 4.18 a)

$$\frac{Lf}{D} = \frac{1.2 \times 4.48}{0.20} = 26.9 > 12$$

Slenderness moment M_a, 20 = KP = 20 × $(26.9)^2/2000$

= 0.072 KP

= 0.072 × 1 × 8.74 = 0.63 tm

M_v = 2.24 + 0.63 = 2.87 tm, C = 8.74 t

Section 120 ×20 b=120cm D=20cm

$$\frac{P_u}{f_{ck}.bD} = \frac{1.5 \times 8.74 \times 1000}{150 \times 120 \times 20} = 0.036$$

$$\frac{M_u}{f_{ck}.bD^2} = \frac{1.5 \times 2.87 \times 10^5}{150 \times 120 \times 20^2} = 0.06$$

$$\frac{d'}{D} = \frac{2.4}{20} = 0.12 = 0.15$$

For $A_{sc} \neq A_{st}$, using Manual chart No. 7.7, we get

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$\frac{d}{D}$ = 0.85, Fe415, X = 0, n = 0.3, $\frac{r}{f_{ck}} = 2 \times 10^{-3}$

r = 0.2 × 15 × 10^{-3} = 0.003

$A_s = 0.003 \times 120 \times 20 = 7.2\ cm^2$, $A_{st} = 7.2\ cm^2$, $A_{sc} = 0$

Provide 10 Nos. $\underset{\phi}{-}$ 10 bars, @ top give 7.85 cm^2

Or, using Design Aids,

$$K = \frac{1.5 \times 2.87 \times 10^2}{120 \times 17.6^2 \times 10} = 1.16\ \mathrm{N}/mm^2$$

$A_{st} = 0.357 \times 120 \times \frac{17.6}{100} = 7.54\ cm^2$

Provide 10Nos. $\underset{\phi}{-}$ 10 bars, @ top = 7.85 cm^2

A - B : M_v = 1.13 tm (Fig. 4.17a) + 0.63 = 1.76 tm

C'= 8.74 t

$\frac{P_u}{f_{ck}.bD} = 0.036$

$\frac{M_u}{f_{ck}.bD^2} = \frac{1.76}{2.86} \times 0.06 = 0.037$

$\frac{r}{f_{ck}} = 0.1 \times 10^{-3}$

r = 0.1 ×10^{-3} ×15 = .0015

A_s = .0015 ×120 ×20 = 3.6 cm^2

Provide 5 Nos. $\underset{\phi}{-}$ bars, @ bottom = 3.92 cm^2

B : M_v = -1.12 - 0.63 = -1.75 tm

C = 8.74 t

Provide 5 Nos. $\underset{\phi}{-}$ bars, @ top (result as per A-B).

Max shear = 3.2 t

$T_v = \dfrac{1.5 \times 3.2 \times 1000}{120 \times 18 \times 10} = 0.22$ N/mm^2

$T_c = 0.35$ N/mm^2(considered as minimum)

Min $\dfrac{V_{us}}{d}$ = 0.0348 b or d which ever is less

= 0.0348 ×18 = 0.626 KN/cm

Nominal stirrups 8/150 provided of capacity = 2.42 KN/cm

D: M_v = 2.24 tm

T = -8.74 t (no slenderness effect in tension members)

$\dfrac{P_u}{f_{ck}.bD} = \dfrac{1.5 \times 8.74 \times 10^3}{150 \times 120 \times 202} =$ -0.036

$\dfrac{M_u}{f_{ck}.bD^2} = \dfrac{1.5 \times 2.24 \times 10^5}{150 \times 120 \times 20^2} = 0.048$

Chart 7.7 of Manual gives

$\dfrac{r}{f_{ck}} = 0.25 \times 10^{-3}$, X = 0

r = 0.25 × 10^{-3} × 15 = 0.00375

$A_s = 0.00375 \times 120 \times 20 = 9.0\ cm^2$

5 $\overline{\phi}$ 10 bars at top + 5 $\overline{\phi}$ 12 bars at top

3.92 + 5.65 = 9.57 cm^2

D - B : M_v = +1.13tm T = -8.74 tm

$\dfrac{P_u}{f_{ck}.bD}$ = -0.036

$\dfrac{M_u}{f_{ck}.bD^2} = 0.024$

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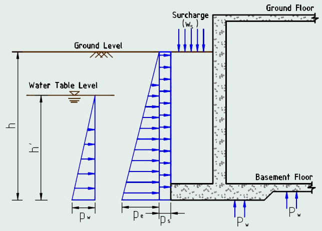
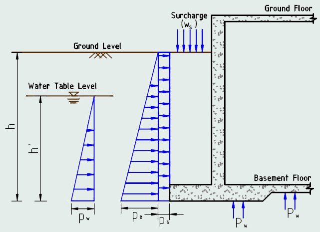
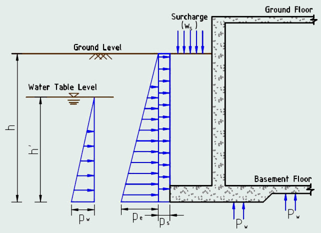


Figure 4.21: Results of design showing top and bottom bars at critical sections.



(a) a



(b) b

Figure 4.22: Plan of stairs showing extra bars for honzontal bending of flights, (a)Plan; (b) Section X-X.

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$\frac{r}{f_{ck}} = 0.2 \times 10^{-3}$

r = 0.003, A_s = 7.2 cm^2

10 $\overline{\phi}$ 10 bars at bottom give area of 7.85 cm^2

B : M_v = -1.12 tm, T = -8.74t

10 $\overline{\phi}$ 10 bars at top (design as per D-B)

The results of design are shown in Fig.(Fig: 4.21).

- Horizontal bending
  M_H= 5.62 tm, b = 20 cm, D = 120 cm, d = 117 cm
  K = $\frac{1.5 \times 5.62 \times 10^5}{20 \times 117^2 \times 10}$ = 2.10 cm^2
  A_{st} = 0.09 $\times$ 20 $\times$ $\frac{117}{100}$ = 2.10 cm^2
  2 $\frac{}{\phi}$ 12 E.F = 2.26 cm^2, nearly OK
  These bars are shown in Fig.(Fig: 4.22)

- Reactions for design of supporting beams.These are given in (Fig: 4.23). The supporting beams at the two floor levels are to be designed for these loads in addition to other loads coming on these beams.

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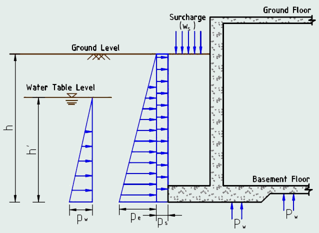

Figure 4.23: Reactions due to stairs on the supporting beams at either level.

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5 Masonry Buildings

5.1. Introduction

Most of the construction in the housing sector, in villages or towns, consists of load bearing brick walls. Brick work is strong in compression and with a planning, which ensures the same wall going from the foundation to the roof, supporting the intermediate floors, it ensures a speedy, economical and safe construction. This is the reason for its widespread use. But its design, in practice, is non-engineered, it is carried out by experienced masons or foremen by thumb-rules, which may lead to cracking and failures in some cases. The design of masonry structures being quite simple, is not being given adequate attention in the educational institutions also. Many young engineers are blissfully unaware of the provisions governing the masonry design. IS: 1905 and SP:20 are the two BIS (Bureau of Indian Standards) publications which control the design of masonry buildings.

Good bricks of crushing strength of 100 $Kg/cm^2$are available in northIndia and West Bengal, while bricks are poor in Rajasthan, Madhya Pradesh and Andhra Pradesh, where crushing strength of bricks is of the order of 35 to 50 Kg/cm^2. In east India, the brick size is 250 mm $\times$125 mm $\times$75 mm, inMadhya Pradesh (Bhopal area) it is 200 mm $\times$100 mm $\times$75 mm, while in the rest of the country, the brick size is 230 $\times$mm 115 $\times$mm 75 mm, which is the most common size. In Rajasthan and the hilly areas of UP and HP, random stone masonry is extensively used in building houses. The stone wall thickness generally varies from 300 mm to 450 mm, which consumes quite a lot of the

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floor area.

The strength of a brick wall is a function of the strength of the bricks (or stones) used and also of the strength of the mortar used. Earlier, clay mortar and also lime mortar were used. But now cement mortar is being used in all engineered buildings. The safe allowable stress in masonry will also depend on the slenderness of wall and also on the eccentricity of the applied loading. When a concentrated load from a beam resting on a brick wall is considered, a 45^0 to 60^0 dispersion of load is assumed. The actual pressures on walls on any section along with height of the wall shall be kept below the allowable masonry pressure as per IS: 1905. The local pressure on a brick wall may be weeded by 50%, this aspect is used to fix the size of a bed block given at themils of beams which rest on brick walls.

In masonry buildings, it is advisable to follow certain restrictions in architectural planning. The room sizes should not be large. The openings in walls shall be restricted. A wall shall be built over the wall, i.e. no wall shall rest on a slab panel. Otherwise, a beam has to be introduced to support the wall directly, directing the load to the cross walls at the ends of the beam. A brick wall does the functions of a beam and columns. The restrictions are not difficult to follow in the architectural planning of houses, flats, hostels, schools, etc. The number of storeys has to be restricted to three or four only. In seismically active areas, brick structures have to be provided with reinforced concrete ties at the plinth and the lintel levels and the full bearing of the concrete floor slabs has to be ensured at all the floor levels. The guidelines are given in IS: 4326.

Brick walls can be either load bearing walls or filler walls. Lintels in load bearing walls shall be carefully designed, taking into account the load from the floor slab also. Filler walls are provided in reinforced concrete framed buildings and these are required to support their own load only. Lintels in filler walls will work out lighter in reinforcement.

Mechanized bricks have the crushing strength of above 200 kg/cm2, these have been used for school buildings with three to four storeys, the slab panels being large for class rooms. Ordinary good bricks of 100 kg/cm2 value, would require walls of 345 mm in thickness, thereby reducing the usable floor area.

5.2. Brick Wall Design Under Vertical Loads

The design of a brick wall under vertical loads is best explained with a numerical example. In (Fig: 5.1 a), a part floor plan of a building is shown and it is desired to design the internal wall Wl. In (Fig: 5.1 b), a section of wall W1 is given through the height of the building, which is a three storeyed building.

(1) Vertical loadings on the roof and floors are given as follows :

Roof

Self weight of slab 0.15 m thick = 0.15 $\times$2500 = 375 Kg/m^2

Average 0.10 m thick lime terracing = .10 $\times$2000 = 200 Kg/m^2

Water proofing (say) = 50 Kg/m^2

Brick tiles on top = .05 $\times$2000 = 100 Kg/m^2

Ceiling plaster (say) = 25 Kg/m^2

DL = 750 Kg/m^2

Live load on an accessible roof LL = 150 Kg/m^2

TL = 900 Kg/m^2

Self weight of a slab, 15 m thick = 0.15 $\times$2500 = 375 Kg/m^2

floor finish 50 mm thick = .05 $\times$2500 = 125 Kg/m^2

Ceiling plaster = 25 Kg/m^2

DL = 575 Kg/m^2

Live load for residential use LL = 200 Kg/m^2

TL = 725 Kg/m^2

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(2) Load on wall W1 is calculated at the various levels 1-1, 2-2, 3-3 as follows. We consider 1.0 m length of wall in plan.

Level 1-1

$$\text{Load from roof slab} = \frac{wl_x}{4}\left(2 - \frac{l_x}{l_y}\right)$$
$$= 900\frac{4.0}{4}\left(2 - \frac{4.0}{6.0}\right) = 1200 \text{ kg/m}$$

$$\text{Load from roof slab} = 900\frac{5}{4}\left(2 - \frac{5}{6}\right) = 1313 \text{ kg/m}$$

Self weight of w = 0.23 ×3.0 ×1.0 ×1900 = 1311 kg/m

P_1 = 3824 kg/m

$$\text{Pressure in solid masonry wall p} = \frac{3824}{23 \times 100} = 1.66\ Kg/cm^2$$

Let us assume two door openings of 1.0 m each in the wall Wl. So, for length of wall less openings = 6.0 - 2 ×1.0 = 4.0 m
Pressure in masonry wall taking

$$\text{openings into account } \rho = \frac{3824}{23 \times 100} \times \frac{6.0}{4.0} = 2.49 \text{ Kg}/m^2$$

Level 2-2

Load at 1-1 level P_1 = 3824 kg/m

Load from slab S4 $\frac{725}{900}$ ×1200 967 kg/m

Load from slab S2 $\frac{725}{900}$ ×1313 1058 kg/m

Self weight of wall 1311 kg/m

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P_2 1311 kg/m

$\sum$ P $= P_1 + P_2$ 3824 + 3336 7160 kg/m

$\rho \dfrac{7160}{23 \times 100} \times \dfrac{6}{4}$ 4.67 Kg/cm^2

Level 3-3

Load upto 2-2 level $= P_1 + P_2 =$ 7160 kg/m

Load from the first floor slab S1 = 967 kg/m

Load from the first floor slab S2 = 1058 kg/m

Self weight 0.23 ×1.0 ×3.6 ×1900 = 1573 kg/m

$P_3 =$ 3598 kg/m

$\sum$ P = 7160 + 3598 = 10758 kg/m

$$\text{P} = \frac{10758}{23 \times 100} \times 64 = 7.02\ Kg/cm^2$$

(3) Allowable pressure on masonry wall

With the crushing strength of bricks to be 100 $Kg/cm^2$and with 1 : 6 cement mortar IS: 1905 gives,

p_a = 8.1 ×Cr Kg/cm^2

where, C_r = reduction factor due to slenderness of wall

$$\frac{h}{t} = \frac{0.75H}{t} = 75\frac{3.15 + 0.45 + 0.15}{0.23} = 12.2$$

Cr = 0.75

p_ϕ = 8.1 ×.75 = 6.08 Kg/cm^2

H = storey height

h = effective height of wall

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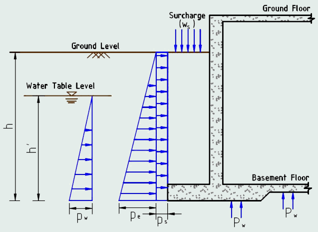

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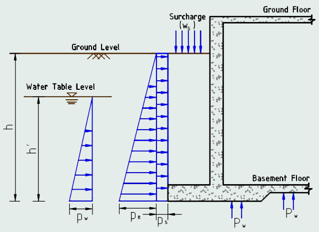

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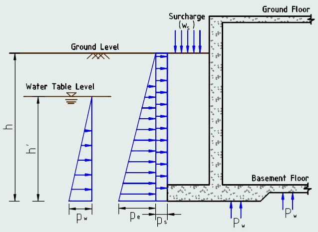

(c) b

Figure 5.1: Load bearing brick wall under vertical load, (a) Part plan of building; (b) Section X-X (c) Details of footing for brick wall W1

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I = thickness of wall

The actual pressure is 7.02 Kg/cm^2, while the allowable pressure is only 6.08 Kg/cm^2, which shows that 1 : 6 mortar is unsafe. 1:6 cement mortar is the leanest mortar used in practice. The next higher cement mortar is 1 : 4.5 (or 1 : 4), for which the allowable pressure on masonry,

p_a = 9.6 ×0.75 = 7.2 Kg/cm^2> 7.02 Kg/cm^2

So, we use 1 : 4.5 cement mortar in the ground storey.

Level 2.2

p = 4.67 Kg/m^2

For 1 : 6 cement mortar

p_a = 8.1 ×Cr

$$\frac{h}{a} = \frac{0.75 \times 3.15}{0.23} = 10.27$$

$C_r = 0.85$

p_a = 8.1 ×0.85 = 6.89 Kg/m^2> 4.67 Kg/m^2

So 1 : 6 cement mortar is safe. We use 1 : 6 cement mortar above the first floor to the roof, as the leanest mortar is 1 : 6

(4) Wall Footing

$$\text{The load at level 3-3} : \sum P = 10.758 kg/m$$
$$\approx 11.0 t/m$$

With a safe bearing capacity of soil at 10 t/m2 with a foundation depth of 1.2m below ground,

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$B = \dfrac{11.0}{10} = 1.1$m, required

We provide a footing width of 1.2 m, with brick steps as shown in Fig. 5.1c where care is taken to have a dispersion angle of 45^0 to 60^0 at all levels.

Pressure in brick work in foundation (on 345 mm thick wail) $= \dfrac{11000}{34.5 \times 100} = 3.19$ Kg/cm^2

1 : 6 cement mortar is more than adequate. The details of the wall footing are given in Fig. 5.1c.

In general, brick work in foundation, being of thick sections, is built in 1 : 6 cement mortar. In the walls above, cement mortar quality can be varied from storey to storey, to get an efficient design. In single or double storey buildings, brick quality may be varied, for general, we may use bricks of crushing strength of 75 kg/cm2 or less, thereby achieving some reduction in cost. The quality of bricks or the mix of cement mortar is kept the same for all walls in a given storey, so that the work can be easily checked and supervised at the site. In the cement sand mortars used in structural masonry, care shall be taken to use coarse sand, which is necessary to get the strength of the mortars presented in IS : 1905. Fine sand is used only in mortars used for plaster in of walls, which is taken as a finishing item.

115 mm thick walls are generally regarded as incapable of supporting load, other than their own weight only. These are built in 1:4 cement mortar with 2 nos 6 mm diameter bars at every fourth course. Further, isolated piers, window jambs are also required to be built in 1 : 4 cement mortar.

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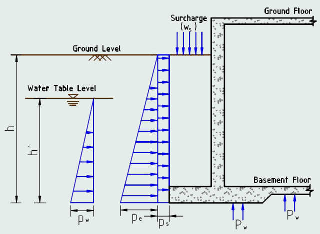
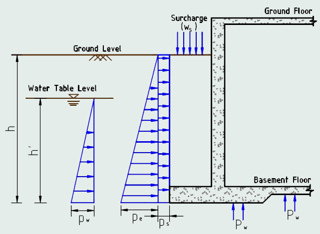


(a) Wind force on an external brick wall



(b) Wind pressure on filler brick walls

5.3. Brick Wall Design Under Horizontal Loads

All external brick walls in a building have to resist wind pressure. These walls span from one floor diaphragm to another, under the wind pressure and there is some tension produced at the mid-height of walls, which is partly resisted by the self-weight of the half-height of wall (Fig. 5.2). IS: 1905 permits tensile stress of 1.0 Kg/cm^2in masonry which may be increased by 33.3% when wind effect is included.
Referring to (Fig: 5.2a), p^w = 150 ×.75 = 112.5 Kg/m^2

h = 4.0 m, t = 23 cm

M_w = 112.5 ×4.0^{2}8 = 225 kgm

$$f_t = \pm \frac{M_w}{Z} = \pm\ 225 \times \frac{100}{100 \times 23\frac{2}{6}} = \pm \frac{22500}{8817} = \pm\ 2.55\ Kg/cm^2$$

p = load of wall at mid-height = 5.0 t/m (say)

$$f_d = \frac{5000}{23 \times 100} = 2.17\ Kg/cm^2$$

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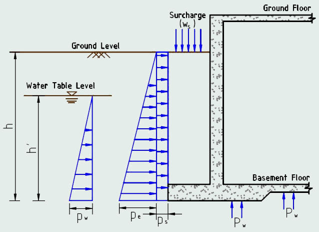

Figure 5.3: Wind pressure on boundary wall.

f = $f_d \pm f_1$ = 2.17 ± 2.55 = 4.72, -0.38 Kg/cm^2

f_{max} = 4.72 Kg/cm^2< pa, , depending on the quality of bricks and the mix of cement mortar used.

f_{min} = -0.38 Kg/cm^2(i.e. tensile stress is less than 1.33 Kg/cm^2). OK

When the external walls are used as filler walls in a framed building, the wall can be checked as follows (Fig: 5.2b).

M_w = 112.5 $\times 3.0^2/8$ = 127 kgm

$$f_1 = \pm \frac{127 \times 100}{8817} = \pm\ 1.44\ Kg/cm^2$$

ρ = load of self-weight of wall of h/2 height

= 0.23 ×1.5 ×1.0 ×1.9 = 0.66 t/m

$$f_d = \frac{600}{23 \times 100} = 0.29\ Kg/cm^2$$

f = 0.29 ± 1.44 = 1.73, -1.15 Kg/cm^2

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The tensile stress 1.15 Kg/cm^2less than 1.33 Kg/cm^2, so, O.K. So, 230 thick wall is safe under wind pressure. Further, there will be openings in walls which will relieve the wind force on the external walls.
A critical situation is created for cantilever boundary walls under wind pressure (Fig: 5.3).

$$M_w = p_w \frac{h^2}{2} = 112.5 \times \frac{2.0^2}{2} = 225 \text{ kgm}$$

$$f_1 = \pm \frac{M_w}{Z} = \pm \frac{127 \times 100}{8817} = \pm\ 2.55 Kg/cm^2$$

ρ = load of self-weight of wall of 2.0 m height

= .23 ×1.0 ×2.O ×1.9 = 0.874 t/m

$$f_d = \frac{874}{2300} = 0.38\ Kg/cm^2$$

f = f_1 + f_d = 0.38 ± 2.55 = +2.93, -2.17 Kg/cm^2

f_{max} = 2.93 Kg/cm^2(compression)

f_{min} = 2.17 Kg/cm^2(tension)

$$\frac{h}{t} = \frac{2 \times 2.15}{0.23} = 18.7$$

For bricks of crushing strength of 75 $Kg/cm^2$and with 1 : 6 cement mortar, IS: 1905 gives, p_a = 5.9 ×0.49 = 2.89 Kg/cm^2nearly equal to 2.93 Kg/cm^2, so safe in compression.

f_{min} = -2.17 Kg/cm^2, i.e tension is, more than the allowable value of 1.33 Kg/cm^2.

So, 230 mm thick boundary wall of 2.0 m height is unsafe under wind loads. Either height of wall may be reduced, say to 1.5 m, with an additional 0.5 m of open railing which may be provided at top or wall thickness may be increased to 345 mm.

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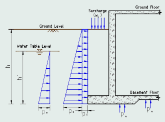

Figure 5.4: Plan of long boundary wall with pilasters.

$$\begin{aligned}
With, z &= 100 \times \frac{34.5^2}{6} && = 19,838 cm^3 \\
f_t &= \pm \frac{225 \times 100}{19838} kg/cm^2 && = \pm 1.13 \\
p &= 0.345 \times 1.0 \times 2.0 \times 1.9 && = 1.31 t/m \\
f_d &= \frac{1310}{345 \times 100} && = 0.38 kg/cm^2 \\
f &= 0.38 \pm 1.13 && = +1.51, -0.75 kg/cm^2
\end{aligned}$$

These values are within the allowable limits. Hence, 345 mm thick boundary wall is safe. In small plots, cross walls may hold the boundary wall at ends, thereby the wind pressure gets resisted in two directions. Further, pilasters at suitable spacing may be provided in long boundary walls for safety against wind pressure (Fig: 5.4).

It should be noted that free-standing brick walls of considerable height are dangerous to passersby, unless these are loaded and held at the top. Many fatal accidents, which get reported periodically in news papers, occur on this account.

5.4. Resistance To Earthquake Forces By Wall Boxes In Plan

Upto four storeys, brick walls may be relied upon to resist successfully earth quake or wind forces acting on the building with load bearing or filler walls. Fig. 5.6a, b gives a plan of typical floor of a four storeyed building and its long elevation, giving the storey heights. The principles involved are best explained by a numerical example, as given below.

- Wind force on long face of building (IS:875) $p_w = 150 \times 0.75 = 112.5\ Kg/cm^2$

  $$p_w = 10.0(12.45 + 0.9) \times \frac{112.5}{1000} = 15.0 \text{ t}$$

- Earthquake base shear(V_B)IS: 1893 gives,

  $V_B = \text{C}\alpha_h, \text{K.W}$

  $\alpha_h = \beta.l.\alpha_0$

  $= 1.5 \times 1.0 \times 0.05(\text{Delhi region}) = 0.075$

  K = 1.0

  $$\text{T} = \frac{0.09H}{\sqrt{d}} = \frac{0.09 \times 12.45}{\sqrt{5}} = 0.5$$

  C = 0.8 = taken as 1.0 for masonry buildings to be taken on the safe side.

  $V_B = 1.0 \times 0.075 \times 1.0 \times \text{W} = 0.075 \text{ W}$

  $$\text{W} = W_r + \sum_{n=1}^{n=3} W_f$$

  $W_r = D_L$ on roof + weight of half height of walls + W_t of parapet wall.

  $= 0.75 \times 10.0 \times + 0.23\ (0.9 + 1.5)\ (20 + 20) \times 1.9$

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$= 37.5 + 42.0 = 79.5$ t

W_f = (D_L + .25LL) on floor + weight of storey height of walls.

$= (0.525 + 0.25 \times 0.2)10 \times 5 + 0.23 \times 3.3 \times 40 \times 1.9$

$= 28.8 + 52.4 = 81.2$t

W $= 79.2 + 3 \times 81.2 = 322.8$t

$V_B = 0.075 \times 322.8 = 24.2\text{t} > \text{Pw} = 15.0\text{t}$

- Distribution of Earthquake Shear (V_b) along the Height of Building(Clause 4.2.1.2 of IS: 1893-1975)

$$Q_i = V_B \frac{W_1 h_1^2}{\sum(w_1 h_1^2)}$$

Level	W_1(t)	h_1(m)	$W_i h_i 2$	$\frac{W_i * h_i^2}{\sum W_i h_i^2}$	Q_i(t)
Roof	79.8	13.95	15529	0.48	11.6
3rd floor	81.2	10.95	9736	0.30	7.3
2nd floor	81.2	7.95	5132	0.16	3.9
1st floor	81.2	4.95	1990	0.06	1.4
Best of footing		0	0	-	-
sum	323.1	-	32387	1.00	24.2

Table 5.1: Table 5.1 gives the details of the calculations and values of Q_i acting at the various floor levels.

Base shear = 24.2 t

Overturning moment (OTM) about the base line

$= 11.6 \times 13.95 + 7.3 \times 10.95 + 3.9 \times 7.95 + 1.4 \times 4.95$

$= 161.82 + 79.94 + 31.005 + 6.93 = 279.7 \approx 280$ tm

Wall area in plan $= 0.23(20 + 20) = 9.2\ m^2$

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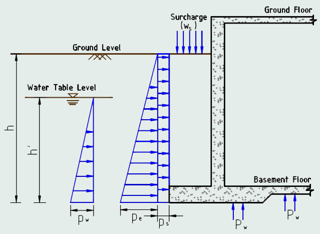
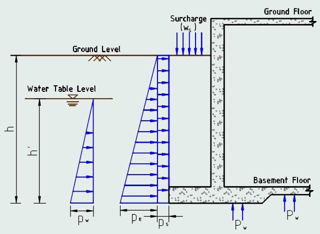


(a) a



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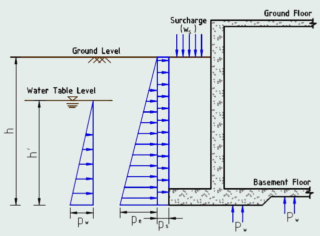

(c) b

Figure 5.5: Typical floor plan of a 4-storeyed building, (a) Plan of a typibalfloor; (b) Elevation of.

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Shear stress in walls $= \dfrac{24.2}{9.2} = 2.63\ t/m^2$

$$= 0.263\ Kg/cm^2$$

The allowable shear stress in walls $= 1.5\ Kg/cm^2 \times 1.33 = 2.0\ Kg/cm^2$(for earthquake)

Stresses due to OTM $f_1 = \pm\dfrac{M}{2}$

Considering the outer box together with the cross walls which along with floor diaphragms keep the box in shape,

$$z = \left(10.0 \times \frac{5.0^2}{6} - 9.54 \times \frac{4.54^2}{6}\right) + 2 \times 0.23 \times \frac{5.0^2}{6}$$
$$= (41.67 - 32.77) + 1.9$$
$$= 8.9 + 1.9 = 10.8 m^3$$
$$f_t = \pm\frac{280}{10.8} = \pm 25.9 t/m^2 = \pm 2.59 Kg/cm^2$$

If these walls are only filler walls, i.e. for vertical loading, we have a separate flexible reinforced concrete beam-columns system, the vertical compression in walls,

$$f_d = \frac{0.23 \times 4.95 \times 1.0 \times 1.9 \times 1000}{23 \times 100} = 0.94 Kg/cm^2$$
$$f = f_d + f_t = 3.51 \pm 2.59 \qquad = +6.1, +0.92 kg/m^2$$

i.e. there is no tension in brick walls. The walls are, thus strong enough to resist earthquake forces by its box like shape in plan.

This is the way, we can explain, how the ordinary houses in brick resistearthquake shocks.

In the recent earthquake (1993) Latur, Maharashtra, it has been explained that as houses there were made in mud-mortar, these collapsed without warning on the residents, who were sleeping during the fateful night. It is also reported that a

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couple of houses which were built recentlv in cement mortar stood the earthquake shocks well and saved the lives of the residents.

It cannot be over-emphasized that masonry buildings should be engineered structures with bricks and mortar of the required strength and with adequately designed foundations.

6 Framed Buildings Under Vertical Load

6.1. Introduction

Vertical load is the actual loading acting on a building. Of this, the dead load acts on the building components for all time, while the live load portion mayact from zero to 100% of its value on the affected building members and further, it may shift its position too. In slabs, live load may be 20% to 40% of the total loading, while in beams and columns, this ratio comes down to 10% to20%. It is, thus, important to note that dead load is the most significant of allloads. Dead and live loads are both gravity or vertical loads and these producesimilar effects in members and so these are added together to give total load (TL) or vertical load (VL).

The structural arrangement of a building shall be so chosen as to make it efficient in resisting the vertcal load. Vertical loads first act on floors (including roof), which consist of a slab panels and beams. Slabs and beams bend between the vertical supporting elements like walls and columns, transferring the load to these vertical elements. Walls and columns transfer the load to the ground (or soil) by means of footings at their bases, so that the ground pressures and settlements are not exceeded beyond their permissible values.

The efficiency of a structural system is to be judged from the fact that how best it resists the vertical load. We have two systems, which resist the vertical load well: (i) load bearing brick walls; (ii) reinforced concrete frames.

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When the spans are large or the number of storeys is large (more than four), reinforced concrete frames cannot be avoided. In practice, there come many instances, where it is difficult to decide which system is more appropriate than the other. In cinema or auditorium buildings, although being single storey, framed system is used, as the spans and the storey heights are large. In electric sub-station buildings, the storey height is kept at 5.0 m to 6.0 m and the frames system is preferred. In temple halls or halls of general gatherings, teinforced concrete frames (i.e. beams and columns in reinforced concrete)are used.

The structural system consists of an efficient floor system (Chapter 3), together with the location of columns. The column spacing or location is to be decided by architects, taking into account the function of building. In practice, the spacing of frames varies from 4.0 m to 7.0 m, the latter value is often used in hospital buildings. 7.0 m $\times$7.0 m is supposed to be an ideal size for an operation theatre. Special requirements may call for even larger frame spacings.

Close frame spacing gives an economical design, but the function of building may not permit it. Subsidiary or non-grid beams are provided in floors to get efficient slab systems of one-way or two-way panels, with slab thickness being kept at 10 cm to 15 cm. More slab thickness leads to uneconomic structure as a whole, as the slab loading affects all other members like beams, columns and footings.

Frames are normally provided in both the principal directions. In certain situations, for resisting vertical loads, frames in one direction may be adequate. But frames have to be provided in the other direction also, in order to resist horizontal loads, which aspect will be discussed in detail in Chapter 7. The author has come across some existing buildings with frames only in one direction, indicating that this aspect of having frames in both the principal directions is not appreciated by some practising engineers.

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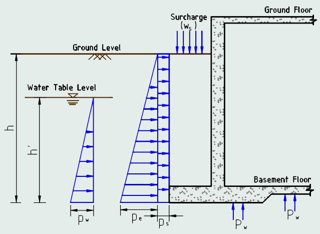
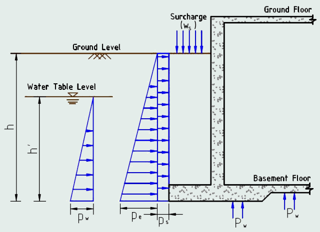


(a) a



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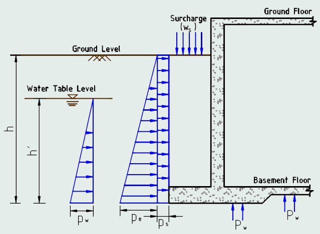

(c) b

Figure 6.1: Slab load carried by supporting beams, (a) Load distribution asgiven by the code; (b) load on beam B1; (c) Load on beam B2.

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6.2. Frame Analysis Under Vertical Loads

A given building may be divided into frames in either principal directions. Beam loadings at all floor and roof levels are calculated and the beam and column sizes are assumed at the outset. Each frame is then analysed by a computer program like STAAD III, which is easily available. In the frame analysis, centre to centre distance between members shall be used as per clause 21.2 of the Code. The program is based on a stiffness matrix approach and it gives values of beam and column moments, shears and axial loads. This appears to be an exact approach, but it does not consider following aspects:

- Beam loadings are calculated on the basis of the equivalent uniformly distributed load (W_b),based on equal moment at the centre of span, with the triangular or trapezoidal loadings as given by clause 23.5 of theCode. Referring to (Fig: 6.1),

$$\text{For beam B1} : W_b = W\frac{lx}{3} \tag{6.1}$$

$$\text{For beam B2} : W_b = W\frac{lx}{6}\left[3-\left(\frac{lx}{ly}\right)^2\right] \tag{6.2}$$

where W =slab loading in t/m^2For beam B1 : $W_s = \quad W\frac{lx}{4}$

$$\text{For beam B2} : W_s = W\frac{lx}{4}\left(2-\frac{lx}{ly}\right) \tag{6.3}$$

By using these values of W_b,we get beam shears, which are more thanthe actual values, thereby, the column loads work out on the high side.
For an equal shear at beam supports, the equivalent uniformly distributed load (W_s) is given by (Fig: 6.1),
With these values ofWs,beam shears and thereby the column loads, work out exact. Some computer programs like STAAD III have provisions for triangular and trapezoidal distribution of load on beams, so that beam moments and shears and

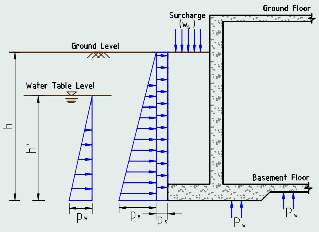

Figure 6.2: Substitute frame method for a floor beam.

also column loads all will work out exactly.

- In the analysis, moment of inertia of beams shall be calculated taking the effect of F-action of slab into account. Its necessity has been nicely explained by Jain and Jaikrishna (p.2, Vol. II). In many computer programs,only rectangular section of beam is considered and the F-section of flange provided by slab is not considered. This leads to more moments in columns than the actual column moments, leading to an expensive column design. These programs do solicit data of flange width and thickness but these are not used for the analysis. These data are rather used only for design of beam section at the midspan.

- Live load variation on different spans cannot be considered by using computer programs at one go. Arrangement of live loads on beams has been given in clause 21.4.1 of the Code, in which it is also stated that live load variation may not be considered, if the live load is less than three-fourth of the dead load, which is normally the case in buildings. So, this aspect may not be considered, where the live loads are within such a range.

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6.3. Approximate Analysis By Substitute Frame Method

The most common way of analysing a floor beam is to consider the beam with its columns regarded as fixed at the bottom and the top floor levels (Fig. 6.2). This is called the substitue frame method in clause 21.4.2 of the Code. For roof beams, the upper columns will be non-existent. The live load variations on different spans of a continuous beam, the effect of T-action of the flange slab on the moment of inertia of beam, can be easily considered by a specially devised brief procedure called the two-cycle moment distribution method.33 It used to be the most popular method followed in design offices during the sixties. Presently, an efficient computer program is available to handle the substitute frame with the UDL (uniformly distributed load) on a span, which is to be given in ten parts and concentrated loads at centre or anywhere else on the pan, can also be accommodated. The program can also consider support moments in the beam due to horizontal loads and it gives both the analysis and the design of the beam. In the analysis results, we get support moments andshears in beams and in the design results, we get steel areas at the supports and the midspan sections of spans with $\frac{V_{us}}{d}$-values for fixing stirrups inbeams. The program can accommodate a maximum of eight spans. This program is widely used in design offices. But as it is, it does not consider the effect of live load variations and .disregards the effect of T-action on the moment of inertia of beams. Further in design of beams, support moment at column face shall be considered. But the program considers the centre-line support moment. It is very much on the conservative side. In the two-cycle moment distribution method, a face correction of Va/3 is applied on the support moment, where V = direct shear at support, a = width of support.
For shear, the critical section as per the Code is effective depth awayfrom the column face and this aspect has been considered in the computerprogram in use.

6.4. Interaction At Junction Of Reinforced Concrete Elements

In the earlier practice, continuous beams monolithic with columns were permitted to be designed as continuous over supports and capable of free rotation. This may be near to reality, if the columns are small.in size, say, 230mm $\times$230 mm and the beam size is large, say, 230mm $\times$600mm. When the columns are large in size, this approximate method leads to errors. This method is, therefore, not much in use now. But this aspect raises the question of interaction of reinforced concrete elements at their junctions, as to what end conditions should be assumed for various members in a monolithic reinforced conrete building.

(Fig: 6.3 a) gives beam-slab junctions, in which the slab may be analysed as continuous over supports, capable of free rotation. This assumption relieves the beam of any possible torsion and it results in a conservative slab design. However, at end supports, the negative moment of $\dfrac{wl^2}{24}$ should be considered as per clause 21.5.2 of the Code. This is normally a small value and it is taken care of, by the top steel provided at the end support by way of good detailing.

(Fig: 6.3 b) gives a junction of a beam Bl, resting on the supporting beam B2 at its ends. The beam Bl is regarded as a simply supported one. In detailing, care should be taken to provide for a nominal negative moment at ends. This way, the suporting beams are relieved of torsion and it makes for a conservative design of beam Bl.

(Fig: 6.3 c) gives a beam-column junction. This junction should be regarded as a rigid connection, so that frame can also resist horizontal loads. If the junction is made capable of free rotation (i.e. simply supported), it can resist only vertical loads, but it is of no use for horizontal loads. This assumption of simply supported junctions is generally used in steel buildings, where the detailing of steel connections becomes easy. But in the reinforced concrete cast-in-situ (or monolithic) construction, it is not realistic to assume the beam-column joint as capable of free-rotation and it is positively harmful

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under horizontal loads. This beam-column interaction may be called as primary interaction, required for the safety of building, while the interactions given in (Fig: 6.3 a), b are subsidiary interactions.

(Fig: 6.3 d) gives the column-footing junction. The columns in buildings are generally regarded as fixed at base. If a hinged base is required, a proper hinge with reduction in column size and crossing of column bars is to be provided. For achieving a realistic fixed base in practice, footings should be given adequate thickness and column bars should be fully anchored into the footing.

6.5. Exact Column Loads And Moments

From the full frame analysis (Section 6.2) or the substitute frame analysis at floor and roof levels (Section 6.3), column loads can be found by the summation of beam reactions at each floor level. By-this way, we shall get column loads on the basis of total loading, i.e. full dead and live load at all the floor levels. But in multi-storeyed buildings, it is important to apply live load reductions as given by IS:875. Live loads at each floor level for each column can be found by the tributary area method and dead loads at each floor level for each column can be found by using the relation, DL = TL - LL. The design column loads can be found at each floor level by following a tabular form as explained below, with a numerical example.

Referring to a column shown in Fig. 6.4, we have its tributary area (TA) = 6.0 m $\times 6.0$ m = 36.0 m^2. The following data are given from the frame analysis.

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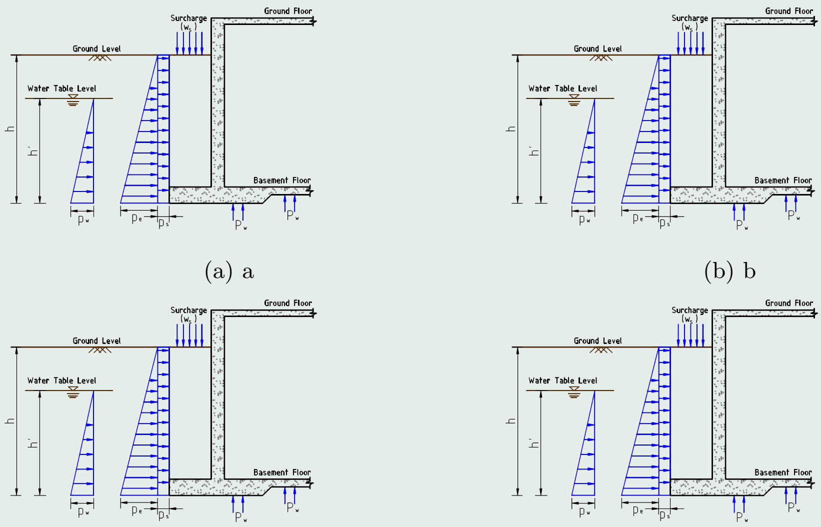

(a) a (b) b

(c) b (d) b

Figure 6.3: Interaction of RC elements at junctions, (a) Slab and beam junctions, (b) Plan of beam to beam junction, (c) Column beam junction, (d) Column footing junctions.

$$\text{Root:} TL = 40.0t, LL = 36 \times 0.15 = 5.4t, DL = 40.0 - 5.4 = 34.6t$$
$$\text{Typ. loor} : TL = 52.91, LL = 36.4 = 14.4t, DL = 52.9 - 14.4$$
$$= 38.5t \,(\text{1st to 5th})$$

The design column loads are given as follows :

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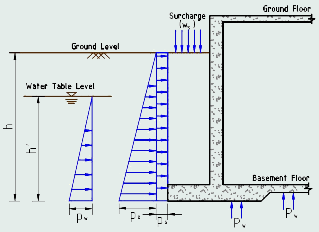

Figure 6.4: Elevation of an example column.

$Level$	$DL + K \times LL$	$= P(t)$
$Roof$	$36.4 + 1.0 \times 5.4$	$= 40$
5th fl.	$73.1 + 0.9 \times 19.8$	$= 91$
4th fl.	$111.6 + 0.8 \times 34.2$	$= 139$
3rd fl.	$150.1 + 0.7 \times 48.6$	$= 184$
2nd fl.	$188.6 + 0.6 \times 63.0$	$= 226$
1st fl.	$227.1 + 0.6 \times 77.4$	$= 274$
GF	274	$= 274t$

For column design at various floor levels, column load is given by P(t) in the above table, which is to be combined with column moments in both the principal directions got from the frame analysis, to be reduced at face (Fig: 6.5). In general, a column is subjected to an axial compressive load P(t) and moments in the two principal directions. Now, these moments shall not be less than the moments due to minimum eccentricity moments. Clause 24.4 of the Code gives,

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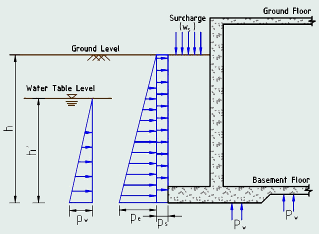

Figure 6.5: Face correction for beam ana column moments.

$$e_{min} = \frac{l}{500} + \frac{(bor D)}{30} (\not< 2.0cm) \tag{6.4}$$

where, l = clear height of colum under a floor beam in cm
b = short side of column in cm
D = long side of column in cm

Minimum eccentricity moments have been prescribed by the Code to account for inaccuracies at the site in the plumb line of a column. ACI Code and its commentary explain that the minimum eccentricity moments shall betaken about one axis at a time and these should not be applied about both theaxes simultaneously.
Further, slenderness effects are important for column design. When l_{ef}/b > 12, a column is regarded as slender and it should be designed with additional moment due to slenderness given by,

$$M_{a,b} = \frac{KPb}{2000} \left(\frac{l_{ef}}{b}\right)^2 \tag{6.5}$$

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$$
\begin{aligned}
where l_{ef} &= \text{effective height of column } (=1.2l) \\
l &= \text{clear height of column in a given storey} \\
P &= \text{column load} \\
k &= \frac{P_{uz} - P_u}{P_{uz}} - P_b \, (\leq 1) \\
&= 0.5 \text{ for heavily loaded column} \, (P_v / f_{ck} bD > 0.4) \\
&= 1.0 \text{ for heavily loaded column} \, (P_v / f_{ck} bD \leq 0.4) \\
P_{uz} &= 0.45 f_{ck} A_c + 0.75 f_y A_{sc} \\
P_{uz} &= \text{ultiamte column load} \, (= 1.5P) \\
P_b &= \text{axial load corresponding to condition of maximum compressives train} \\
&\quad \text{of 0.0035 in concrete and tensile strain of 0.002 in outermost layer, of} \\
&\quad \text{tension steel.} \\
f_{ck} &= \text{characteristic compresive strength of concrete.} \\
f_y &= \text{characteristic strength of steel.} \\
A_c &= \text{net concrete area of column section.} \\
A_{sc} &= \text{area of steel in column section.} \\
&\quad When \; l_{ef}/b \leq 12, \text{slenderness effect is to be neglected. However, when} \\
&\quad l_{ef}/b = 12, Eq : 6.6 gives
\end{aligned}
$$

$$
M_{a,b} = \frac{kPb}{2000} \times (12)^2 = 0.072k.P.b \tag{6.6}
$$

This value of the additional moment due to slenderness with l_{ef}/b= 12 is alsosubstantial, but it is allowed by the Code to be neglected: For k= 0.5, P = 274t, b = 0.3 m, Eq:6.6 gives

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$$
\begin{aligned}
M_{a,b} &= 0.072 \times 0.5 \times 274 \times 0.3 = 2.96tm \\
& \text{when } l_{ef}/b = 6, (Eq: 6.6) gives, \\
M_{a,b} &= kPb(6)^2/2000 = 0.018kPb && \textbf{(6.7)} \\
& Fork = 0.5, P = 274t, b = 0.3m && \textbf{(6.8)} \\
M_{a,b} &= 0.018 \times 0.5 \times 0.3 = 0.74tm
\end{aligned}
$$

which is small enough to be neglected. Strictly speaking.
$M_{a,b} = 0$, when l_{ef}/b = 0
The opposite member of a column can be a hanger or a suspender. The axial load is tensile in a hanger and it should be combined with the frame moments or the minimum eccentricity moments whichever is greater. Slenderness effect is not relevant for hangers, as buckling is a phenomenon relevant for columns only. However, hangers are susceptible to vibrations or flutter and it is recommended by Bulme et al., that hanger load should be increased by 33.33% to take care of the "flutter" phenomenon. Further, tensile stress in concrete isto be restricted in tension members as per clause 44.1.1 of the Code.

6.6. Approximate Methods For Column Loads And Moments

When work is to be done in a hurry and the foundation is to be designed before the superstructure, then column loads can be calculated by the tributary area (T.A.) method. On each floor, dead and live loads are calculated for a column and live load reduction factors are applied to get the design load ateach floor, which may be increased by 5% to account for omission of any unforeseen items. In this method, the effect of elastic shear on beam shears is notconsidered. The increase of 5% is done to compensate for this omission too.

A numerical example is given below to illustrate the procedure for finding the column loads by the tributary area method. For an internal column of 450 mm ×450 mm with a T.A. = 6.0 ×6.0 = 36.0 m2 (Fig. 6.4), the floorloads are calculated as follows:

$$\text{Roof slab loading :DL} = 750kg/m^2$$
$$LL = 150kg/m^2$$
$$\text{Typical floor slab loading :DL} = 625kg/m^2$$
$$LL = 400kg/m^2$$
$$\text{Roof: DL : slab}36 \times 0.75 = 27.0(t)LL : 36.0 \times 0.15 = 5.4t(0.23 \times 0.45 \times 1.0 \times 2.5)$$
$$= 0.26\text{t/m beam}0.26(6+6) = 3.1t$$
$$parapet = -$$
$$(0.45 \times .45 \times 3.6 \times 2.5) \text{ Self wt. of col.} = 1.8t$$
$$DL = 31.9t\ LL = 5.4t$$
$$\text{Typical floor : DL slab } 36 \times 0.625 = 22.5t\ LL : 360.4 = 14.4t$$
$$\text{beams} = 3.1t$$
$$(.115 \times 1.0 \times 3.0 \times 1.9) \text{ 115tk. walls } .65(6+6) = 7.8t$$
$$\text{Self wt. of column} = 1.81$$
$$DL = 35.2tLL = 14.4t$$

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$$\text{Level DL} + \text{k.LL} \ = TL \times 1.05 \quad = P(t)$$

$$\text{Roof } 31.9 + 1.0 \ \times 5.4 = 37.3 \times 1.05 \quad = 39 \tag{6.9}$$

$$\text{5th fl. } 67.1 + 0.9 \ \times 19.8 = 84.9 \times 1.05 \quad = 89 \tag{6.10}$$

$$\text{4th fl. } 102.3 + 0.8 \times 34.2 = 129.7 \times 1.05 = 136 \tag{6.11}$$

$$\text{3rd fl. } 137.5 + 0.7 \ \times 48.6 = 171.5 \times 1.05 = 180 \tag{6.12}$$

$$\text{2nd fl. } 172.7 + 0.6 \times 63.0 = 210.5 \times 1.05 = 221 \tag{6.13}$$

$$\text{1st fl. } 207.9 + 0.6 \times 77.4 = 254.3 \times 1.05 = 267 \tag{6.14}$$

$$\text{GF} = 267t \tag{6.15}$$

Column moments at each floor level can be calculated by using TableIX of IS: 456-1964,3 which is based on the method of slope-deflection, applied on substitute frames. The values obtained from Table IX are reasonably accurate and these moments or the minimum eccentricity moments (which ever are greater) are used in column design.

6.7. An Example Buildings

A typical framing plan of the example building of six storeys is given in (Fig: 6.6). The grid is 6.0 m ×6.0 m. All columns are 450 mm ×450 mm in size and all beams have the size 230 mm ×600 mm, with 150 mm as slab thickness. 230 mm thick brick walls are considered on the peripheral beams, while 115mm thick walls are considered on all internal grid beams. The floor slab panels are to be designed for a light partition wall load of 100 Kg/m^2.

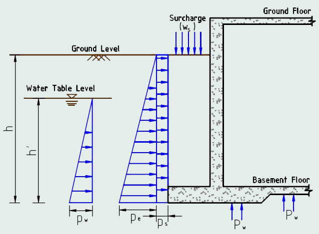

Figure 6.6: Plan of the example building of six storeys.

Roof slab :	Self weight 0.15 ×2500	=	375 Kg/m^2
	Lime terracing 0.10 ×2000	=	200 Kg/m^2
	Water proofing	=	50 Kg/m^2
	Ceiling plaster	=	25 Kg/m^2
	Brick linking	=	750 Kg/m^2
	DL	=	750 Kg/m^2
	(Accessible roof) LL	=	150
	TL	=	900 Kg/m^2
Typical floor slab:	Self weight 0.15 ×2500	=	375 Kg/m^2
	Floor finish 0.05 ×2500	=	125 Kg/m^2
	Plaster	=	25 Kg/m^2
	Light partitions	=	100 Kg/m^2
	DL	=	625 Kg/m^2
	LL	=	400 Kg/m^2
	TL	=	1025 Kg/m^2

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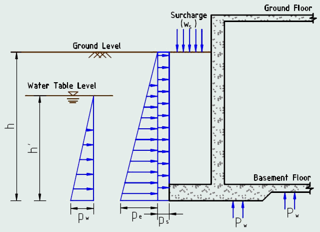
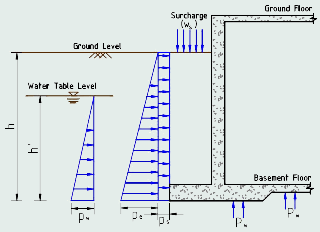
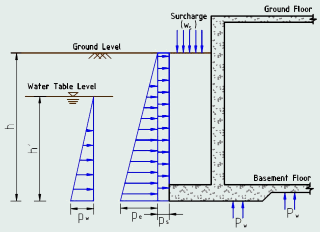


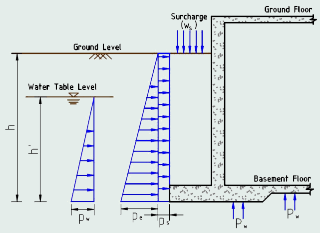
(a) Internal transverse fiames on lines 2, 3, 4, 5 (B2).



(b) End transverse frames on lines 1,6 (B1).



(c) Longitudinal frames on lines B and C(B4)



(d) End longitudinal frames on lines A and D (B3)

Figure 6.7: abc

We wish to design this building by the following four methods and compare the column loads and moments in the base storey. Method I (Sections 6.2 and 6.5). The building is divided into full frames in the two principal directions. FI and F2 are the two transverse frames and F3 and F4 are the two longitudinal frames (Fig: 6.7). The loads on beams are calculated as follows:

- Frames F1(on lines 2,3,4,5)
  Also frame F3 (on lines B,C)

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Typical floor:	W1 = Selfweight of rib-beam 0.23 ×0.45 ×1.0 ×2.5	=	0.26 t/m
	115mm thick brick wall 0.115 ×3.0 ×1.0 ×1.9	=	0.66 t/m
	W_1	=	0.92 t/m
		≈	1.0 t/m
	W2 = Triangular load ordinate	=	1.025 ×6.0/2 ×2
		=	6.2 t/m
Roof:	W1 = Self wt. of beam	=	0.26 or 0.30 t/m
	W2 = 0.9 ×6.0/2 ×2	=	5.4 t/m

- Frames F2(on lines 1,6) Frames F4(on lines A,D)

Floor:	W1 = Self wt. of beam	=	0.26 t/m
	230 thick wall 0.66 ×2	=	1.32 t/m
	W1	=	1.58 t/m = 1.6 t/m
	W2 = 1.025 ×3.0	=	3.1 t/m
Roof:	W1 = Self wt. of beam	=	0.26 t/m
	parapet 0.23 ×1.0 ×1.0 ×1.9	=	0.44 t/m
		=	0.70 t/m
	W2 = 0.9 ×3.0	=	2.7 t/m

Computer analysis of the four frames FI to F4 is done and column loads and moments for columns Cl to C6 are found.Also moments and shears in floor and roof beams are found. The computer program used considers only the rectangular moments of inertia of beams at all levels.

Method II (Sections 6.3 and 6.5). The building is divided into substitute frames at

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the roof and floor levels. Beams B1 to B4 (Fig. 5.5) are analysed by computer at the roof, typical and the first floor levels separately under the loading W_1, W_2 found in Method I. The column loads for colums Cl to C6 have been found from the beam reactions at each level and the live load reductions at each level. The live load reductions have been applied by the tributary area method. Column moments are given by the beam analysis at each floor level. The computer program used considers only the rectangular moment of inertia of beams.

Method III(Section6.6). The column loads are found by (he T.A. method and the column moments are calculated by using Table IX ol'IS:456-1964,3 considering the effect of slab flange on the moment of inertia of beams.

Method IV. In this method, we follow Method II by solving beams by the moment distribution method, taking the T- or L-effect on the moment of inertia of beams at all levels. In a way, this method corrects the pitfalls of the MethodII. The column loads and moments at base are found.

Column	Column	Load	At	Base(f)
Mark	Method 1	Method 2	Method 3	Method 4
C1	107	111	115	111
C2, C3	177	178	180	180
C4	284	275	270	280
C5	177	174	180	174
C6	284	268	270	271

Table 6.2: Comparison of column loads at base by different methods of analysis.

Tables (6.2) and (6.3) give the values of column loads at has and column moments in the base storey by the four methods considered above. Column loads for the column Cl are given 107, 111, 115, 111 by.the four methods.This is a comer column. The T.A. method (Method III) gives a conservative result by 4%. Method I gives the least load of 107 tonnes, which.is 4% less than lilt.For columns C2, C3, the values are nearly the

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Column	Column	moments in	base storey	(tm) mark
	Method 1	Method 2	Method 3	Method 4
C1	3.1/4.8	3.8/3.8	2.7/2.7	2.8/2.8
C2	3.6	5.2	3.2	3.4
C3	4.3	5.25	3.3	3.4
C4	0.2	0.65	≈0	0.6
C5	4.3	5.25	3.3	3.4
C6	0.2	0.65	≈0	0.6

Table 6.3: Comparison of column moments in the base storey by different methods of analysis.

same by all theme methods. For the penultimate columns C4, the values are 284, 275, 270 and 280 t. The T.A. method gives the least value of 2701, which does not take into consideration the effect of elastic shears of beams. Method IV gives 280t which is a realistic value. The internal column C6 has load 284, 268, 270 and2 71 t, which are expected to be less than the load of column C4. Likewise, load on C5 should be less than that on C3. It is seen that Method I of full frame analysis surprisingly does not give reasonable results, the column loads being on the high side and elastic shear effect is not at all visible. Methods IIand IV are nearly the same, the difference will be visible in column moments.Method III of T.A. method compares well with the methods II and IV and its value for column C4 falls short by 4%, due to the elastic shear effect.

Table (6.3) compares the column moment in the base storey for columnsCl to C6. In internal columns C4, C6 moments are negligibly small and the minimum eccentricity moments will govern the design. For comer columnsCl and external facade columns C2, C3, C5, Methods III and IVgive realistic values of column moments as both those methods consider the effect of slab on the moment of inertia of beams. Methods I and II expectedly give higher values of column moments, as only the rectangular section has been considered for the moment of inertia of beams and the effect of the floor slab has been altogether neglected.

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Computer programs used in Methods I and II need to be updated to include the effect of slab on the moment of inertia of beams in frame analysis.

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7 Framed Buildings Under Horizontal Loads

7.1. Introduction

Framed Buildings underHorizontal Loads7.1. The structural design of buildings under vertical loads has to be further checked for horizontal loads, which, in general, act on buildings for a short while. Wind is acting on buildings all the time but its worst effect, say, during a storm acts for a short while. Earthquake phenomenon is also short-lived and may occur once or twice or a few times, during the life-time of a building, which is generally taken to be one hundred years. The worst effect of the temperature variation and shrinkage also occurs once in a while. For this reason, working stresses of materials are increased by 33.33% $(1 + 1/3 = 4/3)$ in the working stress method of design and the load factors are reduced by 25% $(1 - 1/4 = 3/4)$ in the limit state method of design. The structural system chosen for its efficiency in resisting vertical load is to be checked for the acting horizontal loads, which mainly depend on the locality or area in which the building is situated. The higher the building, the more prominent is the effect of the horizontal loads on the structural design. But, all buildings, whatever be the number of storeys, shall be checked for the horizontal loads. There are some engineers who do not consider the effect of wind or earthquake on one or two storeyed buildings. No code gives this sanction. When the ground moves during an earthquake, all the buildings, whatever the height, are affected. An earthquake has no way to know the number of storeys of buildings.1 So thestrict practice should be to check all the buildings for the effect of horizontal loads. It may be, that, for short buildings, one may use approximate and quick methods of analysis, while for tall buildings, strict methods of analysis will

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have to be used.

Horizontal loads act at floor levels. Wind, for example, acts on external walls (or cladding), which span between floor diaphragms and wind load gets transferred to floor levels. The earthquake shear being proportional to the dead load, acts at the floor levels, where most of the dead and the live loads act. Temperature effect also causes expansion or contraction of floor diaphragm, so it also acts at the floor level.

The horizontal load acting at floor level forces the floor diaphragm to move or translate in the direction of the force, thereby the vertical elements like columns, attached to the floor come under bending (Fig: 7.1). The beams being monolithic with columns at the joints also undergo bending. Thus it can be said that the frames resist the horizontal shear. The floor diaphragm, being infinitely stiff in its own plane, distributes the horizontal shear to the various frames, so that, at a given floor level, the horizontal movement or translation is the same for all frames, which is the same as the translation of the floor diaphragm. So the horizontal shear gets distributed to the frames in proportion to the frame stiffness, which is the inverse of the frame deflection. Now the frames, in general, may not be equal. How much of the total horizontal shear or load goes to a given frame is the problem to be solved at the first instance.It may be called allocation analysis.Then, when the frame shear is known, the frame has to be analysed for the known horizontal shears at floor levels, which is called frame analysis. The moments, shears and axial loads in beams and columns given by the frame analysis will be utilized in the design of members.

An interesting complication arises when the centre of stiffness of frames is not coincident with the centre of the applied horizontal load. Then the floor diaphragm not only translates but also rotates about the centre of stiffness (or centre of rigidity), by which the horizontal shear in some frames gets increased, while it is decreased in other frames (Fig: 7.2).

In many texts, only frame analysis is given, while the allocation analysis is not given the attention or importance that it deserves. Earthquake or wind is erratic in direction

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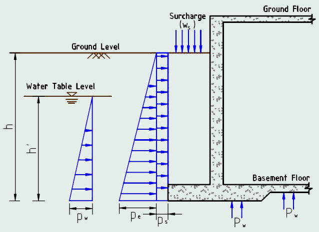

Figure 7.1: Floor diaphragm under translation

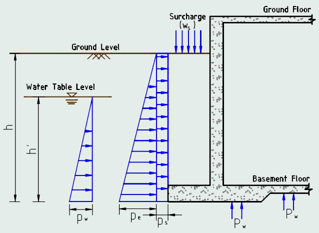

Figure 7.2: Plan of floor diaphragm under translation and rotation. P = horizontal shear due to wind; $T = P_e$(anti-clockwise).

and magnitude. The direction of wind or earthquake can be any direction in space. Its horizontal component is the major action affecting the building design. Its vertical component, being in the direction of gravity or vertical loads, is not important in building design. Further reversal of direction of horizontal component of wind or earthquake is important to be considered. Thus earthquake or wind is applied on a building with its full value in each principal direction separately, in order to take care of its erratic nature.

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It is seen that horizontal loads add to the cost of buildings. Beams and columns require more reinforcement on this account. Slabs and footings remain, in general, unaffected. In order to keep the cost of buidlings within reasonable limits, it is assumed that the worst effects of wind, earthquake and temperature variation do not take place at the same time, during the life time of building.
In order to keep, within reasonable limits, the extra cost of including horizontal loads in structural design, the structures can only be made earthquake-resistant and not earthquake-proof which is not a practical proposition cost-wise. A "premium-free" design is that, in which no extra steel is required to be put for the effect of horizontal loads, in the structural members already designed for vertical loads. But it is rarely achieved in practice for tall buildings.

7.2. Allocation Analysis

An approximate method for the distribution of shear due to horizontal loads in reinforced concrete framed multi-storeyed buildings has been given by Assudani and Varyani and it is given below. The practice of designing bents(i.e. frames) in a framed building for horizontal loads due to wind or earthquake, which are tributary to them (on the analogy of vertical loads on bents)is correct only when the floor-diaphragm is assumed to be flexible. Each bent will then have its own independent deflection. But the floor-diaphragms in are in forced concrete building have far greater rigidity (or stiffness) than the vertical resisting elements like bents and/or shear walls. Due to greater rigidity of floor diaphragms the structure responds as a whole and not as separate individual bents, when subjected to horizontal loads. The horizontal loads, assume to act the level of floors, cause floor diaphragms to translate in the case of concentric shear (Fig. 7.1) and to rotate as well, in the case of eccentric shear (Fig. 7.2). The diaphragm, assumed infinitely rigid, forces all the vertical resisting elements to deflect by the same amount along with it. The total horizontal shear gets divided between the various bents in the ratio of the irresistance to deflection which is termed as rigidity. The more rigid an

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element, the more horizontal shear it resists. Hence, the practice of designing various bents of a reinforced concrete structure for tributary horizontal loads jeopardizes its safety. It is, therefore, necessary to find more precise means of distributing horizontal shear acting on a building to the various bents resisting it.

A simple method of computing the relative rigidity of bents is given byCross and Morgan for low industrial buildings with the assumption of infinitely stiff beams. It is proposed here to extend the method to framed reinforced concrete multi-storeyed buildings, for calculating the shear distribution to various bents basing it on the assumption of infinite stiffness of beams connecting either end of a column in a given storey of building. This gives full fixity at both ends of columns in a storey and neglects flexure of the connecting beams. It is then easy to derive an expression for the rigidity of a column and thereby of a bent in a given storey. The storey shear due to horizontal loads will then be distributed to the various bents resisting it, taking into consideration the eccentricity of the storey shear with respect to the centre of rigidity of bents.

A number of precise manual methods for the solution of this problem re available in literature which take into account the actual stiffness of the beams connecting a column at one or both of its ends. However, the labour involved in their use is quite considerable and proves more often a deterrant to their extensive application. These methods are also prone to the errors of the numerical sort. Their use is advisable only in the design of important and irregular structures, for which now, computer methods are also available.

The steps involved in the proposed method of rigidities are given and explained below :

- `Step 1 :`
  Calculate the total horizontal force due to wind or earthquakeand locate its point of application with reference to any bent in the direction ofthe force.
- `Step 2:`
  Calculate the rigidity of each column. The deflection of a column with both ends

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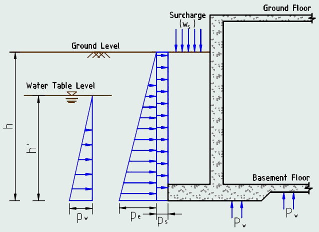

Figure 7.3: Rigidity of a column.

fixed (Fig: 7.3) for a unit shear at top, called the flexibility of a column is given by

$$\Delta = \frac{h^3}{12EL} = \frac{h^2}{12EK} \tag{7.1}$$

where k = /h = the relative stiffnes of column of height h and moment of inertia l E =modulus of elasticity of material of column. The rigidityRof a column, which is defined as the reciprocal of flexibility is then given by

$$R = \frac{1}{\Delta} = \frac{12Ek}{h^2} \tag{7.2}$$

- Calculate the rigidity of a bent. This can be computed with the help of parallel and series combinations of columns.44 For a system "in parallel" (Fig: 7.4), where the shear gets divided between the constituents, the rigidity of a bent equals the sum of rigidities of the columns constituting it. This gives

$$R = R_1 + R_2 + ... + R_n = \sum_{i=1}^{i=n} R_1 \tag{7.3}$$

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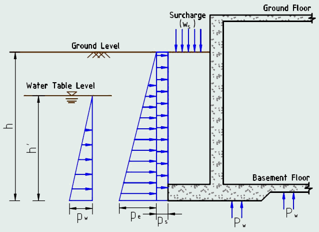

Figure 7.4: System in parallel.

For a system "in series" (Fig: **??**), where the applied shear remains thesame in all the constituents, the flexibility of a bent equals the sum of flexibilities of the constituent columns. This gives

$$\Delta = \Delta_1 + \Delta_2 + ... + \Delta_s \tag{7.4}$$

The rigidity of such a bent is given by,

$$R = \frac{1}{\Delta} = \frac{1}{\frac{1}{R_1} + \frac{1}{R_2} + ... + \frac{1}{R_n}} = \frac{1}{\sum_{i=1}^{i=n} \left(\frac{1}{R_i}\right)} \tag{7.5}$$

The rigidity of a bent in each storey is found by the use of parallel combination and. its over all rigidity by the use of series combination. Fig. 7.6gives an example for calculating the rigidity of a bent employing parallel andseries combinations. Refering to (Fig: 7.6), the following parallel combinationsgive

$$\begin{aligned}
R_{gh} &= 10 + 10 &= 20 \\
R_{abc} &= 8 + 16 + 16 &= 40 \\
R_{def} &= 20 + 20 + 10 &= 50 \\
R_{abcdef} &= 40 + 50 &= 90
\end{aligned}$$

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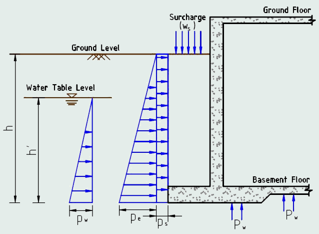

Figure 7.5: System in parallel.

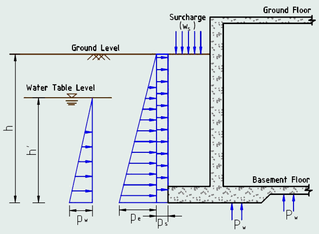

Figure 7.6: System in parallel.

The systems gh and abcdef, being in series, give the overall rigidity of the bent as

$$R = \frac{1}{\dfrac{1}{90} + \dfrac{1}{20}} = \frac{180}{11} = 16.36$$

It is, of course, assumed that bents have a rigidity in one principal direction only but not in both. The individual columns are taken into consideration in both the principal directions. When the rigidities of bents are thus known, the centre of rigidity of the building is located with reference to any given bent in each principal direction. The eccentricity "e" of the applied horizontal shear with respect to the

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centre of rigidity is then known.

- Calculate the direct shear on bents assuming the eccentricity e = 0. Taking the applied horizontal shear P_y in the y-direction to act concentric with the centre of rigidity of bents resisting it, the direct shear P_y on a bent of rigidity R_y is given by

$$P_y = P_y \frac{R_y}{\sum R_y} = P_y r_y \tag{7.6}$$

where, $r_y = \dfrac{R_y}{\sum R_y}$ = relative rigidity of a bent of rigidity R_y
A similar formal in the x-direction can be written as

$$P_x = P_x \frac{R_x}{\sum R_x} = P_x r_x \tag{7.7}$$

The horizontal loads like wind or earthquake, are assumed to act in either principal direction, one at a time.

- Calculate design shear on bents taking into account the effect of eccentricity, if any. When e = 0 or is negligibly small, the results given by step 4 give the design shears on bents. However, when the applied shear does not pass through the centre of rigidity (e $\neq$ 0), the design shear on a bent in the y-direction, situated at a distance x from the origin of axes, located at the centre of rigidity of building is given by (Fig: 7.7)

$$V_y = P_y \frac{R_y}{\sum R_y}\left(1 - \frac{ex\sum R_y}{J_p}\right) \tag{7.8}$$

where.$J_p = \sum\left(R_x y^2\right) + \sum\left(R_y x^2\right)$

and where the clockwise torsional moments are considered positive and forces are considered positive in the positive coordinate directions.
Denoting,

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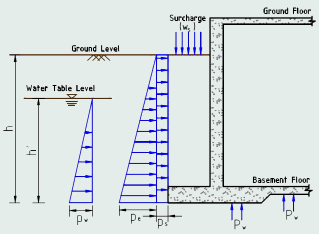

Figure 7.7: System in parallel.

$$F_y = \left(1 - ex\frac{\sum R_y}{J_p}\right) \tag{7.9}$$

Eq: **??** gives with the help of Eq: **??**

$$V_y = p_y F_y \tag{7.10}$$

As p_y is the direct shear on a bent of rigidity R_y when the eccentricity e=0, F_y is the multiplying factor to correct it for the torsional effect due to the eccentricity of the applied shear. When P_y is concentric with the centre of rigidity (e = 0), thenF y= 1.0 and Eq: **??** reduces to Eq: **??**. Similarly, for bents in x direction,

$$V_s = P_x \frac{R_s}{\sum R_x}\left(1 + ey\frac{\sum R_s}{J_p}\right) \tag{7.11}$$

$$P_s = \left(1 + ey\frac{\sum R_s}{J_p}\right) \tag{7.12}$$

$$V_x = p_x F_x \tag{7.13}$$

The eccentricity of the applied shear causes torsion of the floor diaphragm, thereby increasing the shear in some bents and reducing it in the rest.Only the increase

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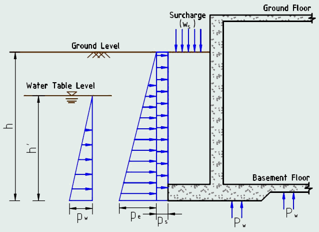

Figure 7.8: System in parallel.

of shear due to torsion is accounted for, in the design and the decrease of shear is neglected, to err on the side of safety.9 Therefore, the direct shear or the shear corrected for eccentricity, whichever greater, is considered for design. This gives,

$$F_y = F_x \geq 1.0 \tag{7.14}$$

This procedure is repeated for each storey of a given building, giving the share of applied horizontal shear of each bent at all floor levels. Normally, the arrangement of bents is kept the same for all storeys in earthquake resistant structures and hence this procedure needs to be attempted only once, giving the ratio of the total storey shear distributed to each bent, which is a fixed ratio for all storeys while the storey shear varies from storey to storey. The procedure given above is called the allocation analysis, while the bents with the horizontal loads so found at all floor levels can be analysed by the methods given under frame analysis later in Section 7.3

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8 Shear-Walled Buildings underHorizontal Loads

8.1. INTRODUCTION

Reinforced concrete framed buildings are adequate for resisting both the vertical and the horizontal loads acting on them. However, when the buildings are tall, say, more than twelve storeys or so, beam and column sizes work out large and reinforcement at the beam-column junctions works out quite heavy, so that, there is a lot of congestion at these joints and it is difficult to place and vibrate concrete at these places, which fact, does not contribute to the safety of buildings. These practical difficulties call for introduction of shear walls in tall buildings. Shear walls in plan, may be deep straight walls or angular, U- shaped or box-shaped in plan, around stairs or lifts or toilets, where there will be no architectural difficulty in extending them throughout the height of the building. Care shall be taken to have a symmetrical configuration of walls in plan so that torsional effect in plan could be avoided. Further, shear walls should get enough vertical load from floors, for which reason, nearby columns should be omitted and load taken to the shear walls by means of long-span beams if required. [13]

The role of floor diaphragm is as important as it is in the case of framed buildings. The floor diaphragm forces all the vertical elements like frames and shear walls to share the incumbent horizontal shear in the ratio of their rigidities or stiffnesses. To calculate the share of the total horizontal shear of each shear wall element is a major task and it may be called 'allocation analysis', while each shear wall under the assigned horizontal shears at floor levels acts as a vertical cantilever beam fixed at base and its analysis and design will be given later under the head 'shear wall analysis and design'.

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8.2. ALLOCATION ANALYSIS

In framed buidlings, horizontal forces due to wind or earthquake are resisted by frames in proportion to their rigidities as indicated in Chapter 7. In tall buildings of moderate heights (say, upto 20 storeys), where both frames and shear walls must be provided, horizontal forces are assumed to be fully resisted by shear walls alone, with frames being designed for at least 25% of the total horizontal load.[11] For taller buildings, the rigidity of shear walls in the upper storeys gets reduced due to the accumulation of deflection of the storeys below, necessitating joint participation of frames and shear walls to resist horizontal forces.[42] The assumption of all horizontal loads being taken by shear walls alone, is then no more valid and more accurate methods must be adopted to apportion the horizontal shear between frames and shear walls (see Chapter 9).[48-50]

It is proposed here to discuss the problems involved in the analysis of shear wall structures which, in essence, means to determine,the share of storey shear resisted by each shear wall for each storey in succession. It is assumed that either there exist no frames in a shear wall structure (which may be practicable in some cases only) or that the frames, if present, do not participate in resisting horizontal forces. It is further assumed that floor diaphragm is infinitely rigid in its own plane or at least it is more rigid than any of the shear walls joining it and that the foundation of shear wall is sufficiently rigid to ensure its fixity at base. The work of Benjamin[51] has been summarized by Assudani and Varyani[52] and it is given below.

8.2.1. Response of Structure

In reinforced concrete buildings, floors have far greater rigidity than the vertical resisting wall elements called shear walls. The horizontal loads caused by wind or earthquake are assumed to act at the level of floors, which deflect (translate and/or rotate) under their effect. The floor diaphragm assumed infinitely rigid, forces all the vertical resisting elements to deflect by the same amount along with it. The total horizontal shear in each

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storey gets divided between the various shear walls in proportion to their resistance to deflection which is termed as rigidity. The more rigid an element, the Tnore horizorital shear it resists. As shear walls are far more rigid than bents, they are assumed to resist the full amount of the horizontal shear. Shear walls, regarded as vertical cantilevers fixed at base, then transfer the horizontal forces, to the foundation.

To produce simple bending of a shear wall, the external horizontal shear acting on it must pass through its shear centre, failing which, bending is accompanied by torsion.[53] Shear walls may assume variety of shapes in plan, some of which require additional forces to prevent bending about the axis perpendicular to the axis of applied shear passing through its shear centre to ensure the condition of simple bending. Certain shapes of shear walls, particularly box-types, offer considerable resistance to torsional rotation, while others have negligible torsional rigidity. It is, therefore, necessary to be familiar with the rigidity characteristics of various shapes of shear walls.

8.2.2. Rigidity of a Shear Wall in a Given Direction (R_x, R_y)

Rigidity of a wall element in a given direction (R_x or R_y) is defined as a force per unit displacement in the given direction. The deflection ($\triangle x$) of a wall element regarded as a deep cantilever beam fixed at base due to a shear V_x applied in the x-direction at a height h' from top is composed of terms due to bending and shear deflections and it is given by (Fig. 8.1),

$$\triangle x = \frac{V_x h^3}{3El_y} + \frac{V_x h' h_2}{2El_y} + \frac{1.2V_x h}{A_y G}$$

Assuming G = E/2.2 for concrete, the rigidity of wall elements in the x-direction is given by definition as,

$$R_x = \frac{V_x}{\triangle x} = \frac{1}{\dfrac{h_3}{3El_y} + \dfrac{h' h_2}{2El_y} + \dfrac{2.64h}{A_y E}} \tag{8.1}$$

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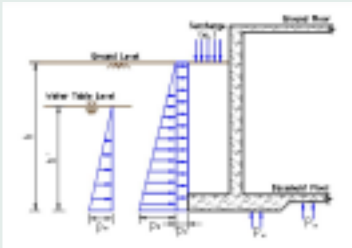

Figure 8.1: Deflection of cantilever shear wall subject to lateral load.

Similarly, the rigidity of the element in the y-direction is given by,

$$R_y = \frac{V_y}{\triangle y} = \frac{1}{\dfrac{h_3}{3El_x} + \dfrac{h'h_2}{2El_x} + \dfrac{2.64h}{A_x E}} \tag{8.2}$$

The notation used above is explained below:

$$\begin{aligned}
E &= modulus of elasticity of material of shearwall \\
G &= shearmodulus of material of shearwall \\
l &= moment of inertia of shearwall about the axis of bending \\
A &= area of web about the axis of bending \\
V &= applied shear in a given direction \\
\triangle &= deflection due to applied shear in a given direction \\
h &= height of shearwall element \\
h' &= height of applied shear above the top of shearwall element
\end{aligned}$$

Considering each storey in turn, it is normally assumed that V acts at the level of floors, that is, at the top of a shear wall making h' = 0 and Eqs. (8.1) and (8.2) are simplified

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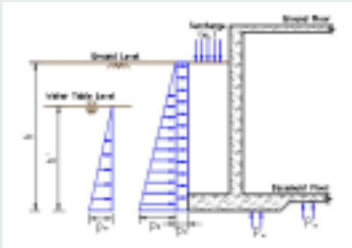

Figure 8.2: Rigidity of a cantilever wall element fixed at base.

to

$$R_x = \frac{1}{\dfrac{h_3}{3El_y} + \dfrac{2.64h}{A_yE}} \tag{8.3}$$

$$R_y = \frac{1}{\dfrac{h_3}{3El_x} + \dfrac{2.64h}{A_xE}} \tag{8.4}$$

Rigidities of several shapes of shear walls, commonly met with in practice, are given in Table 8.1 along with the values of I and A, assuming the thickness of wall to be very small in comparison to its overall dimensions.

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8.2.3. Solid Rectangular Shear Walls

Referring to Fig. 8.2, with t $\ll$ L,

$$l_y = \frac{1}{12} L^3 t$$

$$l_x = \frac{1}{12} L t^3$$

$$A_y = A_x = Lt$$

Substituting the above values in Eq. (8.1), we have the rigidity of shear wall in the direction of its length R_x (say, R),

$$R = \frac{Et}{4\left(\frac{h}{L}\right)^3 + 6\left(\frac{h'}{L}\right)\left(\frac{h}{L}\right)^2 + 2.64\left(\frac{h}{L}\right)} \tag{8.5}$$

The rigidity of the shear wall in the direction of its thickness, R_y is seen to be negligible from Eq. (8.2). Fig. 8.2 gives charts for rapid evaluation of rigidities of solid rectangular wall elements.

8.2.4. Shear Wall with Openings

Piers in a wall formed by openings may be regarded as fixed at both ends, which changes the bending deflection term $h^3/3El$ to $h^3/12El$ in Eqs. (8.3) and (8.4). The rigidity of a pier (Fig. 8.3) is then given in the direction of its length,

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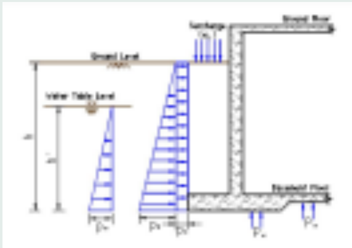

Figure 8.3: Rigidity of a wall element fixed at both ends.

$$R = \frac{Et}{\dfrac{h^3}{L} + 2.64\dfrac{h}{L}} \tag{8.6}$$

Fig. 8.3 gives a curve for rapid evaluation of the rigidity of piers. The rigidity of a pier in the direction of its thickness is negligibly small.

The rigidity of a wall with openings may be calculated neglecting the effect of the axial shortening of piers by the judicious use of the principles of series and paralles in the same way as explained for bents (Chapter 7).[38] It is seen that for normal window or door openings, the rigidity of the wall is not affected to any appreciable extent. The rigidity of a shear wall is due more to its form than to its mass. In order that the effect of openings on the rigidity of shear wall is negligible, the size of the openings should be relatively small and these should be spaced at least a distance equal to the size of the openings in each direction.[54] To restrict the stresses in the shear wall, the width of openings should be limited approximately to 15% of the total length of the connected shear walls and the depth of the connecting beam should be greater than 20% of the storey height.[55]

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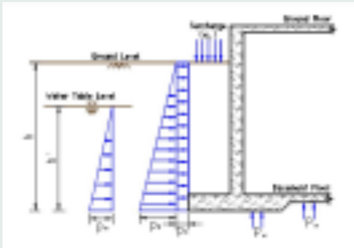

Figure 8.4: Direction of Rxy and Ryx for various dispositions of the angle wall element, (a) Angle in position 1; (b) Angle in position 2; (c) Angle in position 3; (d) Angle in position 4.

8.2.5. Rigidity of a Wall Element Normal to the Direction of Horizontal Shear to Ensure Simple Bending(R_{xy}, R_{yx})

R_{yx} is defined as the horizontal force necessary to prevent y-distortion of a wall element when Rx is applied in the x-direction producing a unit deflection. R_{xy} is also simiarly defined. Fig. 8.4 gives the directions of R_{xy} and R_{yx} for various positions of the angle section. When the principal axes of the shape of a shear wall are parallel to the X and Y axes, R_{xy} and R_{yx} vanish. Table 8.1 gives the values of R_{xy} and R_{yx} calculated as explained by Benjamin.[51]

8.2.6. Torsional Rigidity of a Shear Wall

Torsional rigidity of a shear wall is defined as the torque required to produce a unit rotation. If a torque T acting on a shear wall produces a rotation of θ radians, then the torsional rigidity of the wall is,

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$$J = \frac{T}{\theta} \quad (8.7)$$

The torsional rigidity of any given shape of a shear wall consists of the summation of its torsional rigidities calculated on the basis of uniform and non-uniform torsional theories, the uniform-torsion theory component for open sections only, being given by,

$$J_\theta = \frac{Et^3 \Sigma\theta}{6.6h} \quad (8.8)$$

where $\Sigma\theta$ equals the perimeter of the section of shear wall of height h and thickness t. It, however, works out to be negligibly small in the case of open sections such as channels, angles, tees, etc. being a function of the cube of thickness of shear wall, which is assumed to be very small in comparison to its other dimensions. However, for box sections, it is not a small quantity, as it is proportional to its thickness (as per the expression in Table 8.1): The non uniform torison theory applies to flanged walls of open cross-sections and gives approximate torsion rigidity based on the rigidities of separated flanges opposite to each other, neglecting the web. Referring to Fig. **??** for an I-section, rotation θ in radians due to a unit displacement of either flange on account of force R_f is given by,

$\theta = \frac{2}{a}$

which is produced by a torque
T = R_f x a
where R_f = rigidity of the flange, i.e. a wall element of length b and thickness t (may be easily evaluated by Fig. 8.2).
The torsional rigidity is, by definition, given on the basis of non-uniform torsion theory as,

$$J_n = \frac{T}{\theta} = R_f x \frac{a^2}{2} \quad (8.9)$$

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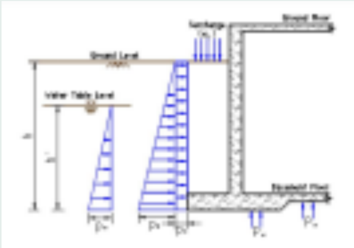

Figure 8.5: Torsional rigidity of a shear wall of open section based on non-uniform torsion theory (R_f = rigidity of wall element b x f).

For box sections, the torsion due to non-uniform theory can be neglected. The torsional rigidity of a shear wall is then given by,

$$J = J_u + Jn \tag{8.10}$$

Table 8.1 gives the values of torsional rigidities of various shapes of shear walls.

8.2.7. Shear Centre of a Shear Wall

The shear centre may be defined as the point through which the plane of loading must pass to eliminate torsion.[53] The shear centres of various shapes of shear walls are given in Table 8.1. The rigidities R_x, R_y, R_{xy}, R_{yx} of a shear wall are all assumed to act through its shear centre.

Table 8.1 gives a summary of the rigidity characteristics of the various shapes of shear walls frequently met with in practice, which have been evaluated on the principles discussed by Benjamin[51] and Timoshenko.[53]

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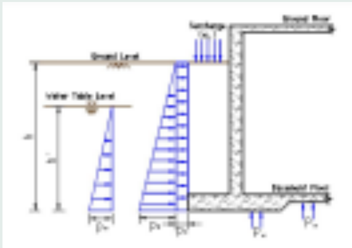

Figure 8.6: Shear centre O of a shear wall structure (Px, Py are not to act simultaneously). Sign convention: clockwise moments +ve in the increasing direction of x and y.

8.2.8. Shear Centre of a Shear Wall Structure

Shear centre of a shear wall structure in a story is the centre of rigidities of all the shear walls which partake of the applied horizontal shear due to the infinite rigidity of the floor diaphragm. The qoordinates of the shear centre of a structure are fixed by taking moments of rigidities R_x and R_{yx} and separately of rigidities R_y and R_{xy} of all the shear walls about any convenient point O_1 and equating them to the corresponding moments of the summation of the above quantities taking them to be concentrated at the shear centre of the structure. This gives (Fig. 8.6) assuming positive sign for all forces.

$$\overline{y}\Sigma R_x - \overline{x}\Sigma R_{yx} = \Sigma(y_1 R_x) - \Sigma(x_1 R_{yx}) \tag{8.11}$$

$$\overline{y}\Sigma R_{xy} - \overline{x}\Sigma R_y = \Sigma(y_1 R_{xy}) - \Sigma(x_1 R_y) \tag{8.12}$$

where $(\overline{x}, \overline{y})$ give the location of the shear centre of the structure and the point chosen O_1 is the origin of axes x_1 and y_1 and the summation extends to all the shear walls joined monolithically with the floor diaphragm. Eqs. (8.11) and (8.12) are solved for x and y to locate the shear centre of the structure, taking proper signs of all the forces involved.

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In the case, when $R_{xy} = R_{yx} = 0$ for all the existing shear walls, Eqs. (8.11) and (8.12) simplify to

$$\overline{x} = \frac{\Sigma(x_1 R_y)}{\Sigma R_y} \tag{8.13}$$

$$\overline{y} = \frac{\Sigma(y_1 R_x)}{\Sigma R_x} \tag{8.14}$$

8.2.9. Evaluation of Applied Horizontal Load

Horizontal force in each storey of a structure is evaluated in accordance with the provisions of IS:875[2] for wind and of IS:1893 [9] for earthquake. The point of application of the horizontal force is decided from the plan of floor under discussion. It is normally assumed that wind and earthquake do not act simultaneously and that it suffices to design a structure for wind or earthquake(whichever is greater) in each principal direction separately. The eccentricity of the applied horizontal force with respect to the shear centre of the structure is then computed. The minimum eccentricity must not be less than 5% of the maximum building dimension.[56] Referring to Fig. 8.6, it is seen that the floor diaphragm is under the action of applied horizontal forces P_x or P_y and a torsional moment T_p equalling $+P_x$.ey or $-Py$.ex respectively, where P_x, P_y are positive in the increasing direction of x ,y and T_p is positive if clockwise.

8.2.10. Distribution of Applied Horizontal Forces to Shear Walls

This is the problem of allocation analysis. It consists in determining the distribution of the applied forces P_x or P_y and the corresponding torsional moment T_p to the various shear walls forming the structure.
Direct Shear When there is no torsion, that is, when the shear centre of structure

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coincides with the point of application of the resultant lateral loads, the direct shear in each shear wall due to P_y or P_x is given by, when only P_y acts with $P_x = 0$,

$$F_y = P_y \frac{R_y \Sigma R_x - R_{xy} \Sigma R_{yx}}{\Sigma R_y \Sigma R_x - \Sigma R_{yx} \Sigma R_{xy}} \tag{8.15}$$

$$F_x = P_y \frac{R_{yx} \Sigma R_x - R_x \Sigma R_{yx}}{\Sigma R_y \Sigma R_x - \Sigma R_{yx} \Sigma R_{xy}} \tag{8.16}$$

when P_x acts with $P_y = 0$,

$$F_x = P_x \frac{R_x \Sigma R_y - R_{yx} \Sigma R_{xy}}{\Sigma R_y \Sigma R_x - \Sigma R_{yx} \Sigma R_{xy}} \tag{8.17}$$

$$F_y = P_x \frac{R_{xy} \Sigma R_y - R_y \Sigma R_{xy}}{\Sigma R_y \Sigma R_x - \Sigma R_{yx} \Sigma R_{xy}} \tag{8.18}$$

When R_{xy} and R_{yx} both are equal to zero for all shear walls, Eqs. (8.15) to (8.18) are reduced to, when only P_y acts,

$$F_y = P_y \frac{R_y}{\Sigma R_y} \tag{8.19}$$

$$F_x = 0 \tag{8.20}$$

when only P_x acts,

$$F_x = P_x \frac{R_x}{\Sigma R_x} \tag{8.21}$$

$$F_y = 0 \tag{8.22}$$

Shear due to Torsion When there is eccentricity of lateral loads, that is, when the point of application of the resultant of lateral loads does not coincide with the shear centre of the structure, there will be torsion in plan and the diaphragm will rotate about the shear centre of the structure. The shears S_x and S_y in directions x and y respectively and the torque T acting on each shear wall due to torsional moment T_p are given by,

$$S_x = T_p \frac{y R_x - x R_{yx}}{J_p} \tag{8.23}$$

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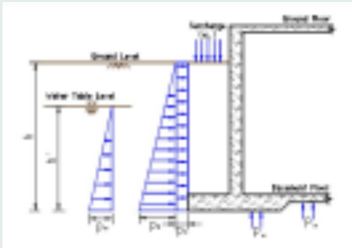

Figure 8.7: Flange forces in wall sections under torsion, (a) Open section; (b) Closed section.

The force in the flange of a shear wall due to the torque T is given by Benjamin [51] as follows:
For open sections (Fig. 8.7)

$$N_x = \pm \frac{T}{a} \tag{8.24}$$

For closed sections (Fig. 8.7)

$$N_x = \pm \frac{T}{2a} \tag{8.25}$$

$$N_y = \pm \frac{T}{2b} \tag{8.26}$$

Final shear The final theoretical shear in each shear wall is equal to the direct shear plus the shear due to torsion and it is given by,

$$Q_x = F_x + S_x \tag{8.27}$$

$$Q_y = F_y + S_y \tag{8.28}$$

where F_x, F_y and S_x, S_y are to be determined from the relevant aligns given above.
Torsion of floor diaphragm, caused by eccentricity of applied forces adds to direct shear in some wall elements and reduces it in the rest. Only the increase of shear due to

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torsion is accounted for, in the design and the decrease of shear is neglected to err on the side of safety. Therefore, the direct shear or the final theoretical shear, whichever is greater, gives the final design shear on walls.

8.2.11. Procedure for Multistorey Shear Walled Buildings

The procedure for the distribution of applied forces to the vertical resisting wall elements explained in the section 8.2.10 is repeated for each storey of a given buidling, giving the share of applied storey shear of each wall element at the floor levels. Normally, the arrangement of shear walls is kept the same for all storeys in structures which are likely to be subjected to high intensity of wind or earthquake. In that case, this procedure needs to be attempted only once, giving the ratio of the total storey shear distributed to each shear wall, which is then considered fixed for all storeys, while only the storey shear varies from storey to storey. Each individual shear wall with the horizontal loads, so found at all floor levels, is then analysed and designed to cover the internal forces fully by adequate amount of concrete and steel.[16]

In some structures at least, it may be possible to provide expansion joints in such a way that there remains only one wall element, generally, a shear-box or a core to resist the horizontal forces applied to the building, in which case, the allocation analysis is neatly avoided and it remains only to design the core for the applied horizontal load of the entire block, together with its tributary vertical load.

Benjamin[51] has shown that application of the above procedure to each storey in succession leads to no appreciable error, provided the structural arrangement of shear walls and the loading on floors are regular throughout the height of building. For neglecting the interaction of frames and shear walls, the structure is required to be of a moderate height.[57] The procedure will be illustrated below by an example.

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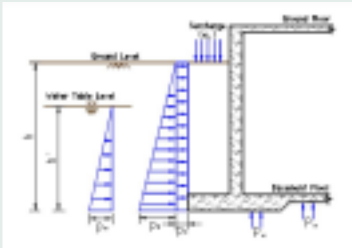

Figure 8.8: Plan of a shear wall structure of the illustrative Example 8.2.12.(Wall thickness = 6” everywhere.)

8.2.12. Illustrative Example

Fig. 8.8 gives a floor plan of a shear walled multistorey structure. The horizontal shear in the storey under consideration is denoted by P_y acting on long side, along the centre-line of the building. The storey height is taken as 15 ft. It is required to distribute P_y to the wall elements A, B, C and D forming the structure.

Step 1: *Evaluation of rigidity characteristics of shear-walls* As all the shear walls forming the structure are of the same material, E is being omitted from all expressions of rigidity characteristics.

Shear Wall A. Case 2 of Table 8.1 gives

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$$
\begin{aligned}
A_x &= at = 40 \times 0.5 = 20.0ft \\
A_y &= 2bt = 2 \times 10 \times 0.5 = 10.0sft \\
I_x &= \frac{a^2 t}{12}(a+6b) = \frac{1}{12} \times (40)^2 \times 0.5(40+60) = 6670ft^4 \\
I_y &= \frac{b^3 t}{3}\frac{2a+b}{a+2b} = \frac{1}{3} \times (10)^3 \times 0.5\frac{(80+10)}{(40+20)} = 250ft^4 \\
R_x &= \frac{1}{\dfrac{h^3}{3I_y} + \dfrac{2.64h}{A_y}} = \frac{1}{\dfrac{(15)^3}{3 \times 250} + \dfrac{2.64 \times 15}{10}} = 0.188 \\
R_y &= \frac{1}{\dfrac{h^3}{3I_x} + \dfrac{2.64h}{A_x}} = \frac{1}{\dfrac{(15)^3}{3 \times 6670} + \dfrac{2.64 \times 15}{20}} = 0.465 \\
R_{xy} &= R_{yx} = 0 \\
J &= R_f \frac{a^2}{2}
\end{aligned}
$$

For $\frac{h'}{L} = 0$, $\frac{h}{L} = 1.5$, Fig. 8.2 gives $R_f = 0.03$.

Then,

$$
\begin{aligned}
J &= 0.03 \times (\frac{40_2}{2}) = 24 \\
m &= \frac{3b^2}{a+6b} = \frac{3 \times (10)^2}{40+60} = 3.0ft.
\end{aligned}
$$

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Shear Wall B. Referring to Case 8 of Table 8.1 and Fig. 8.4. we have,

$$
\begin{aligned}
A_x &= A_y = at = 20 \times 0.5 = 10.0 ft^2 \\
I_x &= I_y = \frac{5}{24}a^3t = \frac{5}{24} \times (20)^3 \times .5 = 835 ft^4 \\
R_x &= R_y = \frac{1}{\frac{h^3}{3I_x} + \frac{2.64h}{A_x}} = \frac{1}{\frac{(15)^3}{3 \times 835} + \frac{2.64 \times 15}{10}} = 0.188 \\
R_{xy} &= +0.6R_x = +0.6 \times .188 = +0.113 \\
R_{yx} &= -0.6R_x = -0.113 \\
J &= 0 \\
m_x &= m_y = \frac{a}{4} = \frac{20}{4} = 5.0 ft.
\end{aligned}
$$

Shear Wall C

$$
For \frac{h'}{L} = 0, \frac{h}{L} = \frac{15}{10} = 1.5, \text{Fig. 8.2 gives}
$$
$$
R = R_y = 0.06 \times 0.5 = 0.03
$$

Case 1 of Table 8.1 with change of axes, gives,

$R_x = 0$

$R_{xy} = R_{yx} = 0, J = 0, m = 0$

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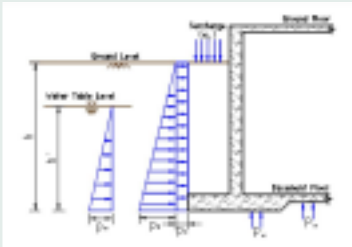

Figure 8.9: Summary of rigidities of shear walls and location of the shear centre O of the structure.

Shear Wall D. Case 5 of Table 8.1 leads to,

$$
\begin{aligned}
A_x &= A_y = 2at = 2 \times 8.0 \times 0.5 = 8.0ft^2 \\
I_x &= I_y = \frac{2}{3}a^3t = \frac{2}{3} \times (8)^3 \times .5 = 171.0ft^4 \\
R_x &= R_y = \frac{1}{\dfrac{h^3}{3I_x} + \dfrac{2.6h}{A_x}} = \frac{1}{\dfrac{(15)^3}{3 \times 171} + \dfrac{2.64 \times 15}{8}} = 0.087 \\
R_{xy} &= R_{yx}0 \\
J &= \frac{a^2b^2t}{1.1h(a+b)} = \frac{64 \times 64 \times 0.5}{1.1 \times 15(8+8)} = 7.8 \\
m_x &= 0
\end{aligned}
$$

A summary of the rigidity characteristics of shear walls is given in Fig. 8.9 and Table 8.2. The directions of the rigidities of shear walls are also indicated in Fig. 8.9.

Table 8.2. Summary of rigidity characteristics of shear walls of the example.

Step 2: *Location of shear centre of the structure.* Assuming the origin of axes (x_1, y_1) to be coincident with the shear centre of the shear wall A, the shear centre of the structure $(\overline{x}, \overline{y})$ is given by Eqs. (8.11) and (8.12), the various quantities required in the

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Shear Wall	R_x	R_y	R_{xy}	R_{yx}	m	J
A	0.118	0.465	0	0	3.0	24.0
B	0.188	0.188	0.113	-0.113	5.0	0
C	0	0.030	0	0	0	0
D	0.087	0.087	0	0	0	7.8
Σ	0.393	0.770	0.113	-0.113	-	31.8

above equations, being computed with the help of Table 8.3. This leads to the following equations:

$$\overline{y} \times 0.393 - \overline{x}(-0.113) = -3.76 - (-9.4)$$
$$\overline{y} \times 0.113 - \overline{x}(-0.770) = -2.26 - 21.83$$

which on solution give,

$$\overline{x} = 32.2ft$$
$$\overline{y} = 5.1ft$$

Step 3: *Calculation of eccentricity of the applied horizontal load.* Fig. 8.9 shows the position of the shear centre of the structure and the location of the line of action of the applied horizontal load P_y, giving its eccentricity.

Table 8.3. Quantities required for location of shear centre of structure.

Table 8.4. Calculation of direct shear on walls.

$\Sigma R_y \Sigma R_x - \Sigma R_{yx} \Sigma R_{xy} = 0.77 \times 0.393 - (-0.113) \times 0.113 = 0.3148$

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Shear Wall	X 1 - Direction						Y1 - Direction					
	R_x	R_{yx}	X_1	Y_1	Y_1R_x	X_1R_{yx}	R_y	R_{xy}	X_1	Y_1	X_1R_y	Y_1R_{xy}
A	0.118	0	0	0	0	0	0.465	0	0	0	0	0
B	0.188	-0.113	83	-20	-3.76	-9.4	0.188	0.113	83	-20	15.60	-2.26
C	0	0	83	15	0	0	0.030	0	83	15	2.49	0
D	0.087	0	43	0	0	0	0.087	0	43	0	3.74	0
Σ	0.393	-0.93	-0.113	-	-3.76	-9.4	0.770	0.113	-	-	21.83	-2.26

Shear Wall	R_x	R_y	R_{xy}	R_{yx}	$R_y\Sigma R_x$	$R_{xy}\Sigma R_{yx}$	$R_y\Sigma R_{yx}$	$R_{yx}\Sigma R_x$	$F_y = \frac{1}{0.3148}$ $P_y(5) - (6)$	$F_x = \frac{P_y(8) - (7)}{0.3148}$
-	1	2	3	4	5	6	7	8	9	10
A	0.118	0.465	0	0	0.183	0	-0.0133	0	$0.580P_y$	$0.042P_y$
B	0.188	0.188	0.113	-0.113	0.074	-0.0128	-0.212	-0.0445	$0.274P_y$	$-0.074P_y$
C	0	0.03	0	0	0.0118	0	0	0	$0.037P_y$	0
D	0.087	0.087	0	0	0.0342	0	-0.0099	0	$0.109P_y$	$0.032P_y$
Σ	0.393	0.770	0.113	-0.113	-	-	-	-	$1.0P_y$	0

Table 8.5. Calculation of J_p

Shear Wall	X - Direction							Y - Direction						
	R_x	R_{yx}	Y	Y^2	Y^2R_x	XY	XYR_{yx}	R_y	R_{xy}	X	X^2	X^2R_y	XY	XYR_{xy}
A	0.118	0	-5.1	26	3.06	164	0	0.465	0	-32.2	1040	484	164	0
B	0.188	-0.113	-25.1	630	118.50	-1280	145	0.188	0.113	50.8	2580	485	-1280	-145
C	0	0	9.9	98	0	503	0	0.030	0	50.8	2580	77.5	503	0
D	0.087	0	-5.1	26	2.26	-55	0	0.087	0	10.8	117	10.2	55	0
Σ	0.393	-0.113	-	-	123.82	-	145	0.770	-0.113	-		1056.7	-	-145

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$$
\begin{aligned}
J_p &= \Sigma(Y^2.R_x) + \Sigma(X^2.R_y) - \Sigma(Y.R_{xy}) - \Sigma(XYR_{yx}) + \Sigma J \\
&= 123.82 + 1056.7 - (-145) - 145 + 31.8 = 1212.3
\end{aligned}
$$

$\Sigma J = 31.8 (From Table 8.2)$

Table 8.6. Effect of torsion on walls.

Shear Wall	Torsional shear												Force in fl		
	R_y	R_x	X	Y	YR_y	XR_y	R_{xy}	R_{yx}	XR_{yx}	YR_{xy}	S_x=0.0089 P_y={(5)-(9)}	S_y= 0.0089 P_y={(6)-(10)}	J	$T = -0.00^{Rq}$ P_y.J	
-	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
A	0.465	0.118	-32.2	-5.1	-0.60	-15.0	0	0	0	0	0.0053P_y	-0.133P_y	24	-0.214P_y	±0.0
B	0.188	0.188	50.8	-25.1	-4.72	9.55	0.113	-0.113	-5.75	-2.84	-0.0092P_y	0.111P_y	0	0	0
C	0.030	0	50.8	9.9	0	1.52	0	0	0	0	0	0.014P_y	0	0	0
D	0.087	0.087	10.8	-5.1	-0.44	0.94	0	0	0	0	0.0039P_y	0.008P_y	7.8	-0.07P_y	± 0.0
Σ	0.770	0.393	-	-	-	-	0.113	-0.113	-	-	0	0	31.8	-	0

$\frac{T_p}{J_p} = \frac{10.8}{1212.3} P_y = 0.0089 P_y$

$e_x = (40+3) - 32.3 = 10.8ft$

The torsional moment acting anti-clockwise (negative) on the structure is then given as $T_p = -P_y.e_x = -10.8P_y$

Step 4: *Calculation of direct shear on walls due to P_y only.* For this case, Eqs. (8.15) and (8.16) give the direct shears F_x and F_y on the various wall elements, the computations being given in Table 8.4.

Step 5: *Effect of torsional moment T_p.* Table 8.5 gives the calculations required to evaluate J_p. Eqs. (8.23) to (8.25) are used to compute the forces S_y, T on shear walls due to the torsional moment T_p, the computations being given in Table 8.6. The calculations for flange forces in shear walls A and D, based on Eqs. (8.31) and (8.32) respectively, are also shown in Table 8.6.

Step 6: *Final Shear acting on shear walls.* Final theoretical shear acting on a wall element is given by Eqs. (8.34) and (8.35), which is calculated in Table 8.7. Final design shears on walls are obtained by neglecting negative torsional shears. The final shears on walls are shown in Fig. 8.10 which also shows the forces on the flanges of shear walls A and D due to their individual torsional rigidities.

Table 8.7. Final theoretical and design shear on walls given as fraction of P_y

Shear Wall	Direct shear		Torsional shear		Final Theoretical Shear		Final design Shear	
	F_y	F_x	S_y	S_x	Q_y=(1)+(3)	Q_x=(2)+(4)	Q_y=(1) or (5)	Q_x=(2) or (6)
-	1	2	3	4	5	6	7	8
A	0.580	0.042	-0.133	0.0053	0.447	0.0473	0.580	0.0473
B	0.274	-0.074	0.111	-0.0092	0.385	-0.0832	0.385	-0.0832
C	0.0370	0	0.014	0	0.051	0	0.051	0
D	0.109	0.032	0.008	0.0039	0.117	0.0359	0.117	0.0359
Σ	1.0		0	0	0	1.00	0	1.1330

8.3. Shear Wall Analysis and Design

Each shear wall acts like a column under vertical load (P) from the supported floors and its self-weight. The wall shall be designed as a column, taking into account joint moments and additional moment due to slenderness. The hori zontal shears at each floor level on a wall element produce shear (H) and overturning moment (M) in wall, with the wall being regarded as a vertical cantilever beam fixed at base (Fig. 8.12). Each section of wall has to be de signed for P, M and H, taking advantage of increased stresses or lowered load factors as the overturning moment M and the horizontal shear H are both the result of either wind or earthquake forces.

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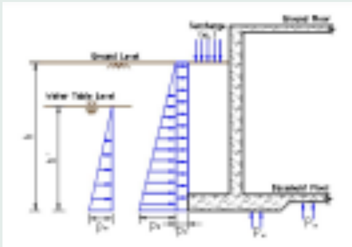

Figure 8.10: Final allocation of applied shear Py to shear walls given as fraction of it.

8.3.1. Solid Rectangular Wall Element

Column charts based on the limit state method as given in SP-16[58] can be used to design the reinforcement required in a rectangular wall element (L ×t). For the following data,

$$
\begin{aligned}
L &= 10.0m, P = 200.0t, f_{ck} = 150Kg/cm^2 \\
t &= 0.3m, M = 200.0tm, f_y = 4150Kg/cm^2
\end{aligned}
$$

$$
\begin{aligned}
H &= 20.0t \\
\frac{P_u}{f_{ck}bD} &= \frac{1.2 \times 200 \times 1000}{150 \times 1000 \times 30} = 0.053 \\
\frac{M_u}{f_{ck}bD^2} &= \frac{1.2 \times 200 \times 10^5}{150 \times 30 \times 1000^2} = 0.005
\end{aligned}
$$

Assuming $\frac{d'}{D} = \frac{12}{1000} = 0.012 \approx 0.05$, (Fig. 8.14) and with $\frac{A_s}{2}$ on each face,

$$
\begin{aligned}
\frac{p}{f_{ck}} &= 0.02, p = .02 \times 15 = 0.3 \\
A_s = 0.3 \times 30 \times \frac{1000}{100} &= 90cm^2
\end{aligned}
$$

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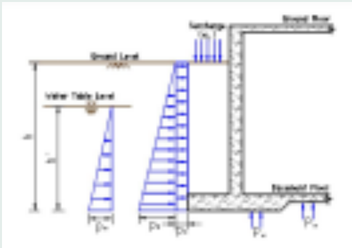
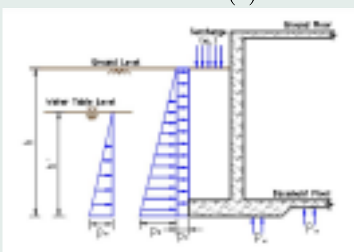


(a) a



(b) b

Figure 8.11: Solid rectangular shear wall, (a) Elevation of shear wall; (b) Section A-A.

Provided by 12 $\overline{\phi}32$ bars $= 96.51\ cm^2$

In the rest of the areas, steel as required for the vertical load $= \frac{P}{L} = \frac{200}{10} = 20.0 t/m$, together with the local moments and slenderness moments shall be provided. With,

$$\frac{P}{L} = 20.0t/m, M_{vL} = 2.0tm/m, l = 3.6m$$
$$\frac{l_{ef}}{b} = \frac{1.2 \times 3.6}{0.3} = 14.4 > 12,$$
$$M_a, 30 = kP \times .3 \times (14.4)^2/2000 = 0.031kP$$
$$= .031 \times 1 \times 20 = 0.6tm/m$$
$$\frac{P}{L} = 20.0tm/m, M30 = 2.0 + .6 = 2.6tm/m$$
$$\frac{P_u}{f_{ck}bD} = \frac{1.5 \times 20.0 \times 1000}{150 \times 1000 \times 30} = 0.067$$
$$\frac{M_u}{f_{ck}bD^2} = \frac{1.5 \times 2.6 \times 10^5}{150 \times 100 \times 30^2} = 0.029$$
$$d'/D = \frac{5}{30} = .167 = 0.20, a_s/2oneachface,$$
$$p/f_{ck} = 0, p = 0$$

Minimum steel as for reinforced concerete walls shall be provided.

Vertical steel = 0.4%, $A_{sv} = 0.4 \times 30 \times \frac{100}{100}$ $=12.0cm^2$/m

$\frac{A_{sv}}{2} = \frac{12.0}{2}$ $=6.0cm^2$/m

Provide ϕ12/150 c/c E.F. $=7.54cm^2$/m

Horizontal steel = 0.2%, $A_{sh} = 0.2 \times 30 \times \frac{100}{100}$ $=6.0cm^2$/m

$A_{sh}/2 = \frac{6.0}{2}$ $=3.0cm^2$/m

Provide ϕ10/200 c/c E.F. $=3.93cm^2$/m

Check on shear

$$T_v = \frac{1.2 \times 20.0 \times 1000}{1000 \times 30 \times 10} = 0.08N/mm_2$$
$$T_c = 0.35N/mm^2(takenasminium)$$

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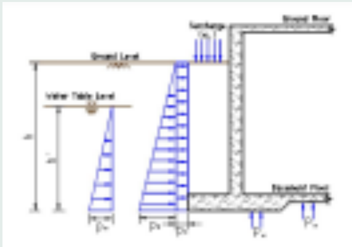

Figure 8.12: Steel arrangement in solid shear wall.

For small openings in walls, the design can be checked for the length available at a given section and extra bars shall be provided on each side of opening. The area of extra bars on both sides shall be equal to the area of bars cut off plus 25% extra. This rule applies to both the horizontal and the vertical steel bars.

8.3.2. Coupled Shear Walls

A coupled shear wall consists of two solid wall elements joined together by deep floor beams, giving openings for corridors in buildings (Fig. ??). The vertical load (P) is shared equally by the two solid elements if equal, other wise each wall element will have its own vertical load like P_1 and P_2. The overturning moment M causes a compression C in one wall element and a tension T in the other, where,

$$C = T = \frac{M}{a}$$
$$where, a = \frac{1}{2}(L + l')$$

The reversal of overturning moment will interchange these actions. The solid wall elements can be designed as columns. The connecting beam is to be designed for a

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shear equal in magnitude to C or T and moment equal to T.l'/2. With the following data,

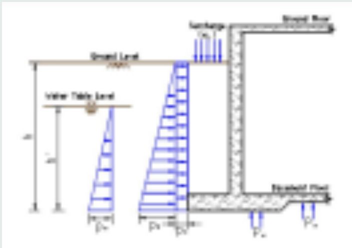
$$L = 10.0m \qquad \frac{L - l'}{2} = 3.5m \qquad M = 200.0tm$$
$$l' = 3.0m \qquad a = \frac{1}{2}(10 + 3) = 6.5m, \qquad P = 200.01$$
$$t = 0.3m \qquad \frac{P}{2} = 100.0t$$
$$C = T = \frac{M}{a} = \frac{200}{6.5} = 30.8t$$

For a solid element of length = 3.5 m, t = 0.3 m,
Max. compresssion = 100 + 30.8 = 130.8 t = 1311
Min. eccentricity moment = .02 P
= .02 x 131 = 2.62 tm

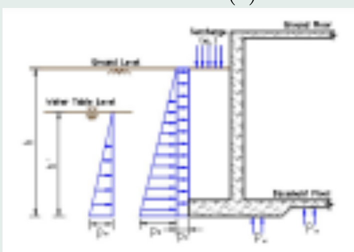
$$\begin{aligned} Slendernessmoment &= .031P \\ &= .031x131 = 4.06tm \\ ForP &= 131t, M30 = 2.62 + 4.06 = 6.68 = 6.7tm \\ \frac{P_u}{f_{ck}bD} &= \frac{1.2 \times 131 \times 1000}{150 \times 350 \times 30} = 0.1 \\ \frac{M_u}{f_{ck}bD^2} &= \frac{1.2 \times 6.7 \times 105}{150 \times 350 \times (30)^2} = 0.017 = .02 \end{aligned}$$

With $d'/D = 5/30 = .167 \approx .20, A_s/2$ on E.F.
$p/f_{ck} = 0$

Minimum wall steel will be provided. Taking it as a solid wall, we shall provide 6 $\overline{\phi}32$ bars at each outer end. For inner ends, steel area cut-off = $\overline{\phi}12/150$ E.F. for 3.0 m



(a) a



(b) b

Figure 8.13: Coupled shear wall, (a) Elevation of coupled shear wall; (b) Section A-A.

length
$= 7.54 \times 2 \times 3.0 = 45.24\ cm^2$
Steel to be provided at each inner edge $= \frac{45.24}{2} \times 1.25$
$= 28.3\ cm^2/\mathrm{m}$
Provided by 4 $\bar{\phi}$32 bars $= 32.17\ cm^2$

The steel arrangement is shown in Fig. **??**.
Tension side wall element is not critical, as the net load is compressive and equa.1 to 100-31 = 69 t. But, because of the reversal of direction of hori zontal loads, both the

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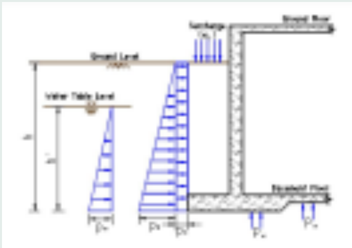

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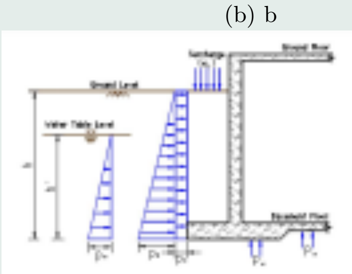
(b) b



(c) b

Figure 8.14: Steel arrangement in coupled shear wall, (a) Elevation of wall; (b) Section A-A of coupled shear wall; (c) Section B-B.

solid elements shall be made identical. For the connecting beam, we have a shear equal to 31 t and a moment = 31.0 ×3.0/2 = 46.5 tm. Due to direct vertical loading of

$$\begin{aligned} w &= .7 + 1.5 = 2.2t/m, \\ M &= 2.2 \times 3.0^2/12 = 1.7tm \\ S &= 2.2 \times 3.0/2 = 3.3t \end{aligned}$$

The total design shear = 31.0 + 3.3 = 34.3 t
The total design moment = 46.5 + 1.7 = 48.2 tm
With b = 30 cm, D = 90 cm, d = 85 cm,

$$\begin{aligned} k &= \frac{1.2 \times 48.2 \times 10^5}{30 \times (85)^2 \times 10} = 2.7N/mm^2, \frac{d'}{d} = \frac{5}{85} = 0.05 \\ A_{st} &= 0.90 \times 30 \times \frac{85}{100} = 22.95cm^2, 3\underset{\phi}{-}32Top = 24.12cm^2 \\ A_{sc} &= 0.18 \times 30 \times \frac{85}{100} = 4.59cm^2, 2\underset{\phi}{-}20Bott. = 6.28cm^2 \end{aligned}$$

But, for reversal of earthquake direction, 3 $\overline{\phi}$32 bars shall be provided both at top and bottom through.

$$\begin{aligned} T_v &= \frac{1.2 \times 34.3 \times 1000}{30 \times 85 \times 10} = 1.61N/mm^2 \\ Forp_t &= \frac{24.12 \times 100}{30 \times 85} = 0.95, T_c = 0.6N/mm^2 \\ V_{us}/d &= (0.161 - 0.06) \times 30 = 3.03kN/cm \end{aligned}$$

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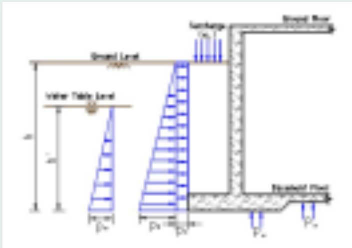

Figure 8.15: Plan of eccentric core in a tall building

Provided by two legged stirrups $\overline{\phi}10/150$ c/c of capacity $= 3.781$ KN/cm, through side face steel $= \frac{1}{100} \times 30 \times 90 = 2.7\ cm^2$/mProvided by 4 $\overline{\phi}10$ bars $= 3.14\ cm^2$/m

8.3.3. Shear Boxes or Cores

Shear cores are very powerful elements, which can be designed to resist the entire horizontal load acting on a tall building. In a 22-storeyed building, a huge shear core, enclosing lifts, toilets and stores was placed initially concen tric in plan. But, later, it was found that it was coming in the way of corridor of a hotel which was housed in its upper six storeys (Fig.8.15). So, the core was placed eccentric, to make way for the continuous hotel corridor on the seventeenth floor and above. The full horizontal shear was applied to the core along with its accompanying torsional moment. The reinforced concrete'walls of the core (230 mm thick) were designed for the tributary vertical load, earthquake shear and the torsional moment in the horizontal plane. The pile foundation was also cleverly designed to take care of these loads, The frames in this building were designed for the vertical loads, together with 25% of the horizontal loads. As an alternative to the above arrangement, Eligator and Nasseta59 have suggested to design the existing frames in the building for the effect of the tor sion in plan, while the core will be designed only for the full horizontal shear (Fig. 8.16). It will relieve

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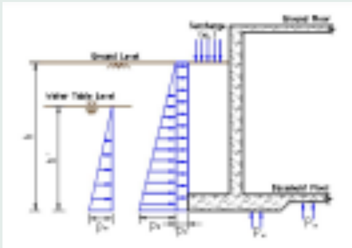

Figure 8.16: Plan of eccentric core in a tall building

the core of the torsional effects. The extra horizontal forces on frames H_1, H_2 , etc. are given by the equations,

$$H_1L_1 + H_2L_2 + H_3L_3 + ...(etc.) = P_y e$$
$$\frac{H_1}{L_1} = \frac{H_2}{L_2} = \frac{H_3}{L_3} = ...(etc.)$$

These equations will give the values of H_1, H_2, etc. and the frames will be designed for the tributary vertical loads, together with the horizontal loads Hi, Hi, etc. or 25% of the total horizontal load, whichever greater. This method will lead to economy in the design of foundations also. The shear cores can be designed as hollow rectangular columns. The curves for column design given in SP: 16[58] can be used with the following changes:

$\dfrac{P_u}{f_{ck}bD}$ will be changed to $\dfrac{P_u}{f_{ck}A_g}$ and
$\dfrac{M_u}{f_{ck}bD^2}$ will be changed to $\dfrac{M_u}{f_{ck}A_gD}$, where,
A_g = concrete area of the hollow core.

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For a rectangular hollow column (Fig. 8.17),

$$
\begin{aligned}
Ag &= (bD - b_1 D1) \\
p &= \frac{A_s}{A_g} \times 100 \\
With, b &= 3.0m, D = 6.0m, b_1 = 2.4m, D_1 = 5.4m, \\
A_g &= (3 \times 6 - 2.4 \times 5.4) = 18 - 12.96 = 5.04m^2 \\
With, P &= 500t, M = 1000tm, f_{ck} = 150Kg/m^2, f_y = 4150Kg/m^2, \\
\frac{P_u}{f_{ck} A_g D} &= \frac{1.2 \times 500 \times 1000}{150 \times 5.04 \times (100)^2} = 0.079 \\
\frac{M_u}{f_{ck} A_g D} &= \frac{1.2 \times 1000 \times 10^5}{150 \times 5.04 \times (100)^4 \times 600} = 0.026 \\
With, d^1/D &= \frac{.15}{6.0} = .025 \approx .05, A_s/2onE.F., p/f_{ck} = 0
\end{aligned}
$$

When the earthquake acts in such a way, that D = 3.0 m and b = 6.0 m,

$$
\begin{aligned}
\frac{P_u}{f_{ck} A_g} &= .079 \\
\frac{M_u}{f_{ck} A_g D} &= \frac{1.2 \times 1000 \times 10^5}{150 \times 5.04 \times 10^4 \times 300} = 0.05 \\
With, d^1/D &= \frac{.15}{3.0} = .05, A_s/2onE.F., \\
p/f_{ck} &= .015 \times 15 = .225 \\
A_s &= .225 \times 5.04 \times \frac{10^4}{100} = 113.4cm^2 \\
16\bar{\phi}32barsgive &= 128.68cm^2
\end{aligned}
$$

The best arrangement of bars will be to put 4 $\bar{\phi}$32 bars at each corner, so that the full steel area is effective in either principal direction (Fig. 8.17). In the rest of the area, minimum wall steel shall be provided as shown for the solid wall element. For openings in shear core walls, same rales as written earlier, apply here too.

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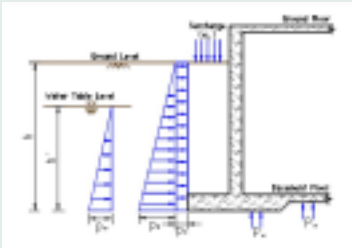

(a) a

(b) b

Figure 8.17: Design of shear core, (a) Plan of hollow core; (b) Steel arrangement of hollow core.

Cambas[60] has suggested use of working stress method for design for shear walls.

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9 Detailing of Reinforced Concrete Members and Structures

9.1. Introduction

Structural analysis of multistoreyed buildings has been given in chapters 1 to 11 and design of reinforced concrete members in chapter 12. The results of the design of members are now to be conveyed to the site for starting the construction work by what are known as structural drawings. These drawings are based on the architectural working drawings. All the grid-line distances, storey heights, lengths of projections are taken from the architectural working drawings. The structural drawings are prepared to give concrete sizes of members and details of the arrangement of steel bars. All foundations details, excavation depth of footings, etc. are also to be covered by structural drawings. In practice, foundation plan gives a complete layout of columns, grid line distances (repeated from the architectural drawings), column sizes, footing sizes, position of plinth beams, basement retaining walls and the basement floor slab, if any. Footing steel is given in footing details or schedule and column steel is shown in column schedule which gives details from the bottom storey to the top, along with the concrete mixes used in column design.

In floor plans, only grid lines are shown and beam and slab panel marks are given. Slab steel is shown in plan with bottom bars shown in dotted lines and top bars in full lines, with thickness of concrete slab also being shown in plan. All lowered panels are also marked in plan. Typical sections are given to further elucidate steel arrangement in slab panels, along with their thicknesses. Floor beam elevations are given in separate

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drawings along with their cross-sections, which indicate beam sizes as well. Stairs are drawn separately giving complete details of all the flights from the ground to the roof along with landing beams and top mumty roof details. Roof slab and beam details are given separately. Generally, typical floor and roof details are given separately. But in tall buildings in earthquake-prone areas, beam design varies from floor to floor. So, in such cases, beam details are given for every two lloors, while the floor plan giving slab design is kept the same as the typical lloor. Roof-level, any how, has to be kept different from the typical floor as at the roof level apart from the slab-loading being different, earthquake effect is small, wall loads on beams are absent and the toilet slab panels need not be lowered. Lift-machine room, mumty roof, water tanks above roof, need separate structural drawings.

Detailing is an important aspect affecting safety of buildings. Failure of a building or its element may be caused by a fault in analysis, design, detailing and construction practices. We are to follow sound detailing methods, so that nothing untoward happens at the site on this account. Clause 25 of the Code gives the requirements of good detailing.

Lap length of bars in tension for concrete mix M15 and steel Fe 415 quality, is given by

$$\begin{aligned} L_d &= \phi \frac{\sigma_s}{4T_{bd}} \\ &= \phi \frac{0.87 \times 415}{4 \times \ 1.0 \times 1.6} = 56\phi \approx 57\phi \end{aligned} \tag{9.1}$$

$$\begin{aligned} where,\ \phi &= \text{diameter of slab} \\ \sigma_s &= \text{stress in bar at the section considered at design load} \\ T_{bd} &= \text{ design bond stress} = 1.0 \times 1.6 \text{ for M l5 and Fe415 deformed bar} \\ L_d &= \text{lap length or development length of bar} \end{aligned}$$

In slabs, beams and footings, steel bars are generally in tension, so lap length or development length for anchoring at ends shall be 57 times the diameter of bar. In columns also, steel has to resist axial compression combinedwith one or two moments, which change in direction during a storey height.So, column bars, too, may come under tension. So for all members, 57 times the diameter of bar is the development length for

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concrete mix M15. For M20 mix, 47ϕ and for M25 mix, 40ϕ lap lengths are used. These latter values will be usefull for column bars when rich mixes like M20 and M25 are used in design

In slabs, beams and footings, steel bars are generally in tension, so lap length or development length for anchoring at ends shall be 57 times the diameter of bar. In columns also, steel has to resist axial compression combined with one or two moments, which change in direction during a storey height. So, column bars, too, may come under tension. So for all members, 57 times the diameter of bar is the development length for concrete mix M15. For M20 mix, 47ϕ and for M25 mix, 40ϕ lap lengths are used. These latter values will be usefull for column bars when rich mixes like M20 and M25 are used in design.

Clear cover to reinforcement bars shall be as follows

Slab : 15 mm or diameter of bar whichever is greater
Beams : 25 mm or diameter of bar whichever is greater
Columns : 40 mm or diameter of bar whichever is greater
Footings : 40 mm or diameter of bar whichever is greater
Presently, 32 mm is the largest diameter of deformed bars available in the market.

Normally, we use 8 mm and 10 mm diameter bars in slabs. It is good practice to use small diameter bars at close spacing in order to control cracking. But it involves more labour in placing more bars. So in countries wherelabour is costly, small number of large diameter bars are used to save on the labour component of cost. In beams, it is a good practice to use large diameter bars in one layer, as much away from the neutral axis as possible. This leads to less congestion and better concreting in moulds. Beam stirrups generally work out to be of 8mm diameter, while 10mm or 12mm diameter stirrups may be used in beams of depth equal to or larger than 900mm. This helps in stiff imformation of steel bars or the top bars are better supported on 10mm diimrici stirrups than on 8mm diameter stirrups which may get buckled during 11ii process of bar placement. In columns too, the practice is to use a few large Inimcter bars in order

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to avoid congestion and achieve good concreting in 11 Kill moulds. Footings are large-sized concrete members and large diameii i burs will be required in all their components. In general,8mm diameter ims ure not provided in footings, as these are buried in the ground and the uiicrcte remains in constant contact with earth.

Minimum steel in members has been specified by the Code as follows:

$$\text{slabs} : 0.12\% \text{of gross area}$$

$$\text{beams} : \frac{A_s}{bd} = \frac{0.85}{f_y} = \frac{0.85}{415} = 0.00205 \tag{9.2}$$

or 0.205% of effective area

$$\begin{aligned} where,\ A_s &= \text{minimum area of tension reinforcement} \\ b &= \text{width of beam} \\ d &= \text{ effective depth of beam} \end{aligned}$$

Side face steel = 0.1% of gross area, where beam depth exceeds 750 I without torsion) or 450 mm (with torsion.)
Minimum shear reinforcement(A_{sv})

$$\frac{A_{sv}}{bS_v} \geq \frac{0.4}{f_y} \tag{9.3}$$

$$\begin{aligned} \text{or min}\frac{V_{as}}{d}(\text{KN/cm}) &= 0.87 f_y \times \frac{A_{sv}}{S_v} = 0.4b \times 0.87 = 0.348b(\text{in mm}) \\ &= 0.348b(\text{in cm }) \end{aligned} \tag{9.4}$$

For wide beams (b > d), B.P. Hughes gives,

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$$\frac{A_{sv}}{dS_v} \geq \frac{0.4}{f_y} \tag{9.5}$$

$$\text{or } \min\frac{V_{as}}{d}(\text{KN/cm}) = 0.348d(\text{in cm }) \tag{9.6}$$

Columns: 0.8% of gross area
Footings and pile caps: 0.12% of gross area as slabs or 0.205% of effective area as beams (to be decided by the designer)

Minimum steel in members is useful to build resistance of members liguinst the effects of temperature and shrinkage which are normally neglected in design.

Maximum steel in members has also been restricted by the code as follows.
Beams: Max. tension steel $>$ 4.0% of gross area of section.
Max. compression steel $\not>$ 4.0% gross area of section.
Columns: 6.0% of the gross area of section.

These limits are never crossed in practice, in order to achieve economy in design.

9.2. Slab Detailing

In one-way and two-way continuous slab panels, bent up bars are used as shown in Fig: 9.1. Generally, midspan steel is made equal to the support steel, so that half the bars from each adjoining panel are bent up to give the support steel. It makes for easy placement of bars and with more than the required steel area at the midspan, it helps in satisfying the deflection requirements. With the bars shown in Fig: 9.1a, with equal spans, all bars are identical. By using bent up bars, we do not need extra chairs to support top bars. In some offices, straight bars are used which need extra steel for providing chairs to support the top mesh of bars.

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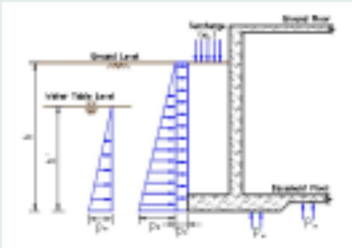
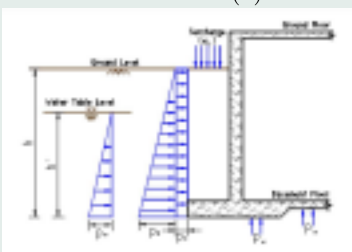


(a)

(b)



(c)

Figure 9.1: Steel arrangement in one -way and two -way continuous slab pnnels (a) Plan of one ways lab panels (b) Section A-A (c) Plan of two way slab panels

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In two-way slab panels,bars are bent at $0.15l_x$ or $0.2l_x$ and $0.257l_x$ in both the directions (Fig: 9.1c). The reason for this practice is found in the shape of the deflected shape of a two-way rectangular slab panel (Fig: 9.2). When a small panel is sandwitched by two bigger panels or a small panel is continuous with a big panel, there is a likelihood of tension taking place at the top of midspan of the small span. In such situations, top and bottom straight bars have to be provided in the small span (Fig:9.3) When a slab panel is small, say 2.0m or less, bent up bars are not used. Instead, straight top and bottom bars are used as main bars. Distribution straight bars are also added top and bottom, to make meshes at the top and the bottom levels.

Cantilever slabs need top steel as main bars which need to be fully anchored ($57 \times$ dia of bar) in the adjoining slab panel or beam (Fig: 9.4). When a cantilever span is large, it is better to introduce cantilever beams and muke the slab to act as one-way slab or thickness of cantilever slab is reduced at free end and bottom steel is also added, in order to take care of reversal of earthquake forces (Fig: 9.5). Overturning of cantilever slabs must be fully countered by designing the supporting beams adequately for torsion and other relevant effects. It may be noted that, cantilever slabs cannot be brought out from a brick wall, but these can be easily and safely brought out from a reinforced concrete wall (Fig: 9.6). When cantilever slabs in two directions form a comer in plan, extra diagonal top bars shall be provided to prevent the corner of the slab from sagging (Fig: 9.7). This may happen as the diagonal cantilever span at the comer is larger than the designed cantilever span.

Small openings in slabs, which do not alter their structural behaviour, lo be provided with two extra bars ($\phi 10$ or $\phi 12$ bars) on all four sides illi anchorage length on either side (Fig: 9.8). However, when an opening ilab is large, it may change the structural behaviour of the slab panel. For ample, if ther is a huge cut-out in a two-way slab at the centre, it will not nave as a two-way slab. Different strips will be designed as one-way slabs pmately (Fig: 9.9), with extra bars around the opening or a large opening is to be surrounded by subsidiary beams (Fig: 9.9), then extra bars in slab around the opening will not be required.

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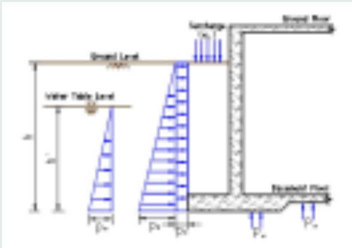
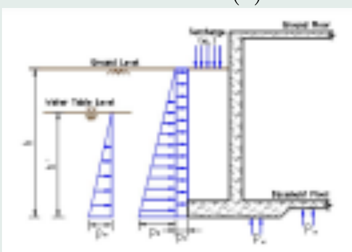


(a)

(b)



(c)

Figure 9.2: Deflected shape of a two -way slab panel, (a) Plan of defected slab panel (b) Section X-X (c) Section Y-Y.

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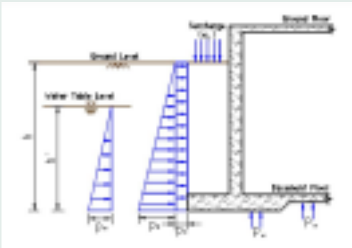

Figure 9.3: Small panel sandwiched by bigger panels.

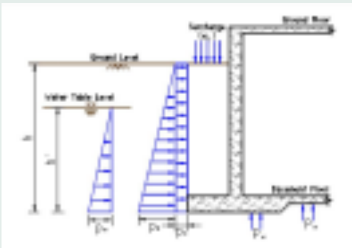

Figure 9.4: Steel arrangement in cantilever slab.

Detailing of stairs involves an important point of note, in that, the bars in tension are not to be bent sharply, as then the concrete cover has a tendency to fall off. (Fig: 9.10b).

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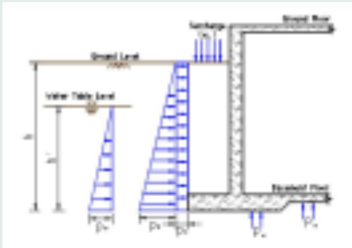

Figure 9.5: Cantilever slab of large span.

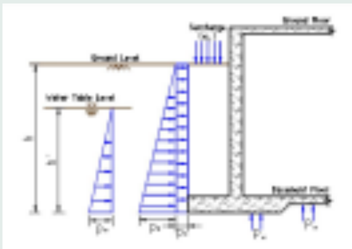

Figure 9.6: Cantilever slab from a RC Wall.

9.3. Beam Detailing

Critical sections in beam design are the mid-span section and the support sections. Elsewhere, the steel bars may be curtailed or stopped. For a simply supported beam, i.e. a beam resting on brick walls or cross beams at either end, half the bars required at the mid-span are taken to the support, where theoretically, no steel is required. The code permits 1/3rd the midspan steel to be taken to the supports. This steel is provided at the bottom only. On top of beam, only nominal 2 Nos. 12 mm diameter bars are provided at the comers of shear stirrups. When the span (L) is long, say, 6.0 m or more,

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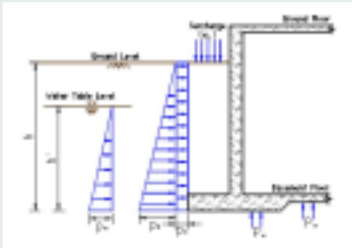

Figure 9.7: Plan of extra top barsat corners.

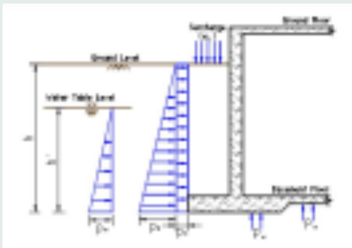

Figure 9.8: Plan of small cut-out in a slab panel

stirrups are losely spaced at L/4 distance near supports and these are more widely spaced the center half of span (Fig: 9.11).

In continuous beams, straight bars are used at all critical sections with 2 bars of the same diameter are provided through, top and bottom. Bent pipes in beams are not being used in practice, being costly in labour. Fig: 9.12 distances for curtailment of bars in continuous beams. In some offices, 2ϕ 12 bars are used as hanger bars over top at mid-lengths of beam spans, while over supports, the required steel is provided. This practice is not appreciated, as there comes a sudden fall in top steel where the support steel stops and hanger bars get lapped. With variations in bending moment diagram, at these sections, there may be some amount of negative moment, which may require

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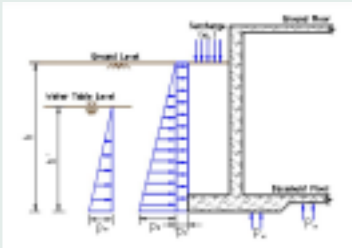
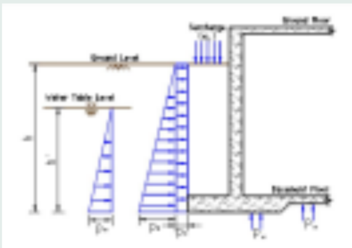
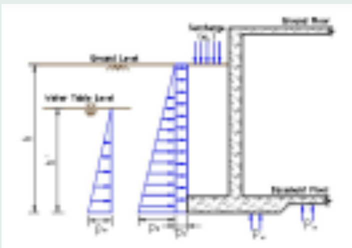


Figure 9.9: Plan of a large open in a two-way slab.



(a) Extra beams around large cut-outs in slab in plan.



(b) Detailing of flight slab.

Figure 9.10

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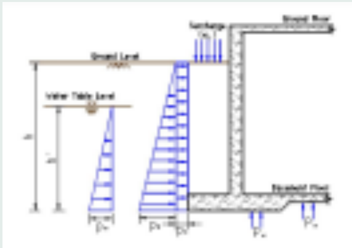

Figure 9.11: Longitudinal of simple supported beam.

more than 2ϕ 12 bars. Hence, the author prefers to use bars of the same diameter in a given beam with 2 nos. going top and bottom through (Fig: 9.13)

When a beam span is under torsion, two-legged closed stirrups have to be provided. Four-legged stirrups are not effective for resisting torsion, as the material is not placed on the periphery of the section. In place of four-legged stirrups, we should use two-legged stirrups placed side by side and these are called double stirrups (Fig: 9.14). Side face steel of 0.1% of gross area is to be provided when beam depth is 750 mm or more. Bent-upbars for shear resistance are now not being used and these bars are of no assistance in resisting torsion.

Ductile detailing of beams for earthquake resistance requires stirrups to be closely spaced at d/4 for a distance of L/4 from either support and in the rest of the span stirrups spacing will be d/2 , where d is the effective depth of beam and L is the span of beam. At the end supports, both the top and the bottom bars shall be bent into column to the extent of development length of bars. Three details of end support are shown in Fig: 9.15a, b, c. These three details are in use in diffrent design offices. Detail of Fig: 9.15a is satisfactory, but it leads to congestion at the beam-column joint. The detail of Fig: 9.15b does not allow the column to be cast at the bottom level of beam, so that a protion of the column is difficult to cast. The detail of Fig: 9.15c is morally used in our office, where it is felt that the longitudinal bars of the cross beam will compensate for

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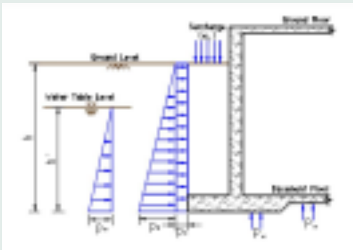

(a)

(b)

Figure 9.12: Longitutnal section of contributed beam (a) Elevation (b) Section X-X

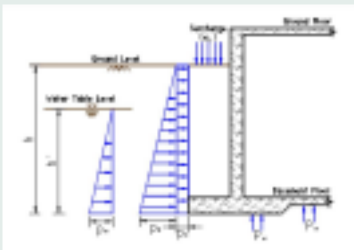

Figure 9.13: Longitudinal section of two span continous beam.

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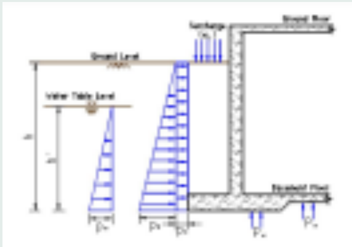

Figure 9.14: 4 -Legged Stirrups and 2-Legged double stirrups,(a) 4 legged stirrups at spacing S (b) 2 -legged double stirrups at spacing S

the insufficient development length of the top and ilu' bottom bars of the beam. This way, the pitfalls of Fig: 9.15a, b are avoided. When the earthquake moment is high at an end support, hair-pinluped bars are used, which are effective on both faces and have no problem inadequate development length (Fig: 9.16) and cause less congestion at in-beam-column joint.

Cantilever beams require careful detailing with top bars adequately an armed in the adjoining span and it may cause tension on top at mid-span of the adjoining span (Fig: 9.17). It is risky to take out a cantilever beam from a column directly, without the aid of an adjoining span. It will affect the column and the footing design. If the cantilever span is small it can be gaurded as a bracket, which needs both the horizontal and the vertical stirrups (Fig: 9.18).

A cantilever beam can be taken out from a brick wall, with an embedment length equal to or more than the span of the cantilever beam. On the em bedded portion, twice the load of the cantilever portion shall be ensured from the adjoining slabs and the top wall, in order to get a factor of safety of 2.0 against overturning. Such cantilever beams shall be checked not only for moment and shear but also for overturning (Fig: 9.19)

When a beam rests at the center of a beam, shear stirrups as required by design shall be

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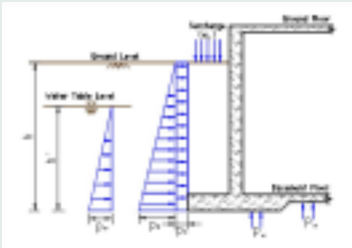
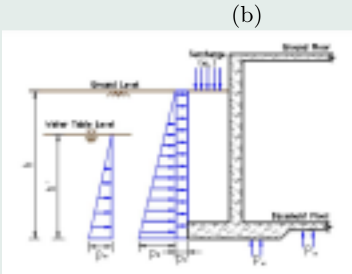


(a)

(b)



(c)

Figure 9.15: End supportde tailing for earthquake moment,(a) Both top and bottom bars are taken up (b) Top bars down and bottom bars up (c)Top bars down and bottom bars up but within the beam deapth(with insufficient l_d

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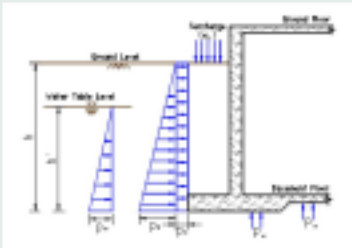

Figure 9.16: Extra U-bars to be provided at anend support where earth quake moment is high

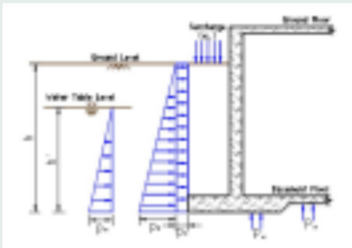

Figure 9.17: Detailing of cantilever(l_c) with top bars anchored in adjoining span

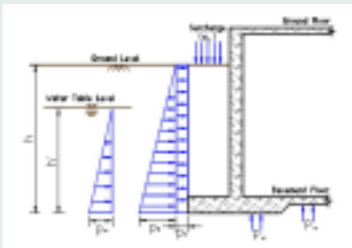

Figure 9.18: Detailing of bracket from a column

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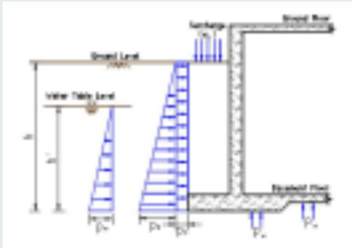
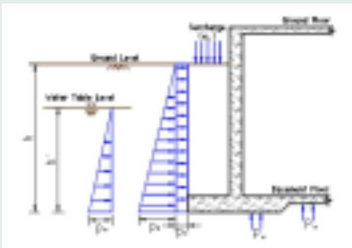
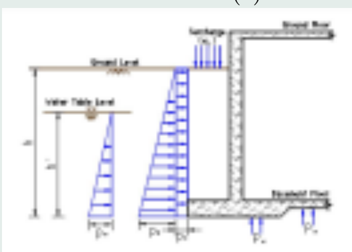


Figure 9.19: Cantiliver beam of brick wall



(a)



(b)

Figure 9.20: Extra long bars to support beam at the lower level; (b) Extra bent-up bars under concentrated load of the supporting beam.

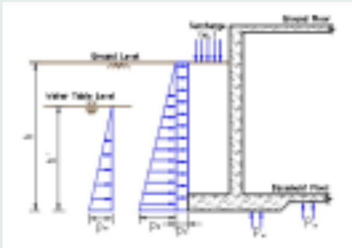


Figure 9.21: Elevation of deap beam

provided at the uniform spacing throughout the span. In rein forced concrete construction, the supporting beam may be deeper or even shallower than the beam supported. They may not be even at the same level, bin the floor slab shall connect both the beams. Extra stirrups or U-bars shallbe provided to resist the tension equal to the beam reaction (Fig: 9.20a). Bent-up bars may be provided to distribute the concentrated load (Fig: 9.20b), this being a good German practice. In deep beams, bottom bars shall be taken through to the supports with out any curtailment and these are to be fully anchored into the supporting columns, as the bottom bars act as tie-steel for the arch action provided by the concrete of the deep beam (Fig: 9.21). Minimum steel in deep beams at 0.205% of the effective area works out more, so the American practice is to provide 4/3rd the calculated area of steel. This practice leads to economy. Vertical stirrups in deep beams shall be 10mm in diameter or even more, for supporting the top bars.

Detailing of beams is given by the following two ways: (i) full beam elevations and cross-sections; (ii) beam schedules with a typical beam elevation and cross-section. The first method is, comparatively, fool-proof and all types of beams can be drawn and explained fully with the required details. While in beam schedules, certain details get omitted and remain unexplained causing difficul ties at the site.

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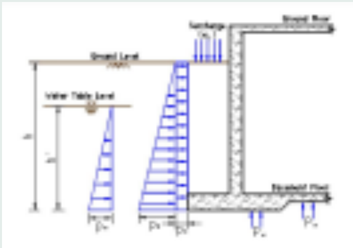


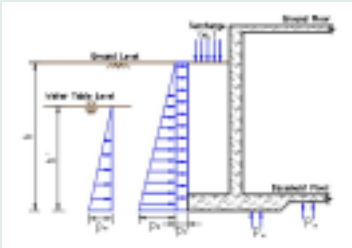
(a)

(b)

Figure 9.22: Steel arrangement in a column section, (a) $A_s/4$ on opposite faces (b)$A_s/4$ on each face (12 bars arrangement shown as an example, 8 to 24 bars can be provided)

9.4. Coloum Detailing

Steel arrangement in a column section depends on the charts used for design. The most serious error in column detailing is that steel arrangement in the drawing may be different from that given by the chart used for design. We have two steel arrangements in column design charts (Fig: 9.22), (i) with $A_s/2$ on opposite faces; (ii) with $A_s/2$ A on each face.



(a)

(b)

Figure 9.23: Steel arrangements in long rectangular coloum,(a) 10 -bar arrangement (b) 12 -bar arrangement

The first arrangement (Fig: 9.22a) is suitable for a column under uniaxial bending, while the other one (Fig: 9.22b) is effective for a column under biaxial bending, for which square or squarish column sections are suit able. For long rectangular column, 10- or 12-bar arrangements as given in Fig: 9.23 are very effective, as all the steel is effective about the width direction which is critical, while 8 bars are effective about the depth direction, which is otherwise a strong direction.

Spacing of longitudinal bars along the periphery of the column section should not exceed 300mm. For this reason, 2 Nos. 12 mm diameter bars may he provided on long

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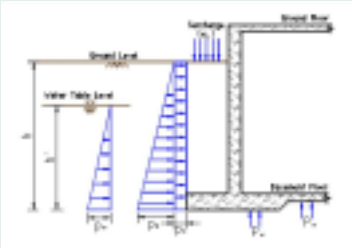
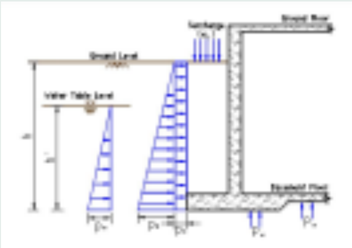


(a)



(b)

Figure 9.24: Stell arrangements in circular coloumns,(a) 8 -bar arrangement with 2-nos. rectangular ties and one circular tie (b) 6 -bar arrangement with 2 -nos. triangular tiesand one circular tie.

faces (Fig: 9.22a). Minimum diameter of a column bar in specified to be 12 mm. In circular columns, minimum numbers of bars shall he six, to be spaced uniformly on the periphery of the column section (Fig: 9.24). The same restriction applies to the ring-shaped columns too.

For column ties, the rale is to hold all column bars at the comers of col umn tics, unless these are spaced at 75 mm or less. In some cases, open links also be provided, these are suitable for wall-type columns. The diameter "I column ties shall be l/4th the diameter

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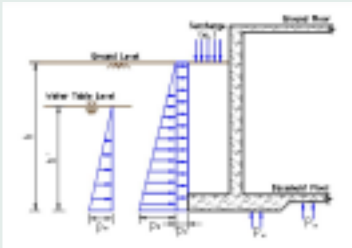

Figure 9.25: Coloum ties in earthquake prone area.

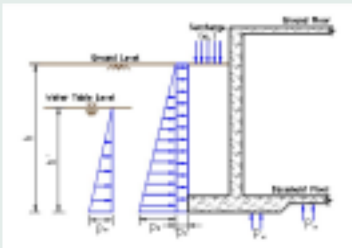

Figure 9.26: Straggered labs for coloum bars.

of column bars. For 25 mm column bars, 6 mm column ties may be provided. In circular columns, one circular tie and other triangular or rectangular ties are provided to hold all column bars in position (Fig: 9.24). For ductility requirements in earthquake-prone areas, 8mm diameter ties at 100mm c/c are to be provided in l/6th the storey height near floor levels and in the mid-height portion, 8mm diameter bars at 200mm c/c are to be provided (Fig: 9.25).

Column steel in multistoreyed buildings is given in the form of column schedule, along with a typical detail for variation in column ties along a storey height (Fig: 9.25). Steel in a column is kept the same in a given storey, laps in column bars consume a lot of steel and these shall be staggered (Fig: 9.26). Column size may also be varied along

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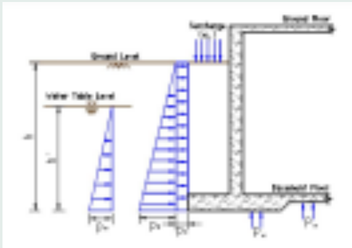

Figure 9.27: Detail of column bars when lower column is bigger and the upper column is small.

the height of building, this is particularly done for internal columns (Fig: 9.27). External column size is kept the. same throughout the height of building, for elevational purposes. If ihe variation in column steel for any two storeys is small or negligible, col umn bars of two storey length are used in order to avoid lapping of bars. Or, if some bars are of the same diameter in the given two storeys, these bars of double length can be used to save on the lapping of these bars. These long bars are to be held in position by extra labour at the site.

Rich concrete mixes M25 or M20 are used in the lower storeys to save on the steel consumption in columns. Concrete mix is also stated in the col umn schedule and it remains the same for all columns in a given storey. This practice aids in good supervision and avoids errors at the site. Likewise, we prefer to have the same grade of steel bars at the site. Different grades of steel will create confusion at the site and it may lead to serious errors.

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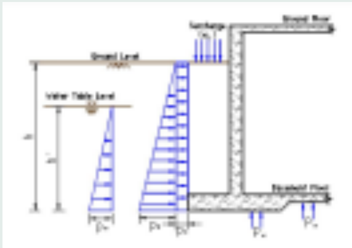
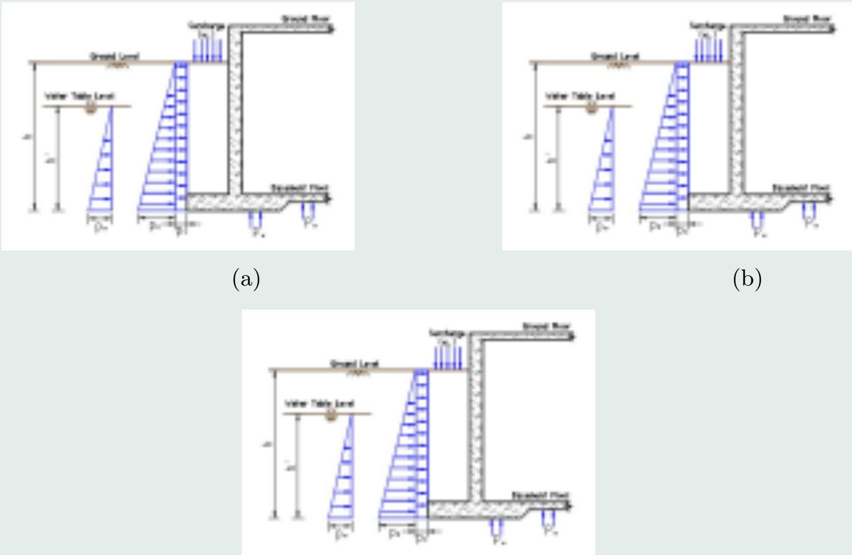


Figure 9.28: Deatiling of an isolated footing

9.5. Footing Detailing

All footings are basically designed as inverted floors, but there are quite a few significant differences in the detailing of footings from that of floor slab and beams. In isolated footings, a bottom mesh of bars is provided as tension reinforcement. The development length of bottom bars in either direction shall be ensured and if need be, the ends of bars shall be taken up, for this reason, particularly in the case of isolated footings of small dimensions in plan (Fig: 9.28). Column bars shall be fully anchored into the footings with the development length of column bars into the footing concrete.

In combined and strip footings, beam detailing is kept different from that of a floor beam , Bar curtailment is done at very safe places, as there can be large variations in column loads from the calculated values (Fig: 9.29), due to various live load dispositions. In slab-type combined footings, extra bars, top and bottom, at the column locations must be given to work as hidden beams (Fig: 9.30). In slab-type raft foundations, top steel is required at the mid-span sections, while bottom steel is the main steel at the support sections, being in accordance with the concept of an inverted floor. Top and bottom meshes of steel bars are provided, with chairs being provided to support the top mesh of steel. No stirrups are provided, in general, in slab-type rafts and the concrete depth is designed to fully resist the shear at the critical sections, with the allowable shear



(a) (b)

(c)

Figure 9.29: Detailing of strip foundation, (a)Elevation (b)Section X-X (c)Section Y-Y

strength (T_c) equal to 0.35 N/mm_2, when p_t <0.25 (Fig: 9.31). Pile caps are given stirrups in both directions, with an additional bottom mesh of steel if necessary (Fig: 9.32).

Plinth beams support the wall load and also neutralize the column base moments due to the vertical and the horizontal loads, together with the axial tension with a load factor of 1.2 and charts given in SP:16,58 with equal steel top and bottom are used for design (Fig: 9.33). Basement retaining walls with or without columns are designed for vertical loads from above and for horizontal loads due to the earth and the ground water retained. Water proofing treatement is also to be given as per the specifications

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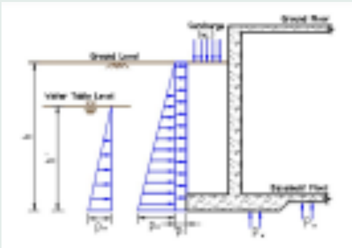
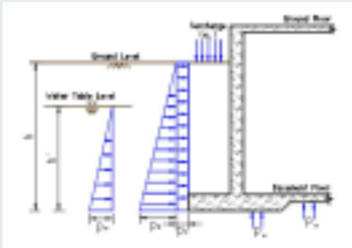
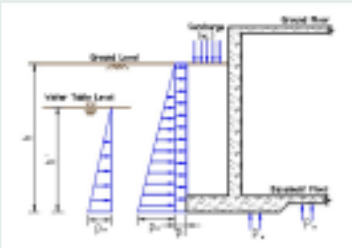


(a)

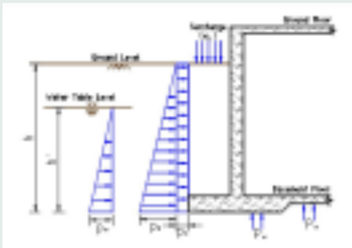


(b)

Figure 9.30: Slab-type combined footing, (a)Plan of footing (b)Section X-X



(a)



(b)

Figure 9.31: Slab-type raft (a)Plan of raft footing (b)Section X-X

(Fig: 9.34).

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9.6. Dealting of Special Chracters

Shallow domes, as roofs o f circular reservoirs are in compression, for which concrete alone is sufficient. A bottom mesh of bars is provided. Near the ring beam, dome slab is thickened and top bars are also added (Fig: 9.35). Ring beam is under tension combined with bending. It is provided with top and bottom circular bars with suitable links.

In folded plates and cylindrical shells, we should provide top and bottom transverse steel (Fig: 9.36). The traverses at ends of spans are to be carefully detailed and the shell slab or the folded plate steel bars are to be fully anchored into the traverses. Traverses keep the form of the shell or of the folded plat intact (Fig: 9.37).

In circular silos and also in circular shafts supporting overhead water banks, we should put steel meshes at each face. We have to check for global for local buckling of the shaft walls (Fig: 9.38).

Detailing of reinforced concrete members and structures SP:34 and Rice and Black.

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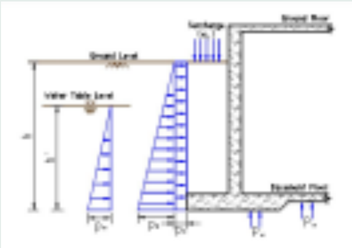
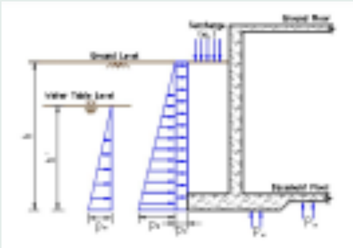


(a)



(b)

Figure 9.32: Pile Cap Detailing, (a)Plan of Pile Cap and Pile Group (b)Section X-X

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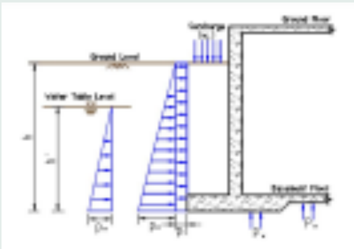
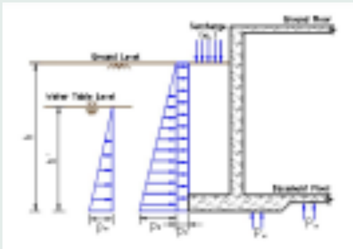
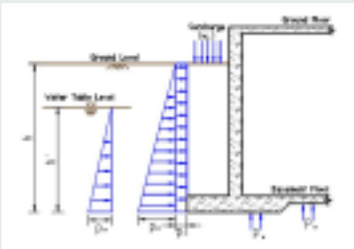


(a)



(b)

Figure 9.33: Detailing of Plinth Beam PB-1, (a)Elevation of Plinth Beam PB-1 (b)Section X-X



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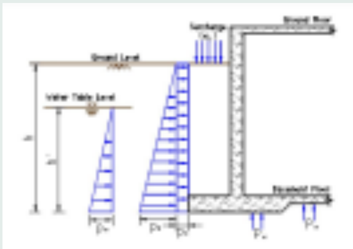

(b)

Figure 9.34: Detailing of basement retaining wall, (a)Basement retainning wall support G.F. slab (b)Basement retaining wall with columns with ventilator at top

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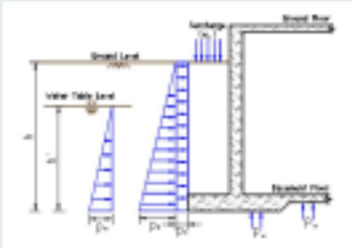

Figure 9.35: Detailing of Domes

(a) (b)

Figure 9.36: (a)Cylindrical shells; (b)Folding plates

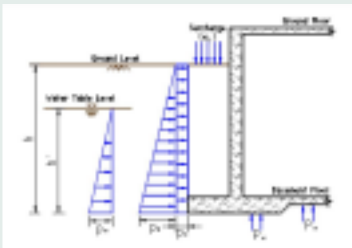

Figure 9.37: Traverse and folded plate detail

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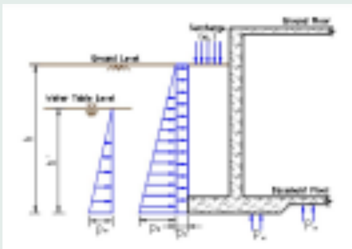

Figure 9.38: Traverse and folded plate detail

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10 Economy in Building Design

10.1. Introduction

Safety, serviceability and economy are the essential requirements of structural design. Although, safety and serviceability are the basic requirements, the rest of an acceptable structural design is economy. Steel is one of the costly materials, which go into the making of a reinforced concrete structure. Therefore, the test of economy is prominently indicated by the amount of steel consumption in a building at the rate of weight of steel per square meter of the covered area of building.

Economy primarily depends on the choice of the structural system for a given building. Then, it depends on the accurate design of critical sections of structural members. In slab design, a concrete mix of M15 and steel type Fe415 lead to economy. Further, slab thickness should be kept as small (between 100mm to 150mm) as practicable. Thicker slabs will lead to an overall expensive design. For beams, concrete mix of M15 will lead to economy. Further, beam depths should be chosen on the high side and beam widths on the low side. More depth for beams will lead to less consumption of steel, while more width for beams will have no effect on steel consumption, it will be helpful in shear resistance only.

For columns, rich concrete mixes like M25 and M20 in the lower storeys of multistoreyed building will lead to economy. Further column size should be chosen on the high side. It will lead to less consumption of steel and slenderness effects are also easily avoided.

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Thin columns add to the cost of building. Further, in earthquake-prone areas, square columns will prove more economical than rectangular columns, as these columns will have to be designed for earthquake effect in each principal direction.

In foundations, we should aim at using more depth of concrete members. This will lead to less consumption of steel. As footings rest directly on the soil below, the self-weight of footings produces no additional moments and shears.

Further, to save on the cost of shuttering, a few standard sizes of beams and columns should be repeated in a building, too many sizes of beams and columns will lead to additional cost.

10.2. Consumption of Materials in Building

For planning the construction of a building, an idea of steel and cement quantities, likely to be required, is to be had at the first instance. This will help in the procurement of these scarce materials.

Cement consumption is of the order of four bags per square meter of the covered area (i.e. 0.2 tonne per m^2 of C.A.). Steel consumption in buildings is given in Table 10.4, which varies from 20Kg/$m^2$to 80Kg/$m^2$of the covered area, depending on the type of the structural system adopted. In Table 10.4 an alternative method of finding steel quantity is also given, which depends on the assumed sizes of structural members. This is a longer method.

Example: Find the expected quantity of cement and steel for a framed building of eight storeys with a total covered area of $10,000m^2$.

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Expected cement consumption $= 0.2 \times 10,000 = 2000$ tonnes.
If it is assumed that this building will take about a year to complete, we may order about 500 tonnes of cement every quarter.

S.No	Structural System	Method 1 (kg/m^3 of concrete)				Method 2	
		slabs	beams	cols	footing	Kg/m^2 covered area	Kg/m^2 covered area with one storey in future
1.	Load bearing brick walls with RC slabs, chajjas, lintels, beams with shallow brick wall footings with small spans	100	150	-	-	20	22.5
2.	Mixed structure with RC colums and load bearing brick walls with shallow footings for walls and colums	100	150	200	80	30	35
3.	RC frames structure to resist vertical loads with brick walls resisting horizontal load by box-action in plan with shallow footings for walls and columns	100	150	200	100	40	50

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4.	RC framed structure to resist both vertical & horizontal loads with brick walls to act as filler walls only with plinth beams & shallow column footings.						
	(i)(ii) N<8 storeys	100	200	250	150	50	60
	(ii) N>8 to 12 storeys	100	200	300	200	60	70
5.	RC framed structure with 50% more earth quake as in hospital buildings, with large spans 7.0 m ×7.0m with basemet walls & slab beam type combined footings (upto 8 storey)	100	250	300	200	70	80
6.	Flat slab or flat plate structure designed for vertical loads with brick walls resisting horizontal loads by box action in plan with shallow footings (upto 4 storeys)	150	200	250	150	70	80

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7.	Flat slab or flat plate structure designed for vertical and horizontal loads with brick filler walls, withshallow footings (upto eight storeys)	200	200	300	200	80	90

Table 10.4: Estimation of steel consumption in buildings

Item	**Diameter of bars** (mm)							**Total**
	8	10	12	16	20	25	32	
Slabs	100	20	5	–	–	–	–	125
Beams	15	5	5	20	30	40	10	125
Columns	10	–	–	20	50	30	15	125
Footings	–	5	10	10	50	50	–	125
Total	125	30	20	50	130	120	25	500
%Ratio of total steel	25	6	4	10	26	24	5	100%

Table 10.5: Diameter-wise distribution of steel quantity (tonnes)

Expected steel consumtpion (Table 10.4 S.No.4) $= 10,000 \times \frac{50}{1000} = 500$ tonnes

Now, for procurement of steel (Fe415, Deformed bars), we need to know the diameter-wise quantities of steel. This is worked out in Table 10.5. It is assumed that 25% of the total steel quantity is consumed in each of the four major elements like slabs, beams, columns and footings. Table 10.5 gives the diameter-wise distribution of the total steel

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quantity of 500 tonnes. This is good enough only for procurement purposes. Initially, one may procure only half the above quantity, i.e. 250 tonnes only. Later on, when the structural drawings are finally prepared, the actual steel quantities are to be calculated out from the drawings and the final instalement of steel to be ordered can be fine-tuned at that stage.

10.3. Cost of Buildings

Cost of a building is a vital quantity which is a function of all the materials, services and labour, that go into its making. It greatly depends on the specifications of materials to be used. For low cost buildings, we have to specify cheaper materials, finishes and fittings. Of course, there will be no compro mise on the structural safety of buildings. In general, the break-up of the cost of a building is given as follows:
The average cost of building, at the present time, works out to be Rs.$400/ft^2$ (Rs.$4000/m^2$)

Structure	= 40% of the total cost
Finishes, doors and windows, partitions	= 30% of the total cost
Electrical	= 20% of the total cost
Plumbing	= 10% of the total cost
Total	= 100%

of the covered area. If the specifications are made on the cheaper side, the cost may go down to Rs. $300/ft^2$ o f C.A. and if the specifications are rich, it can go upto Rs.$600/ft^2$ or even more.

11 Construction and Site Supervision

11.1. Introduction

Construction of a building is done by a contractor who is to be selected from the result of a bidding competition, in which a number of contractors participate. The contracts are, in general, of two types: i) item rate contract, ii) lump sum contract.

In an item rate contract, rates of various items involved in a building are quoted by contractors. On award of work, the rates quoted are binding on both the parties (the client and the contractor) while the quantities may vary. In this type of contract, the drawings may. be revised during the progress of work. The final cost of the building will be known at the end of contraction.In a lump sum contract, the drawings of the building are made complete and the quantities are worked out accurately. The contractors are asked to quote their rates of items and also the total cost of building. On award of work to a contractor, the cost of building is finally fixed. If any change is thought to be necessary, it is to be considered separately and it will affect the final cost only lightly. Extra items my crop up in either system of contract. Rate analysis on ihe basis of market rates is to be given for each extra item and the extra cost of iliese items has to be paid to the contractor.

The contractor is required to execute the work in accordance with the drawings supplied to him. For supervision of contractors work, the architect appoints a clerk of works, who is paid by the client but he remains under the technical guidance of the architect.

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The clerk of works has to ensure the quality and speed of the work at the site and he is also responsible for coordinaimn of the drawings supplied by architect and also by structural, plumbing, electrical and air conditioning engineers. He is also co-responsible for measurements of the works executed at the site and preparation of contractors bills of payment, which are checked and approved by the architect for payment by the client.

The structural engineer of the building under construction is required to visit the site periodically and help the clerk of works in the supervision of the work.

11.2. Periodic Site Supervision By Structual Engineer

The layout of the building is checked by the concerned architect. The structural engineer first visits the site, when the footing trenches are fully excavated, but lean concrete under footings is not yet laid. The purpose of this visit is to assess the quality of the soil and it is also to ensure that no footing rests qn a filled up soil. If in any trench, one comes across ashes, potteries etc., it indicates a filled up soil and then one must increase the depth of foundation in the trench, till a firm natural soil is reached. Further, each trench should be carefully inspected to find out any soft pockets of soil, which shall be fully scooped out and replaced by lean concrete. The soil investigation report is normally based on a few bore holes in a huge area and its results should be got confirmed in all footing trenches by visual inspection. This is a very vital matter for the safety of building, so, a senior structural engineer should visit the site for the purpose of soil examination

The other visits of structural engineer to the site coincide with the checking of steel provided in structural elements like footings and retaining walls, columns, floor slab and beams at all floor levels, roof slab and beams and for structures above the roof level. The following is the check-list of the items to be checked by the structural engineer, during his visits to the site.

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The steps involved in the proposed method of rigidities are given and explained below :

(1) Checking quality of sand, stone aggregates, cement, steel bars, water, bricks, etc.

(2) Checking soil in the footing trenches.

(3) Checking steel bar diameter and spacing (or number of bars) in various structural memebrs like footings, columns, beams, slabs, retaining walls etc. in accordance with the structural drawings.

(4) Checking clear covers over steel bars in members. Clear cover is overmain bars, but in practice it is wrongly taken over stirrups.

(5) Checking laps of steel bars in members.

(6) Checking provision of chairs and stone or concrete gitties for positioning of top and bottom meshes of steel bars.

(7) Checking anchorage lengths of bars in members, particularly at the following locations: i) column bars in footings; ii) beam bars in columns; iii) slab bars in beams; iv) shell slab or folded plate bars into traverses, etc.

(8) Checking the level of the floor shuttering and the strength and the spacing of vertical props and their bracings. This aspect is responsible for many accidents at sites.

(9) Checking distances of curtailment of bars in slabs and beams.

(10) Checking anchorage length of top bars in cantilever members and ensuring their safety against overturing, when the shuttering is removed.

(11) Checking the vertical depth of bent-up bars in slabs. It is invariabley short of depth in practice.

(12) Checking overall concrete sizes of members, like slabs, beams, columns, footings, etc.

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(13) Checking record of concrete cubes taken at the site and tested by an approved testing agency in order to ensure strength of the concrete mix used inconcrete members.

(14) Noting the remarks of his visit in the site order book for record and for future action to be taken by the contractor.

11.3. Special Site Problems

An interesting problem at the site is the production of concrete of adequate strength as prescribed in the drawings. Concrete is a material which is to be manufactured or prepared at the site, unlike say, steel which is a finished and tested product. Ingredients of concrete, cement, sand, stone aggregates and water are to be mixed in the given ratios, either by volume or by weight and are mechanically mixed thoroughly and then the material is ready to be placed in moulds. The quality of materials, the nature of mixing, the proportions of the mix and later the way concrete is placed (and also the time taken in placing), vibrated and finally cured, all these items affect the final strength of concrete. As a check on the strength of concrete mix, we take out a number of cubes from each mix and keep these cubes in a water tank for curing and then cmsh them at 7 days and also at 28 days. The 7-day strength of concrete is about 2/3rds concrete strength at 28 days. It serves to give an early warning or information on this vital structural material to all concerned. Knowing the way a cube is filled and vibrated carefully and cured so well, the Code makes a difference in the cube strength of concrete and the design strength of concrete actually available in a structure. Only 0.67 times the cube strength of the concrete is considered in design. This has been done to account for vagaries and variations at the site. Further, the material safety factor for concrete is laken at 1.5, while the same for steel is only 1.15. This high value for concrete also considers the imponderables in the production of concrete at site. In spite of all these precautions taken in the design of structures, we may face several problems at site on this account. The cube strength values may fall short of requirement, the concrete member may have

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cracks after the shuttering is removed or slab panels may deflect or vibrate excessively. These problems call for a visit to the site and the defective concrete may have to be broken and removed and to be redone after ascertaining the cause. Sometimes, use of defective cement may be the cause of poor strength of concrete and of cracking and excessive vibrations of slab panels. In such situations, the slab concrete is to be removed, with steel bars kept in position and the slab panel is re-concreted using good, fresh cement. 10% shortfall in the cube strength may be allowed, as concrete gains about 20% in strength in about a years time. Often, load tests in accordance with the code are to be conducted on the affected members for acceptance of an already laid piece of concrete.

Foundations also pose, sometimes, difficult problems at site. Sometimes, old wells or cavities may be found in footing trenches. If the cavities are small, these may be filled up with lean concrete, but well-like cavities will be costly to fill up in this way. A well may be regarded as a cut-out in the footing area, so that, footing area has to be accordingly increased and the footing slab has to be designed for local bending due to the well opening. Sometimes, some buried structures like septic tanks may be encountered, which may necessitate redesign of footings. These problems are best studied at the site and a solution in keeping with the suggestions of the site staff is to be worked out. Sometimes, two isolated footings in the drawing may be interfering with each other at the site. This may happen due to some error in the drawing or lack of information on vital data or distances from the site. So, a new solution like combined footing, which will not create further problems of collision with other footings at the site, has to be suggested.

Often, alterations or extensions from the already cast or existing members are required to be done at site. For these reasons too, a structural engineer has to visit the site and discuss with the site engineers the problems at hand and suggest solutions which are practical and convenient at the site. During or after the construction of a building, cracks may appear in certain members. This can be a serious affair, so the design engineer then is required to inspect the building and study and analyse the crack formations and suggest remedies for their elemination.

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A very useful material for study on this subject is given by Camallerie.

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12 Structural Failures or Forensic Engineering

12.1. Introduction

In spite of the best attention paid to the analysis, design, detailing and construction of buildings, it is a sad fact of life that failures - small or big - do occur in practice. These failures often get reported in the daily newspapers and shatter the confidence of public in the profession of Structural Engineering. Failures of members or structures may take place due to errors in the analysis, design, detailing, construction and maintenance of buildings. One can learn a lot from failures of structures. When a member fails (say, cracks), it is trying to tell the structural engineer something about its behaviour, about its deficiency on several counts. The structural engineer is required to understand the language of the structure and analyse the cause of the deficiency and then suggest a suitable remedy so that the member is made good and strong again. This aspect is similar to the work of medical doctors and so the phrase t'orensic Engineering has come into vogue in recent years. The structural engineer may, then, be called as 'doctor of buildings '.

The author has come across many failures of structures and in this chapter, some of these failures will be described and analysed in detail, for the benefit of readers.

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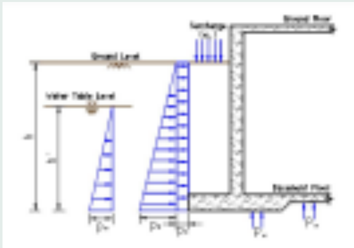

Figure 12.1: Limestone storage underground structure

12.2. Analysis and Design Errors

(1) In a fertilizer project (in the late fifties), a lime storage underground building was shown in the scope drawings (Fig 12.1) to support a huge mound of limestone, for which, the equivalent design uniformly distributed load was given in Kg/m^2. On conversion to ft pounds system, currently in use in those days, the value of load was wrongly taken at 300 lb/ft^2(i.e. 1500 Kg/m^2), while the actual loading worked out to be 3000 lb/ft^2(i.e. 15000 Kg/m^2). The designer thought 300 $lb/ft^2$to be quite a high value and the structure was designed and the structural drawings were prepared and checked and these along with a copy of the calculations were sent to the site. At the site, the footing burs and wall steel were in position. At that stage, the design firms representative at the site spotted the error in the loading and the work was stopped at the site. The design was corrected, keeping the concrete sizes the same as before, so that shuttering work at the site remained unchanged. Steel requirement was changed. Additional steel bars were provided along with the previous steel arrangement. The structural drawings were accordingly revised and the work was completed at the site. The design error was spotted at the site in the nick of time and a major catastrophe was thus avoided.

(2) In a cinema building, a parapet of large depth, above the roof level, was proposed by the architect. The structural engineer in his wisdom, thought of providing a

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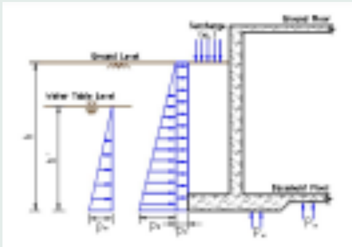

Figure 12.2: Channel shape of parapet over roof of a cinema building

(a) a (b) b

Figure 12.3: Northlight folded plate failure, (a) Elevation of traverse; (b) Section X-X

channel section, in order to save on the weight of the materials and thereby achieve economy (Fig12.2). After a few days of the construction, the channel section, with its shear centre outside the section, was seen to he rotating outside and large cracks were formed in the concrete parapet. Had it been a rectangular section (b ×d), this rotational effect would not have taken place. The channel shaped concrete parapet was dismantled and in its place, a rectangular parapet was built, which is standing since then without any problem.

(3) A five-plate north-light folded plate over a workshop was built and after Mimetime, the glass panes in the glazing were reported to be breaking on and oil The structure

was propped from below and the site was visited and the drawings were studied. The problem was in the traverse design and its arrangement. The lower part of the traverse was made of reinforced concrete and its upper part was made in brick (Fig 12.3). The entire traverse should have been made in reinforced concrete, so that, the folded plate is held in shape at all points along its cross-section. Because of the brick work, the upper portion o f the folded plate was opening out, pressing on the glazing and breaking its glass panes thereby giving ample warning to those who would listen and understand!

The structure could have been repaired by removing the brickwork and making the full traverse in the reinforced concrete. But, an alternative steel solution was adopted at the site, whereby, the folded plate is acting as just the covering slab resting on steel trusses, which were introduced to support the existing folded plate.

(4) In a single-storey auditorium building, cracks developed in the supporting brick piers above the roof. The long-span roof consisted of a waffle slab resting on brick piers all around the periphery. The deflected shape of the roof structure pushes up the parapet above the roof, causing cracks in piers on the outside above the roof level (Fig 12.4). This sort of crack formation cannot be avoided. Rather, a groove can be left at this level to accommodate the separation o f the parapet masonry from that of the supporting brick pier. The deflection of a bending member is a natural phenomenon and this sort of cracking is not structurally harmful.

(5) In a single-storey house of a military doctor, the canopy slab, at the level of the main roof, was found to be sagging (Fig 12.5). The water proofing over the roof was already laid. On enquiry, it was found that no proper structural engineer was engaged for design, rather a novice had done the design of the house. It was like a patient treated and badly spoiled by a quack was brought to a doctor. The client, a military doctor himself, appreciated the sitution he was in. We introduced two steel I-Joists as shown in the plan (Fig 12.5) under the slab, supported on the existing brick walls and these stabilized the cantilever canopy slab. The client had to spend much more than at heImd saved by appointing a novice.

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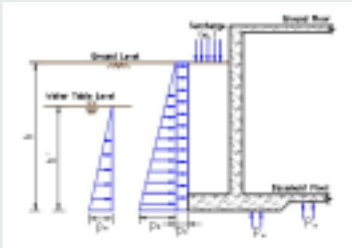

Figure 12.4: Cracks in supporting brick piers of an auditorium roof

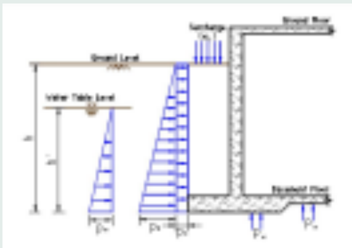

Figure 12.5: Canopy slab at the roof level of a house

(6) In a multistorey building, there was a floor of 40'-0" span. It consisted of closely spaced ribs of 40'-0" span with the topping slab (Fig 12.6). The depth of the ribs was kept at the minimum required of $l/20 = (40 \times 12)/20 = 24$". On this floor, a few residential units were built by means of 115 thick brick walls. When the building was occupied, after sometime, there were many cracks in the 115 thick brick walls, while the supporting slab and ribs were absolutely crack-free. The cracking in brick walls is caused by the deflection of the i ibbed one-way slab. At the centre of span, the deflection is more, while near the supports it is less. Further, it is less on the front and more on the back miles. Due to this variation in deflections, brick walls get cracked, as these are brittle and cannot absorb these

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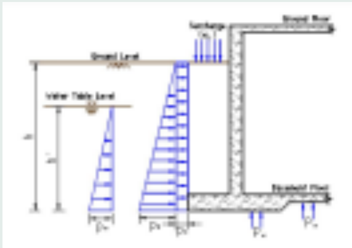

Figure 12.6: Canopy slab at the roof level of a house

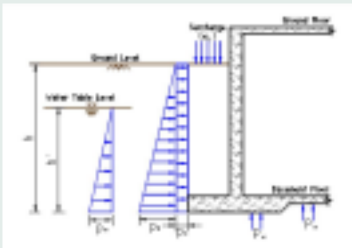

Figure 12.7: Faliure of Gabion retaining wall

deflections. After a certain period of time, these cracks were repaired with a hope that most of the deflection of slab had already taken place.

(7) A gabion retaining wall was used in the spillway portion of an earth check in a foreign country. A gabion is a wire-net box filled up with stone aggregate. It is used as revetment on earth dam slopes. Hut to use them as structure retaining wall like brick masonry or reinforced concrete is conceptually wrong. But it was so used in a project, where the spillway portion failed during a flood. It was recommended to build again spillway portion in concrete block masonry (Fig 12.7).

(8) A workshop building with a flat roof failed suddenly. The roof consisted of red sand stone, slabs, resting on J_-steel sections, which were resting on I-joists. The

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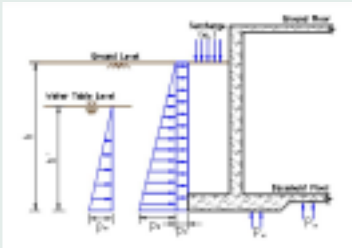

Figure 12.8: Section of a flat roof on a workshop building

I-joists were resting on RC columns (Fig 12.8) . On the stone slabs, will be put, earth laid to slope for roof drainage. When the earth was being laid on top, the roof suddenly collapsed. Luckily there was no loss of life. On detailed analysis, it was found that I-joists failed by lateral bucking as 1-sections were only resting on I-joists, these should have been welded to the I-joists. The twisted shape of the I-joists of the failed roof also testified to correctness of this finding.

12.3. Detailing Errors

(1) In a hostel building, there is a continuous cantilever balcony along the moms which also turns across at the end of rooms (Fig 12.9). When the shuttering of the balcony slab was removed, it was found that the corner(A) on the diagonal was sagging. This was due to the fact that no extra diagonal top bars were provided along the diagonal. The sagging portion of the slab at the corner was sawed off and the shape of the balcony in plant was changed on this account.

(2) The columns o f a cantilever platform shed were developing horizontal cracks at a fixed height above the ground floor (Fig 12.10). When the shuttering was removed, these cracks were appearing slowly in all columns at a fixed height. Immediately,

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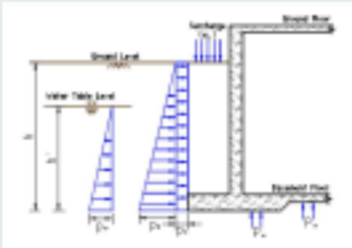

Figure 12.9: Sagging of a diagonal corner of a balcony slab

the frames were propped at free end of long arms and study of the design and drawings was made. It was seen that laps in column were provided at a distance of 35 ×DIA of bars as it is done normally for bars in compression. But in this case, the cantilever moment is very high and the bars on this side were clearly in tension, so a dowel length of 45 ×DIA of bar (as per the old code)3 was required. It was suggested to expose the bars near about the crack line and weld the dowel bars with the column bars and concrete this portion. After the curing period, the props on the free ends of long cantilever arms were removed and the cracks, mercifully, did not appear again. The correct diagnosis, in this case, led to the correct remedial measures.

(3) In a canteen building, a vertical crack at midspan was observed in a deep beam over the service counter (Fig12.11). The drawing of the beam showed no side face steel. Two bars on each face of beam were welded to the stirrups shown in Fig 12.11. The stirrups at those locatibns were exposed, bars welded and then reconcreted. The cracked portion was repaired. Provision of side steel in deep beams was not mandatory in the previous code,3 as it is now in IS: 456-1978.6.

(4) In a workshop building, cylindrical shell roof was provided with steel arrangement as shown in Fig: 12.12 and it is taken from Jain and Jaikrishna Vol.II.5 After completion of the building, extensive thin cracks were observed in lie shell roof from the inside. After a thorough study of the analysis and design of the shell

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(a) a (b) b

Figure 12.10: Cantilever platform shed columns, (a) Frame; (b)Column end view A

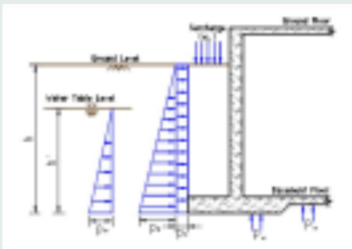

Figure 12.11: Vertical crack in a deep beam

roof, it was concluded that transverse steel should have been put on both faces of the shell thickness (Fig: 12.13) all through.

(5) A serious failure took place due to a detailing error in the case of a ramp slab, which was cantilevered out from a central wall shaft (Fig 12.14). The lamp was separated from the adjoining floor by an expansion joint. The span of the cantilever span was 2.35m and section 1-1 (Fig 12.14) gives the steel arrangement as given in the relevant structural drawing. The embedment length of the bottom 10 bars was given 300mm. As the word typical was added in the drawing, the embedment length of top 16 bars was also provided 300mm at the site , instead of the required

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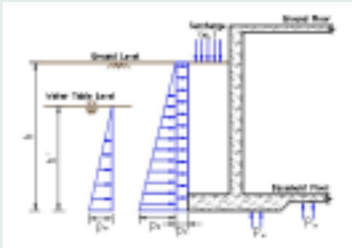

Figure 12.12: Transverse steel in cylindrical shell roof (incorrect detail)

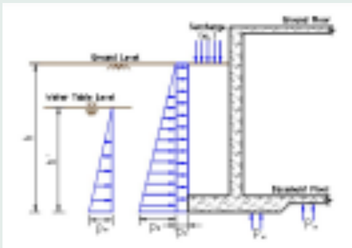

Figure 12.13: Transverse steel in cylindrical shell roof (correct detail)

700 mm. Because of this detailing error, when the shuttering was removed, slowly a crack was forming at the junction of the slab with the wall. Immediately, props were provided along the free edge of the ramp slab and a steel beam with steel columns were permanently provided to support the free edge of the ramp slab, so that the final behaviour of the ramp slab was simply supported with a span of 2.35m, for which 10/200 C/C bars at bottom were adequate. The lesson from this failure is that, in case of cantilever slabs and beams, the main bars should be anchored and their embedment lengths should be separately and clearly mentioned in the drawings.

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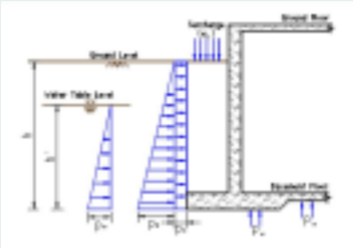

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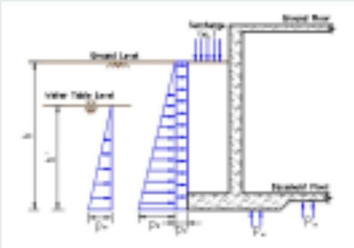

(b) b

Figure 12.14: Plan of ramp in a hospital, (a) Plan; (b) Section 1-1

12.4. Construction and Maintainence errors

(1) A petrol pump station structure was a hypar shell structure supported on a single column (Fig 12.15). This structure was built in accordance with a stadard drawing and cracks had developed on roof top as shown in plan of (Fig 12.15). The structure was propped at free edges. On visiting the site, it was felt that the contractor had not been able to understand the supplied standard drawing. He had put more steel on the free edge and less on the column side, unlike shown correctly in the drawing. It was suggested to the contractor to domolish the hypar shell and redo this work

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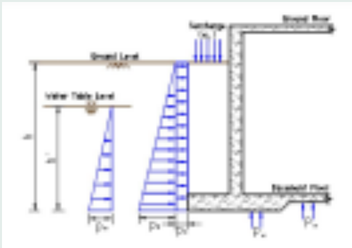

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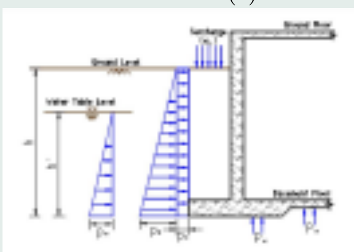

(b) b

Figure 12.15: Petrol pump station structure, (a) Top plan of the hypar shell structure; (b) Section X-X

again. However, before he could do it, the structure collapsed one night, due to vibrations of a passing truck. Luckily, there was no loss life. This failure may be ascribed to the in competence of the contractor and also to the lack of supervision at the site.

(2) A single storey bungalow was under occupation. The neighbouring plot was empty and its owner started digging foundations for his building deeper than the foundations of the bungalow. The footing trenches were kept dug for long and during this period, it rained too, with the result that the end wall of the bungalow rotated and there were large cracks in the ground floor also (Fig 12.16). For protecting the

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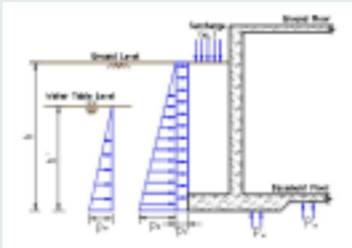

Figure 12.16: Failure of an wall of an existing bungalow

existing property, one should not dig near the existing wall. One may keep about 3' 0" distance from the existing wall and then the rest of the plot can be dug out as required. Later on, taking due precautions, the excavation near the existing wall should be done in parts and quickly the work is to be completed, so that the existing property is not damaged.

(3) A retaining wall, which was supporting a high ground with a swimming pool, was observed to be tilting outwards (Fig 12.17). On visiting the site, it was found that water was leaking from the swimming pool and it was exerting extra pressure on the retaining wall. The remedy suggested was to drill holes in the retaining wall and insert pipes to act as weep holes, so that water pressure on the wall can be relieved. This added to the safety of the wall.

(4) An oil tank roof failed. The roof consisted of radial and circumferential trusses with a top steel plate. The radial trusses were supported on the steel tank walls. The client wanted cement plaster to be done on the roof. That to be excessive load and the joint of the radial truss with the verticall steel wall failed (Fig 12.18). The roof was redone without the cement plaster.

(5) A well-planned bungalow was designed by an architect and its structural drawings were prepared. When the work started at the site, neither the architect nor the structural engineer were employed by the client. The contractor, in his wisdom, thought beams were not required, so he provided two bars top and bottom at

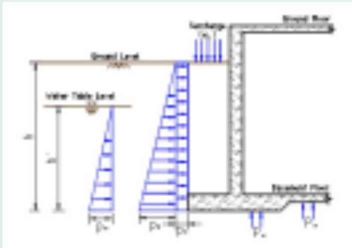

Figure 12.17: Retaining wall failure

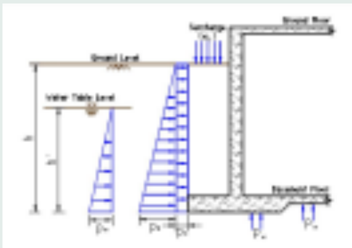

Figure 12.18: Joint of steel tank wall with radial truss

all beam locations. The slab was cast and when the shuttering removed, cracks started at many locations. On visiting the site, it was seen that slab panels were large and cantilever balcony slabs were also of large spans. On all critical sections, the cracks were bound to occur due to the omission of all beams. So, those beams were again provided by suitably in breaking the concrete and welding of bars with the old ones, then reconcreting and curing these new concrete patches in slabs and beams. This work was similar to the plastering done by an orthopaedic surgeon when he sets right broken bones. The client had to spend a good amount, but the structure was set right.

(6) A reinforced concrete framed factory was being extended upwards. The beams

developed cracks as these were not propped from below, when the up per storey was being extended. The load of shuttering, the weight of the green concrete and the erection loads exceeded the load capacity of the existing beams. Immediately, the beams were adequately propped. This failure is paused by laxity on the part of construction staff.

(7) In a running steel plant, the roof of a shed collapsed due to overloading caused by dust collection on top. Dust contained heavy iron particles too. This is a maintenance failure.

12.5. Some Well Known Structural Failures

The following structural failures were reported in newspapers or other technical literature.

(1) A barrel shell roof failed in Calcutta. The traverse was o f the type of a bow-string girder. The tie steel was not adequately anchored at the ends. This was the cause of the failure.

(2) The hanging corridors over a dance hall at ground floor in a hotel in USA fell down on the dancing crowd, killing many persons. The hanger bars failed due to a faulty connection detail.

(3) A foreign airport building collapsed killing about 300 persons, The old airport was being extended. The new extension was resting on the old structure, which brought it down.

(4) Ronan-Point (UK) failure o f apartments built of precast concrete elements was due to an explosion of a gas cylinder in a kitchen.

(5) Silos tilted in Transcona (Canada) due to soil failure.

(6) Apartment buildings failures were reported in Cairo and recently in Calcutta too, due to additions of one or more storeys, for which the buildings were not earlier designed.

(7) Three-storeyed childrens ward in a Jammu hospital collapsed resulting in casualties. Reason? Inadequate amounts of cement and steel used.

(8) Free-standing brick walls often fail killing labourers resting in their shade. Brick walls, with load (slab) on top are safe. Long brick walls (unloaded) free at the top are quite dangerous!

(9) Cantilever balconies often fail killing people due to overloading, wrong design or detailing or both.

An excellent book on this subject is by Raikar,[9] 8 in which, a wealth of detail on this subject is available.

12.6. Additional Case Studies of Failures

The following case studies o f some failure have been contributed by J.D. Buch:

(1) On a hill area in Bombay, a multistorey apartment building was constructed in seventies. The plateau on a hill, on which this building is constructed is about 12m above the surrounding area. Foundations were provided after soil investigation and there was no problem for the main building. The owner wanted to construct a single storey garage block. The structural engineer decided to support one end of the garage block on the existing old 12m high stone masonry wall (Fig 12.19). During rains, the entire garage block toppled and rested on a building 12m below on the low ground. This failure was due to the failure of the old retaining wall during rains.

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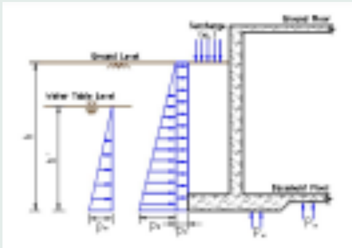

Figure 12.19: Failure of garbage block in a hill area in Bombay

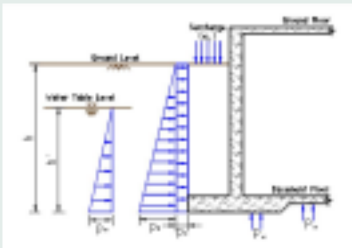

Figure 12.20: A section through a single-storey factory building

(2) A single-storey factory building with RCC roof (Fig 12.20) was constructed near Delhi in seventies. The 12.0m span roof was earlier cast. Later 15.0m Span was cast. When the shuttering was removed, 15.0m span collapsed. It was noticed that concrete in the bottom layer of 15.0m span beam was honey-combed and it did not offer any bond to the reinforcement and the structure collapsed. As the collapse started, top 2.0m height of the intermediate column was pulled in (towards 15.0m span) and failed in bending due to the pull o f the collapsing beam. It is recommended that for beams with spans greater than 6.0 m, lapped bars should atleast be tack-welded to avoided this type of failure.

(3) 30.0m span storage hall (or stock yard) building used to be provided in the earlier

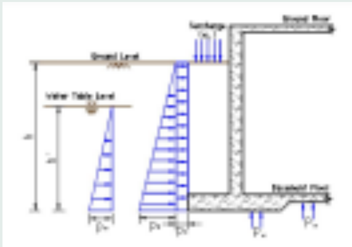

Figure 12.21: A section through the stock yard building

plants with a provision of lO.Otonne lifting capacity grab crane (Fig 12.21). To feed materials to the various hoppers, the crane operated continuously for 24 hours. In many plants, there was excessive deflection of columns during crane operations and the crane used to get derailed. In one plant, due to excessive deflection of columns, the roof collapsed. After a study of the various failures in similar plants, the following conclusions were drawn: (i) Crane horizontal surge for high fre quency operated grab cranes should be 10% of the maximum static wheel load per wheel. The provision in the prevailing IS code was in adequate. (ii) After studying such failures, the plant suppliers specified relative deflection between adjoining columns in longitudinal direction to be limited to 2.0 cm. This should, however, be worked out on the basis of limiting clearance between the rail and the wheel (Fig 12.22). A conclusion from the above is that design, will be governed by the deflection limitation for serviceability of crane and not by stress considerations.

For detailed paper on this subject, reference may be made to “Design aspects of material stockyard buildings” by S. Majumdar and J.D. Buch - Seminar on Engineering Design and Plant Development of Cement Plants, 23-25 Nov. 1974, Cement Research Institute, Theme E, pp. 1 to 19.

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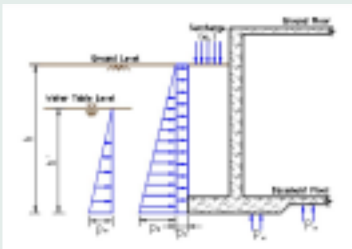

Figure 12.22: Limiting clearance between rail and the wheel

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13 Procedure for Analysis and Design of Buildings (Steps in Design)

13.1. Introduction

The procedure for analysis and design o f a given building will depend on the type o f building, its complexity, the number o f storeys, etc. First, the architec tural drawings o f the building are studied, structural system is finalized, sizes of structural members are decided and brought to the knowledge of the concerned architect. The procedure for structural design will involve some steps which will depend on the type of building and also its complexity and the time available for structural design. Often, the work is required to start soon, so the steps in design are to be arranged in such a way that foundation draw ings can be taken up in hand within a reasonable period of time.

Further, before starting the structural design, the following information or data are required: (i) A set of architectural drawings; (ii) Soil Investigation Report (SIR) or soil data in lieu thereof; (iii) Location of the place or city in order to decide on wind and seismic loadings; (iv) Data for lifts, water tank capacities on top, special roof features or loadings like cooling towers, etc. if any.

The structural system is finalized and columns, floor beams and slab panels are all marked and numbered on the architectural, plans. Now, the building is ready for structural design to start.

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13.2. Design Office Practice For A Tall Office Or Apartment Building

The following procedure or steps in design may be followed for a multisloreyed office or apartment building:

- `Step 1:` *General Loading.* For each floor or roof, the loading intensity of slab is calculated taking into account the dead load of the slab, finish, plaster, etc. including partitions and the live load expected on the floor, depending on the usage o f the floor or roof. The linear loading of beams, columns, walls, parapets, etc. are also calculated.

- `Step 2:` *Computation Of Column Loads.* Floor dead and live loads on each column are calculated separately, taking its tributary area into account. Then the total column load is calculated taking the reduction in live load into account as peritted by IS.875. 5% extra load is generally considered for unforeseen items like elastic shears etc.

- `Step 3:` *Analysing of Building for Horizontal Loads*

  a. *Calculation of horizontal load due to wind*: Wind pressure intensity is fixed in accordance with IS:875 depending on the location o f the building and its over-all height above ground. The exposed area multiplied by the wind pressure gives the wind load which is applied at each floor level and then the cumulative wind load at base is calculated.
  b. *Calculation of seismic force*: Dead and live load at each floor are calculated separately either by summation of dead and live loads of-all columns coming on the floor (already calculated in step 2) or by finding the area of the floor and multiplying it by dead and live load intensities, considering all beams, columns, etc. coming on it. The base seismic shear is found in accordance with IS: 1893, taking the seismic intensity o f the area, the soil data and the flexibility of the building,

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etc. into account. If the base shear due to earthquake is greater than the base shear due to wind (calculated in step 3a), then earthquake base shear along the height of the building is calculated in aci ordance with IS: 1983.
c.*Allocation analysis*:The following methods may be used for allocation analysis: (i) Joint coefficients method (Reference 33); (ii) D-values method Reference 42); (iii) Rigidities method (Reference 38, chapter 7).

The choice of the method depends on the importance of the structure, the degree of irregularity of its planning, etc. The allocation analysis gives the shear at each floor level for each bent. As the horizontal shear can act on a building in any random direction, the allocation analysis is to be done in either principal direction separately.

For a symmetrical building, allocation analysis can be done by inspection only.
d.*Frame analysis*:For the horizontal loads calculated in step 3c, each frame is analysed by either of the following methods: (i) Method of multiples (References (46,47); (ii) Factor method (Reference 51); (iii) Portal or cantilever methods (Reference 99); (iv) Rigidities method (Reference 38) depending on the importance of the building and the degree of accuracy desired. Plane frame computer program may, instead, be used to get accurate results and this program has made the above methods nearly obsolete.

At this stage, we get the column and beam moments due to horizontal load and also beam shears and increase or decrease in column loads due to the effect of overturning. Face correction has to be applied to reduce the moment to the face of the members, as against their values at the centreline of members. Further, the sign of stress resultants due to horizontal loads shall be taken as , taking into account the reversible nature of horizontal loads.

- `Step 4:`*Estimation of Frame Moments in Columns due to Vertical Loads.* This can be done by solving a few substitute frames under the vertical load by moment distribution or by using table IX of IS:4 56-1964. These moments should not be less than the minimum eccentricity moments in columns as specified in IS:456-1978.

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- `Step 5:`*Design of columns*: With the knowledge of (i) Vertical load increment of load due to overturning; (ii) Moments due to horizontal loads on either axis; (iii) Moments due to vertical loads on either axis, acting on each column, at all floor levels of the building , columns are designed by charts (References 17, 58, 78, 81) with a load factor of 1.5 for vertical load effect and with a load factor of 1.2 for the combined effects of the vertical and the horizontal loads. This step confirms the size of columns assumed in the architectural drawings. The design o f each column is carried out from the top of foundation to the roof, varying the concrete mix and the amount of steel reinforcement and taking an appropriate age factor for the concrete cube strength in accordance with IS:456-1978. All columns of the building are arranged into a few suitable groups for ease in design. Further, slenderness effects in each storey are considered for each column group.
- `Step 6:`*Design of foundation*:With the knowledge of the column loads and moments at base and the soil data, foundations for columns are designed. The following is a list o f different types of foundations in order to preference with a view to economy: (i) Individual footings with or without seismic ties; (ii) Combination o f individual and combined footings with or without ties; (iii) Strip footings with retaining wall acting as strip beafn wherever applicable; (iv) Raft foundations of the types (a) slab (b) beam-sla b and (c) celluler raft. i.e. top and bottom slabs with deep beams in both directions with hollow spaces in-between; and (d) annulardraft, i.e. strips in both directions with cutouts left in plan; (v) Pile foundations. In this case, the column loads and moments are tabulated along with the plan of the grid of the building, which is attached with the N.I.T. (Notice Inviting Tender) for pile foundations, the design o f which is given by the piling contractor, to be got approved by the structural engineer.
  The brick wall footings are also designed at this stage. Often, plinth beams are provided to support brick walls and also to act as earthquake ties in each principal direction. Plinth beams, retaining walls if any, are also dc signed at this stage, being considered as part of foundations.
  At this stage, the draftsman takes up the work of preparing drawings ol the foundations and the column schedule of the building. These drawings canthen be issued to the site for execution.

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- Step 7: *Design of floor slabs and beams.* Design of floor slabs and beams is taken up with the ground floor upwards (a basement is assumed here). The slabs are designed as one-way or two-way panels, taking the edge conditions of the supporting edges into account, with the loading already decided in step 1. The midspan steel is put the same as the support steel and the deflection i liccked to get slab thickness as less as practicable. The beams are designed as countinuous beams, monolithic with reinforced concrete columns with-their far ends assumed fixed (substitute frames). The variation in the live load position u taken into account by following the two-cycle moment distribution. The moments are applied a face correction to reduce them to the face of the members. The moments due to horizontal loads are added to the above moments. Each section of the beam is designed for the greater of the following moments:
  (i) moment due to vertical loads.
  (ii) 0.8 ($1.2/1.5 = 0.8$) times moment due to vertical loads + moment due to horizontal load).
  The effect o f the shear due to vertical and horizontal loads is also similarly taken care of. It may be noted that the shear component due to wind or earthquake may be significant and it may affect the size and the range of shear stirrups. Bent-up bars are not effective for earthquake shear due to its alternating nature. The beam design can be easily done by a computer program which will give reinforcement at various critical sections along the length of the beam and also shear stirrups required. It saves considerable time and labour of it designer.
  The drawings are prepared floor-wise, starting from the ground floor (i c. basement roof) and upwards and issued to the site for execution in the nine order.

13.3. Design Office Practice For Short Buildings With Irregular Layout

Auditorium, cinema and factory buildings may come under this category of buildings. The building is expected to be short i.,e within about 4 storeys and ii is assumed that

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no one floor is identical with the other . The following steps m design may then be followed:

- `Step 1:` Design of roof slab and beams for vertical loads.
- `Step 2:` Design o f floor slabs and beams for vertical loads (3rd floor, 2nd floor and first first floor).
- `Step 3:` Computation of column loads. This step is best done by the summation of beam reactions already known in steps 1 and 2.
- `Step 4:` Analysis o f the building for horizontal loads
- `Step 5:` Design o f columns. Column load is known by step 3, column moments due to vertical load are known by steps 1 and 2 and those due to the horizontal loads by Step 4.
- `Step 6:` Design of foundations
- `Step 7:` Re-checking design of floor beams for horizontal loads. In small buildings, the vertical load design o f beams may remain unchanged except at a few locations. In this procedure, the entire structural design is complete and the drawings can be got prepared from the foundation level upwards and issued to the site for execution in the same order.

13.4. Design Office Practice For Tall Building With Shear Walls

The followings steps in design may be followed:

- `Step 1:` General loadings.
- `Step 2:` Computation o f column and shear wall loads by the tributary area method.

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- Step 3: Analysis o f building for horizontal loads as explained under Section 18.2 with the following changes:

  i) For buildings upto moderate height, say upto 20 storeys, full horizontal shear is given to the shear'walls. Allocation analysis is done by the Reference 52 (chapter 8) and each shear wall is designed as a vertical cantilever beam fixed at base. Frames are to be designed for at least 25% ol the horizontal shear.

  ii) For buildings more than 20 storeys, interaction of shear walls anil frames is considered approximately by Reference 25 (chapter 9) or accurately by Reference 48. Frames are to be designed for minimum 25% of the total horizontal shear.
- Step 4:Design of columns and shear walls. Design o f shear walls is given in Section 8.3. The simplest procedure for design of a shear wall is to find the stress diagram on the basis of gross concrete area and cover the stress diagram with steel, wherever the need be. This is based on the working stress method of design. This is analogous to the design of reinforcement for shells.100 Foiwalls with openings, similar procedure is followed, taking the effect of over turning and local bending o f piers into account and the reinforcement is calculated to cover tensile or excessive compressive stresses as given by the stress diagram.15 For the beam effect over openings, extra reinforcement is to be provided which is calculated for the local bending moment and checked forshear.
- Step 5: Design of foundations.
- Step 6: Design of slabs and beams floorwise, upto the roof level. Structural drawings will be prepared from the foundation level upward I issued to the site in the same order.

13.5. Computer Aided Design (CAD)

A powerful computer program STAAD-III is available in market for the analysis and design of multistoreyed buildings. We may consider the following methods o f approach

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in order to tap the full capacity of STAAD-III. Method-1:*Using STAAD-III for the Building as a Whole Steps in design*

- `1.` Manual design of slabs, stairs, chajjas, non-grid beams, etc
- `2.` Horizontal load analyis giving horizontal loads at joints o f all frames in both the principal directions. If we consider six degrees of freedom at each joint, then the rotational effect o f the earthquake shear, if any, will be considered automatically.
- `3.` Preparing data for STAAD-III program at all levels.
- `4.` Using computer, feeding data and getting the out-put. It gives beam and column design at all levels and it also gives column loads for design of foundations.
- `5.` Manual design o f plinth beams.
- `6.` Manual design o f footings
- `7.` Preparation of structural drawings with foundations and upwards by AUTO-CAD.

Merits: (i) Data preparation takes more time, (ii) computer has to be free i'ii u long time to take the in-put. (iii) Live load reduction in column loads has in be applied manually.

Method-II*Using continuous beam and plane frame programs*

For small-rise buildings (say upto 4 storeys) *Steps in design*

- `1.` Manual design of slabs, stairs and non-grid beams at all levels.
- `2.` Using continuous beam program for grid beams at all levels, using tri angular and trapezoidal loading shapes.
- `3.` Finding column loads by beam reactions and moments due to VL at all floor levels.
- `4.` Horizontal load (HL) analysis, and frame analysis for horizontal loads using plane frame program.

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- 5. Column design for VL and HL.
- 6. Checking grid beams for HL effect.
- 7. Design of plinth beams manually.
- 8. Design o f footings manually.
- 9. Preparation of structural drawings with foundations upwards.

Merits: (i) Reasonable use of computer, (ii) Accurate column loads

Demerits: (i) More manual work involved, (ii) Live load reduction in column loads to be done manually.

b. *For medium-rise building Steps in design*

- 1. General loadings at all levels.
- 2. Column loads by tributary area method .
- 3. Horizontal load analysis: use plane-frame program for frame analysis.
- 4. Column moments under VL approximate analysis manually.
- 5. Column design for VL and HL.
- 6. Plinth beam design.
- 7. Foundation design.
- 8. Preparation of foundation and column drawings and these may be issued to the site 'for the work to start.
- 9. Super-structure design-levelwise, with ground floor slabs and beams and upwards, till roof slab and beams and above. Here, we use continuous beam program for grid-line beams at all levels under both VL and HL.
- 10. Structural drawings for super-structure levelwise.

Merits:

- 1. Computer use is minimum and easy.

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- 2. Data preparation for compu'er work is quick.
- 3. This method gives results quickly.
- 4. Live load reduction in column loads is considered easily and it is a pan of the system.

Demerits:

- i Column loads approximate. 5% extra loads considered for unforeseen items to take care of this shortcoming.
c.*For medium-rise building(8 to 12 storeys)* This is given as an alternate method to (b).
- 1. Slabs and non-grid beams to be designed at all floor levels manually.
- 2. Divide the building into frames in two perpendicular directions.
- 3. Horizontal load analysis to get frame loads at joints.
- 4. Use computers plane frame program for all frames. This gives all grid beams design at all levels in both the directions.
- 5. Column loads to be assembled from frame analysis already done above under Step 4.
- 6. Column design.
- 7. Plinth beam design.
- 8. Footings design.
- 9. Structural drawings to be prepared with foundations and upwards.

Merits:

- i Optimum use of computer.
- ii Accurate column loads, with live load reduction to be done manually.

Demerits:

- i Data preparation takes time.

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- ii Live load reduction in columns is to be done manually.

It may be noted that where computer is used, some manual work may be needed to check or change the computer results. Also, some sketching work may be needed to explain details to structural raftsman for the drawing work lo start.

13.6. Use of computer in Structural Design of building

In earlier periods, say in sixties, engineering calculations were done by slide rules. Then in seventies, came the era of calculators, which replaced slide rules completely. The calculators are very much in vogue these days but to aid the structural engineer have come computers which are high-speed calculators mid much more. When similar members (say, slabs, beams columns, footings) are to be designed, relevant computer programs can be used to get the results meetly and speedily. Further in analysis, computers help us to use more ac tuate methods which were earlier difficult and time-consuming to use manually For example, frames under horizontal loads were analysed by portal or cantilever methods manually. These gave only approximate results and we liml to be satisfied with them. But now, computer can be used effectively to. H accurate results. Likewise, difficult structural problems o f analysis and de-sign can be solved by using appropriate computer programs.

13.6.1. Computer Programs in Use

Computers have recently been brought in to aid structural designers. These machines are as effective as the available software (i.e. computer programs). Programs are available for design of slabs, simply supported and continuous beams, columns under biaxial

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bending, isolated footings, combined footings and rats. There are also available 2-D plane frame programs, grids, retaining walls, etc. Not all the available programs may be to the liking of a designer. The assumptions made in the development of some programs may not be ac ceptable to the structural designer and in one case, it was found that the column program purchased was valid only for short columns and not for long columns, although at the time o f purchase, it was claimed to be valid for both short and long columns.

In practice, continuous beams, frame analysis under horizontal loads are almost always solved by a computer. Sometimes, a building is divided into frames in both the principal directions. Earthquake analysis is performed manually. All the frames with the vertical and the horizontal loads are put in the computer and the solution is got which includes the complete design of beams at all the floor levels. Column loads are then assembled manually. Column design under biaxial bending at all levels is done by a computer program Footings of all varieties are also designed by a computer. A raft is analysed by a computer program which is based on the elasticc method using sub-grade modulus of soil. A very powerful program STAAD-III is also available in the market which can be used for 3-D analysis of a building as a whole.

For specialized problems, reference is, always, made to the specialists or academicians in the field for their expert advice including the use o f the specialised computer programs available with them.

13.6.2. Prospects in Future

The immediate prospect in a near future, is the replacement of a desk calculator with a personal computer (PC). Each structural engineer will be provided with a PC, by which he can analyse and design all parts of a given building. He can only note down the results of design o f members by which he can gel the structural drawings made again by Autocad. The details of analysis and design of building can be kept in memory in a separate file for each building. The paper work will be drastically reduced. Each building will have a set oi structural drawings which will contain the results of the design of the building.

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14 New Reinforced Concrete Codeand Structural Design

14.1. Introduction

Indian Standard Code of Practice for Plan and Reinforced Concrete was first published in 1953 and its several revisions came out in 1957, 1964, 1978 and 2000, the latest being its fourth revision. The second revision IS: 456-1964 mainly dealt with the working stress method of design and gave the ultimate load method of design in its Appendix B. But the position of these two methods of design was reversed in the third revision IS: 456-1978. The working stress method was relegated to Section 6 of the code, while the limit state method being a rationalization of the ultimate load method, given in Section 5, formed the main body of the code. In the fourth revision IS: 456-2000, [112] limit state method occupies Section 5 of the code and it remains unchanged as given in IS: 456-1978. The working stress method has been pushed out of Annexure B of the code and it also remains the same as given earlier in IS: 456-1978. The main changes occurring in IS: 456-2000 refer to Sections 2 and 3 of the code which give new provisions regarding durability and fire resistance of concrete members. Structural designers, used to IS: 456-1978 for about two decades, find it difficult to use some of the provisions of the sections 2 and 3 of the new code (IS: 456-2000) and it is the aim here to give guidelines so that I he new code can be easily and correctly applied in structural design.

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14.1.1. Durability and Fire Resistance Requirements

Sections 2 and 3 of the code give requirements for durability of concrete members. Reddi has explained these provisions in detail with deep insight into the background o f these additions in the code. But, in practice, design engineers are facing many difficulties in applying section 2 of the code. The first thing to be decided before starting the structural design is to fix the nature of exposure zone as per Table 3 of the code. There are many situations given under each exposure zone from mild to extreme conditions. A practising engiueer has suggested to have a map of India with exposure zones clearly marked therem, on the analogy o f earthquake zones, in order to ease this problem.

Further, Table 3 of the code may suggest one concrete mix for floors and another one for roof in a given exposure zone. It may also require one concrete mix for internal columns and another one for external columns. For basement walls and foundations, concrete mix has to be different. This way, we shall end up with four to five concrete mixes for a given building. This is just not practical in the field and it will cause confusion at the site during both execution and supervision. The present practice is to have a general concrete mix for all reinforced concrete work, except for columns in lower storeys, where richer concrete mixes are to be used to save steel consumption in columns. An engineer should stick to this time-honoured practice.

The following prominent changes from the previous code IS :56-1978 have been observed in the new code IS: 456-2000.

(1) Exposure zone for the location of building has to be decided. Assume mild exposure zone for structures in mid-land areas like Delhi, etc. Assume moderate exposure zone for buildings in coastal areas like Mumbai, Chennai, Kolkatta, etc. Assume severe exposure zone for structures immersed in sea water.

(2) Minimum concrete mix is changed from M-15 to M20, M25 and M30m respect of mild, moderate and severe exposures zone respectively.

(3) Clear cover to outer bars has been increased. Previously, the clear cover was specified over main bars.

(4) Minimum slab thickness has to be 110 mm for 1-1/2 hour fire resistance. Previously, minimum slab thickness, in practice, was 100 mm.

(5) Clear cover over outer slab bars has been increased from 15 mm to 20mm, taking both tables 16 and 16A of the code into account.

(6) PCC shall be M10 (1:4:8).

(7) Age factor = 1.0 shall only be considered. Values of age factor greater than 1.0 shall not be considered as per the new code,

(8) Minimum shear strength of concrete shall be considered as given for $p_\tau < 0.15$ in Table 19 of the code. For M20, for example, $\tau_c = 0.28$ N/mm^2 for $p_\tau < 0.15$. Previously, were assuming $\tau = 0.35$ N/mm^2 for $p_\tau < 0.25$.

(9) Values of cover ratios for using column design as given in SP-16 will now work out more and this aspect will lead to an increase in column steel also in compression steel in beam sections.
Table 19.1 gives requirements of IS 456-2000 for durability and fire resistance in design of buildings and it is based on Table 3, 5, 16 and 16A of the code. This table will be helpful to designers in applying IS 456-2000 to the structural design of buildings.
It is to be noted that use of IS: 456-2000 tends to increase both the quantity and quality of concrete, leading to an increase in cement consumption in buildings. This aspect will increase cost of reinforced concrete buildings by about 3%.

Notes:

(1) The other two exposure zones, very severe and extreme, have not been given here, being applicable to special situations.

(2) Slab covers have been reduced by us in moderate and severe exposure zones in order to restrict crack width and also to reduce dead load of buildings.

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Exposure Zone	Where applicable	Minimum Concrete Mix	Nominal covers for members (mm)	Maximum thickness of slab	Remarks
Min	Concrete surface protected against weather or aggressive conditions in non-coastal regions	M20	slabs 20 Beams 25 Columns 40 Footings 50 Ret.Walls 25	110	For buildings in midland areas like Delhi, etc.
Moderate	Concrete surface sheltered from saturated salt air in coastal areas	M25	slabs 30 (20 to use in practice) Beams 30 Columns 40 Footings 50 Ret.Walls 30	110	For buildings in coastal areas like Mumbai, Chennai, Kolkatta etc.
Severe	Concrete surface exposed to coastal environment or completely immersed in sea water	M30	slabs 45 (30 to use in practice) Beams 45 Columns 45 Footings 50 Ret.Walls 40	140	For structures immersed in sea water

Table 14.6: Requirements of IS: 456-2000 for durability and fire resistance indesign of buildings (based on Tables 3, 5, 16, 16A of the code).

14.1.2. Shortcomings in the Code IS 456-2000

Basu has pointed out some shortcomings in the present code. Some more shortcomings of the code are given below.

(1) Partial safety factors for loads, as given in Table 18 of the code (p. 68) are erroneous. Table 18 of the code gives,

$$U = (1.5(DL + LL)) \tag{14.1}$$
$$U = (1.5 \text{ or } 0.9DL + 1.5WL) \tag{14.2}$$
$$U = (1.2(DL + LL + WL)) \tag{14.3}$$

The factor 1.5 is to be applied to permanent load like DL and it should not be applied to WL (or EQ) which is an occasional load.ACI code gives for occasional loads a multiplying factor of 0.75. On this basis, the partial safety factor for including WL will be equal to 0.75 x 1.5 = 1.125 which has been rounded off to 1.2 in our code. SP: 24 states the equations (1) and (3) should be applied in building design and equation (2) should be applied to chimneys and cooling towers, where the lateral loading (wind or earthquake) is the primary imposed load. Use of equation (2) in building design will lead to huge consumption of steel and concrete in buildings and this wastage of materials must be stopped forthwith. The new code should have considered this aspect and should have ensured compatibility of the code with SP: 24.

(2) In analysis o f structures, effect of slab on the moment of inertia of beams should be considered for the sake of realistic behaviour of rein forced concrete structures. Clause 22.3 o f the code is silent on this aspect.

(3) The code gives two methods of checking the adequacy of deflection of bending members like slabs and beams. One method is the method of calculation of deflection and the other method is that of span to effective depth ratios of members. The first method is a strict method and the second method is approximate and easier method and it is based on the first method. But, both these methods do not lead

to the same result in a given example, unless we consider adequate camber and also consider only half the value o f deflection due to creap and shrinkage. This is shown elsewhere on the basis of IS 456: 1978, the deflection criteria remaining unchanged in IS: 456-2000.

Further, the computed total deflection should not exceed L!250 - this condition is not important, as it can always be satisfied by a suitable camber. The second condition o f the code (clause 23.2b) that the partial deflection after the erection of partitions and finishes including the effects o f live load, temperature, creep and shrinking should not exceed L/350 or 2.0 cm condition is very severe for beams of spans longer than 7.0m. It is suggested that 2.0 cm lequirement should be deleted and only L/350 should be kept. This is in line with ACI code. It may be noted that L /d -ratio method is strictly not valid for two-way slabs, flat and girds, as these all are two-way spanning systems. The method of calculation of deflection along with the allowable limits o f deflection of LI 250 or LI 350 is valid for these systems. This is the correct position but the code has not made this point clear.

(4) Slab thickness is governed by deflection. Slab design in accordance with the code is an interactive process and it leads to thick slab panels. This aspect leads to an increase in the dead load of buildings and consequently higher cost. Thin slab panels can be designed by putting at midspan some more steel, say equal to the support steel. This way, we get efficient detailing of slab bars and also achieve less thickness of slab panels. This method has been suggested by SP: 24 (p. 55). This aspect has not been mentioned by the code.

(5) Stirrups in a circular column section shall be provided on the same principles as one does for a square column section. Lateral ties should be provided in a circular section so that each bar is located at a comer of a rectangular or a triangular stirrup. One extra circular link is also provided to keep the bars in position along the circular periphery. Alternatively, helical stirrups should be provided which are costly to provide in practice. This aspect has not been mentioned by the code and its detailing has also not been given by SP: 34, a BIS publication on the subject of detailing. This aspect provides disagreement among engineers specially during checking process.

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(6) Minimum vertical reinforcement in reinforced concrete walls shall be 0.4% as per SP:24, but the code gives only 0.12% in its clause 32,5 which is very much on the low side. Minimum horizontal reinforcement in walls has been given by the code to 0.2% which is a reasonable value. Action of a wall is similar to that o f a column and vertical steel should be more than the horizontal steel in walls. The code and SP: 24 should have been made to tally with each other.

(7) Age factors for increasing concrete strength have been withdrawn by the code. Use of age factors was sanctioned by the previous version of the code and it was effective in reducing steel consumption in columns of multistoreyed buildings, particularly, where future provisions of one or more storeys was required to be kept in design. This aspect will result in an increase in steel consumption in buildings.

(8) Fig. 4 of IS: 456-2000 (p. 38) gives some additional values of $\mathrm{f_s}$ = 120, 145, etc. and these will be useful when $\mathrm{f_s}$ ¡ 0.58 $\mathrm{f_y}$, which happens when one provides more steel than that required for moment. A better and more useful figure has been given in the Handbook of Ghanekar and Jain (p. 292) which should have been included in the new code for effectiveness'. Further, it will be better to give the following formula (SP:24).

$$\mathrm{MF} = \frac{1}{[0.225 + 0.00322 f_s - 0.625 log\,()]}$$

where

$$f_s = 0.58 f_v \times \frac{A_{st}(reqd)}{A_s}$$

A_{st} = steel area provided.

This formula is the basis of Fig. 4 of the code and it can be directly used in design without any reference to Fig. 4. This will be a better and more accurate and convenient approach.

(9) Clauses 25.12 and 39.7 of IS: 456-2000 give design o f slender columns of solid sections. If a column is hollow inside with an annular section, both global and local buckling effects must be considered in design. This aspect is not mentioned by the code. Some circular staging shafts supporting concrete overhead tanks have failed on account of neglecting the effect of local buckling of shaft walls. For circular shafts, as used in concrete silos, IS 4955-1974$_{13}$ gives a permissible concrete stress of 0.15 f_{ck}, which takes into account both the overall and the local bucking effects. Buckling effects o f overall or local varieties - are serious effects and these need to be carefully considered, in order to prevent failure of thin hollow members under compression.

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