STRUCTURAL DESIGN OF MULTI-STOREYED BUILDINGS

SECOND EDITION

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An interesting ancient question
'What is that, knowing which,
we shall know everything?'

Vedas, quoted by Swami Vivekananda in his
Collected Works Vol. I, p. 362

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Preface to Second Edition

The first edition of this book came out of press on 26 June 1999 and it is now exhausted. The second edition is therefore being brought out to meet the demands of the profession.

Two extra chapters have been added in this edition giving aspects of structural design with emphasis on the new reinforced concrete code IS: 456-2000. Further, appendix II has also been added giving twenty four useful tables, chapter wise, which will be of benefit to structural designers in their day-to-day office work.

The text of the book has been thoroughly checked by my engineer friend Lt. Col. G.L. Sethi (MES, Retd) and corrections have been made wherever needed. I am grateful to him for his interest and efforts.

The book is humbly dedicated to the profession of structural engineering.

New Delhi
1 January 2002

Preface to First Edition

Having spent my entire professional life in the structural design of reinforced concrete buildings, I wish to share my knowledge and experience with the profession of Structural Engineering, in the form of this book.

The book consists of seventeen chapters. Chapter 1 is of a general nature and it serves to introduce the subject matter of the book. Chapter 2 gives vital information on structural systems for buildings, while chapters 3 and 4 give details of types of floors and stairs respectively. Design of masonry buildings, a topic which generally goes by default, is given in chapter 5. The core of the book is given in chapters 6 to 9, which deal with framed and shear-walled buildings under vertical and horizontal loads. Chapter 10 introduces the reader to space structures in their application to sloping roofs. Foundations of buildings are covered in Chapter 11. Chapter 12 again touches a vital aspect of design of reinforced concrete elements and it gives the material which can be considered as additional to my earlier book 'Manual for Limit State Design of Reinforced Concrete Members'. Detailing of reinforced concrete elements is given in chapter 13. Practical considerations like economy in building design and precautions in construction and site supervision are covered in chapters 14 and 15 respectively.

Chapter 16 gives a rare study of structural failures. Finally, steps in design , followed in design office practice, are given in the last chapter 17, together with the present stage of computer applications in the structural design of buildings. The text is interspersed with 32 numerical examples, 253 figures and 23 tables. The references studied during the preparation of the book are given in Appendix I and their numbers are noted within the text of the book. The notation is not given separately, it is rather explained where the symbols first appear in the text.

What is unique about this book is that the material otherwise spread in various technical journals and books is collected at one place and the subject matter is presented in the way as it is followed in a design office. There are not many books on the subject of structural design of multi-storeyed buildings, rather chapters on this subject can be found in many publications. So, a comprehensive book on this subject was called for and this book is expected to fill up the need of the profession of structural engineering.

I am highly obliged to my senior colleague Mr. J.D. Buch for his encouragement and advice in this venture and for the efforts put in by him in reviewing the entire manuscript and giving valuable suggestions for the improvement of the subject matter of the book. Heartfelt thanks are due to my colleagues – both engineers, draftsmen and typists – in the office who have been enthusiastically helpful in the preparation of the manuscript. Special thanks are due to Mr. Rajesh Kumar, Senior Structural Draftsman, for his willing help in the preparation of neat and clear figures and tables.

The acknowledgment of thanks will not be complete without the mention of my dear friend and a excellent engineer Mr. P.L. Assudani who is the co-author of the two papers published in the sixties which form the core of this book.

This book is humbly dedicated to the profession of structural engineering, with a hope that it will prove to be of immense benefit to all the readers, in the fact that it will give a second and considered opinion on most of the problems involved in the structural design of buildings.

New Delhi.
13th April 1999

U.H. VARYANI

CHAPTER 1

Introduction

1.1 NEED 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. 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 extraordinarily tall out of prestige. For example, Delhi Development Authority'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.

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.

1.3 LOADS

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 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.

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.[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. 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^2 per cm depth of snow. With 30 cm snow depth, the snow loading will work out to be 75 kg/m^2, which may be reasonable for sloping roofs.

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 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[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 $(0.115 \times 1.0 \times 3.0 \times 1900)/3 = 218.5$ 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

* The numerals refer to the serial number of references given in Appendix I at the end of the book.

thick 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 , 115 mm thick brick walls are used as partition walls.

1.3.2 Temperature and Shrinkage Loading

Temperature and shrinkage also act on a building and these can also be regarded as a load on it. Shrinkage is equivalent to –15 °C, where negative stands for fall of temperature.[3] The temperature differential is taken at $\pm\frac{2}{3}(t_1 - t_2)$, 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 ±17 °C for moderate climates and at ±25 °C for extreme climates. The combined effect of temperature and shrinkage is given below:

For moderate climates: $\pm 17 - 15 = +2, -32$ (°C)
For extreme climates: $\pm 25 - 15 = +10, -40$ (°C)

IS:456 1978[6] (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,[7] what is meant by an ordinary building. It is, of course, clear that temperature and shrinkage loading has an effect on the design of long concrete buildings,[8] which can be neglected if the length of building is restricted to 45 m (clause 26.3 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.[8]

1.3.3 Wind Loading

Dead and 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 gives values of wind pressures varying from 100 kg/m^2 to 200 kg/m^2 acting on building upto a height of 30 m 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.[9] India has been divided into five zones with basic horizontal seismic coefficient (α_0) varying from 0.01 to 0.08. The base shear (V_B) is given by

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

where,

C = coefficient 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 coefficient = $\beta I \alpha_0$

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 + appropriate amount of live load (25% to 50% I.L)

β = 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.

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

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 or tall, shall be checked for horizontal effect of earthquake loading.

1.3.5 Blast Loading

In certain coalmine 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[10] gives the blast effects,

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

$$= K_2 \frac{Q^{0.83}}{R^2} \tag{1.2}$$

where, $K_2 = 4$

Q = charge per delay in kg

R = distance of structure from blast point in metres.

The calculated a/g value for blast may be taken as a fraction of earthquake effect given by α_0. The building can be analysed for, say, earthquake and then the blast effect can be 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. (1.2) gives,

$$\left(\frac{a}{g}\right)_{\text{blast}} = 4 \times \frac{(20{,}000)^{0.83}}{(1200)^2} = 0.01$$

This indicates that the blast 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:875 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 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 load of 1000 kg/m^2 together with an impact allowance of 100%, giving a load of 2000 kg/m^2.

1.3.7 Earth and Water Pressures and Surcharge Loads

These loads act on a retaining wall. In Fig. 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

$$p_e = \text{earth pressure} = \frac{1 - \sin\phi}{1 + \sin\phi}\,\gamma\, h \tag{1.3}$$

$$p_w = \text{water pressure} = w\, h' \tag{1.4}$$

$$p_s = \text{surcharge pressure} = \frac{1 - \sin\phi}{1 - \sin\phi}\, w_s \tag{1.5}$$

where, ϕ = angle of repose of earth

γ = unit weight of earth

w = unit weight of water

w_s = surcharge loading = 500 kg/m^2 for pedestrian traffic
= 1000 kg/m^2 for vehicular traffic
h = height of the earth retained
h' = height of the subsoil water retained

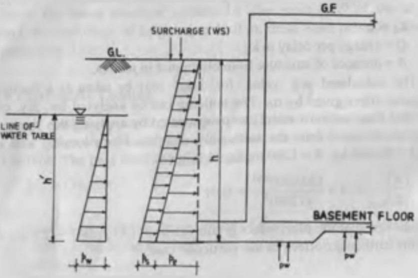


Figure 1.1. Pressures on a basement retaining wall.

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 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,

$$U = 1.2\,(D + L \pm W \text{ or } E \text{ or } T) \tag{1.7}$$

where the value of 1.2 is derived from $0.75 \times 1.5 = 1.125$ 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 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 for isolated tall structures,

$$U = 1.2(0.9D \pm W \text{ or } E \text{ or } T) \quad (1.8)$$

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,

$$U = 1.5\,(D + L + B) \quad (1.9)$$

Also, it 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 Eqs. (1.5) to (1.9) is explained below.

U = ultimate load
D = dead load
L = live load
W = wind load
E = earthquake load
T = temperature and shrinkage load
B = blast load

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 form the basis on which the other professionals like structural, estimating, plumbing, electrical and airconditioning engineers work to prepare their own drawings and estimates. An architect is primarily concerned with the function and the area utilization of building. He takes care of the visual look of the building, its orientation and its position in the surrounding 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 tenderers.

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 contractor's 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. 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.

CHAPTER 2

Structural Systems for Buildings

2.1 INTRODUCTION

Choice of an appropriate structural system for a given building is vital for its economy and safety. It is an important decision which is to be taken by a senior structural engineer. Small buildings like houses, etc. generally use load bearing brick walls with reinforced concrete floor slabs. For taller buildings, reinforced concrete frames in both the principal directions are provided with brick 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. Likewise, more intricate and innovative structural systems may be thought 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 suitable for buildings upto four storeys or less in height (Fig. 2.1). 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^2 minimum 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.[11] 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 necessary. IS: 1905[12] is the code governing the design of brick wall structures.

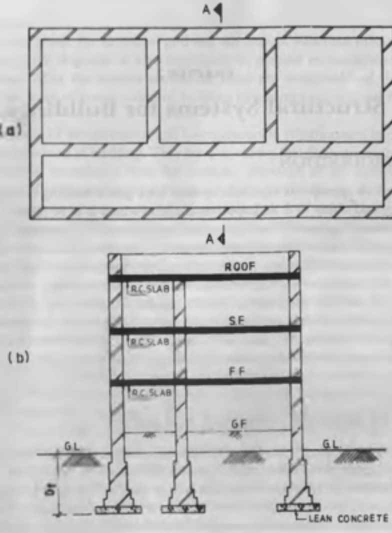


Fig. 2.1. Structural system consisting of load bearing brick walls. (a) Plan of buidling; (b) Section A-A.

2.3 TWIN SYSTEM OF BRICK WALLS 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 × 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 × 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, restricting optimum use of the covered area. This is an innovative structural system, leading to great economy, where applicable.

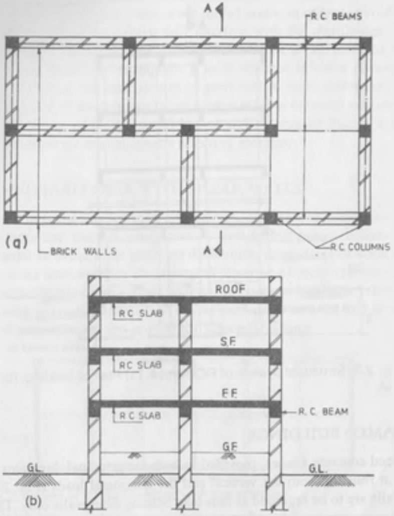


Fig. 2.2. Twin system of brick walls and RC columns. (a) Plan of building; (b) Section A-A.

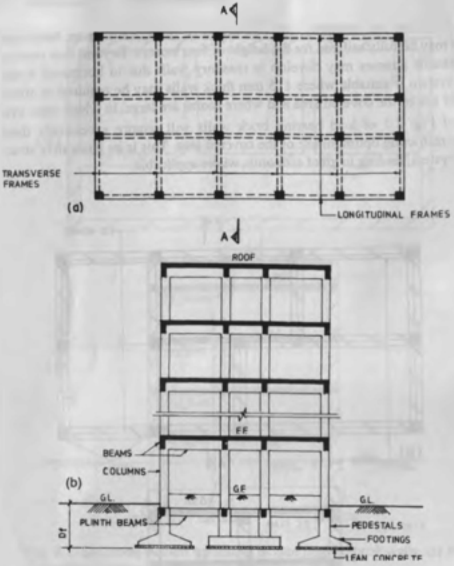


Fig. 2.3. Structural system of RC frames. (a) Plan of building; (b) Section A-A.

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. The finishes and the partition walls 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 walls 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 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.

2.5 FRAMED BUILDINGS WITH SHEAR WALLS

When a building exceeds ten to twelve storeys, column and beam sizes work out 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.

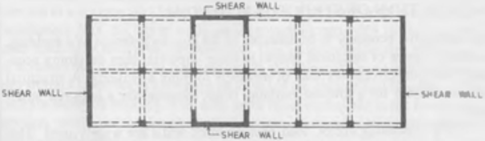


Fig. 2.4. Plan of framed building with shear walls.

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.[13] This system is suitable for 10 to 20 storeys.

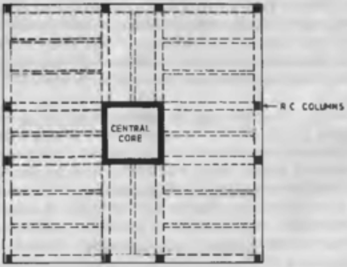


Fig. 2.5. Plan of framed building with central shear core.

A shear core (or box) housing lifts, toilets and other services may be provided preferably placed at the centre 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 on the periphery to act as a core-within-core system (Fig. 2.6).[14] For buildings higher than 40 storeys, multi-cored systems may have to be devised.[15]

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 structure.

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.

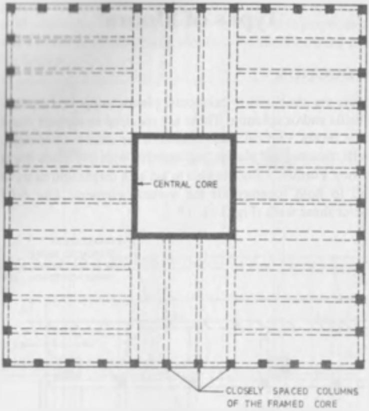


Fig. 2.6. Plan of building with a core-within-core system.

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 centre to centre. 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.

CHAPTER 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).

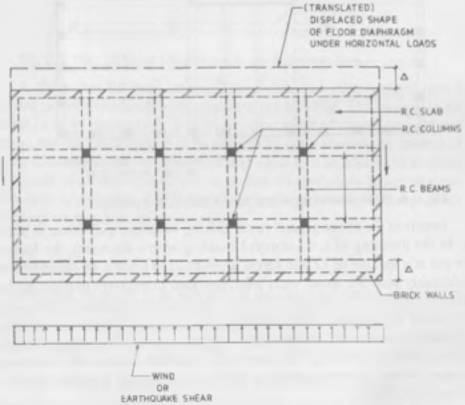


Fig. 3.1. Diaphragm action of floor.

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 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 in-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.

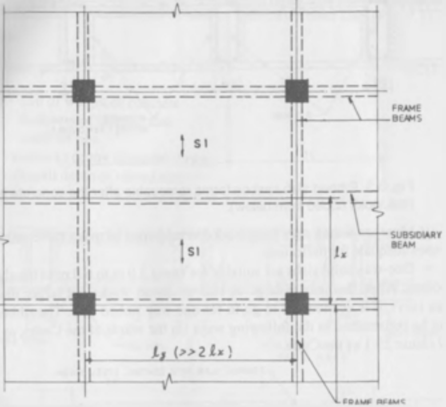


Fig. 3.2. Arrangement of one-way slab panels in a floor plan.

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 $l_y/l_x \gg 2$, 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 centre 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.[3] For,

Single span one-way slab : $D = l/30$
Continuous one-way slab : $D = l/35$

where, l = span of one-way slab
D = over all depth of concrete slab
l_x = span in short direction
l_y = span in long direction

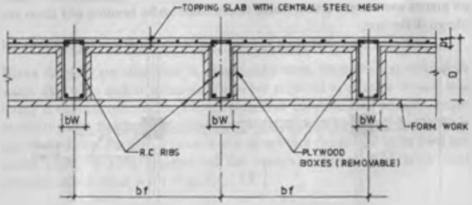


Fig. 3.3. Ribbed slab cast on forms removable after the concrete has set (this looks ribbed from below).

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 200 mm (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.

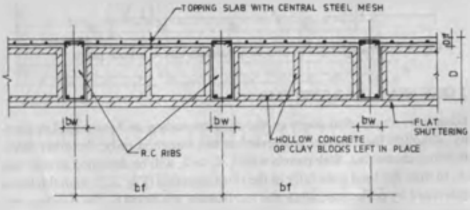


Fig. 3.4. Ribbed slab with precast blocks left in place (this looks flat from below).

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

b) As a series of concrete ribs between precast blocks which remain part of the 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).

c) With a continuous top and bottom face but containing voids of rectangular, 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,[17]

$$w_s = \rho_c\left[D_f + \frac{b_w}{b_f}(D - D_f)\right] \tag{3.1}$$

where ρ_c = unit of weight of concrete
D_f = thickness of topping slab
b_w = width of ribs
b_f = centre to centre distance of ribs
D = overall depth of ribbed slab

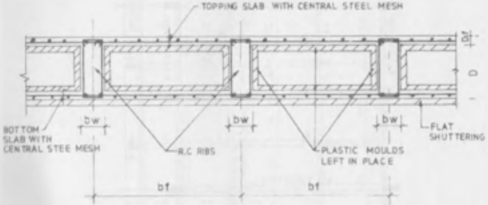


Fig. 3.5. Ribbed slab with continuous top and bottom slab (this looks flat from bottom).

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 of 20 cm thick slab = 0.20 × 2500 = 500 kg/m^2. With b_w = 0.15 m, b_f = 0.75 m, D_f = 0.05 m, D = 0.30 m and ρ_c = 2500 kg/m^3, the self-weight of the ribbed slab is given by Eq. (3.1),

$$w_s = 2500\left[0.05 + \frac{0.15}{0.75}(.3 - .05)\right] = 2500\,(.05 + .05) = 250 \text{ kg/m}^2$$

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

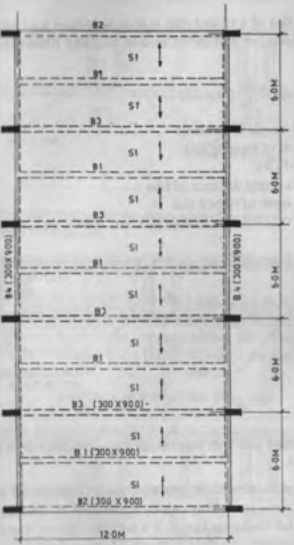


Fig. 3.6 Efficient arrangement of one-way slab panels in a floor plan.

Two structural arrangements for a floor with one-way slab panels are given in Figs. 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 B1, B2 and B3 have less loading, while the short-span beam B4 has more 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 beams, the ceiling may look ugly. It is however, seen that the concrete quantity is nearly the same in these two arrangements.

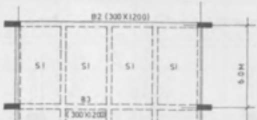


Fig. 3.7. Inefficient arrangement of one-way slab panels in a floor plan.

3.3 TWO-WAY SLAB SYSTEMS

Two-way slab panels are efficient as the material gets stressed in the two perpendicular directions. This system is suitable for square or squarish panels with $l_x/l_y \leq 2$ (Fig. 3.8). The slab panel bends in the two cross directions and it assumes a saucer-like deflected 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.[3]

For a panel simply 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

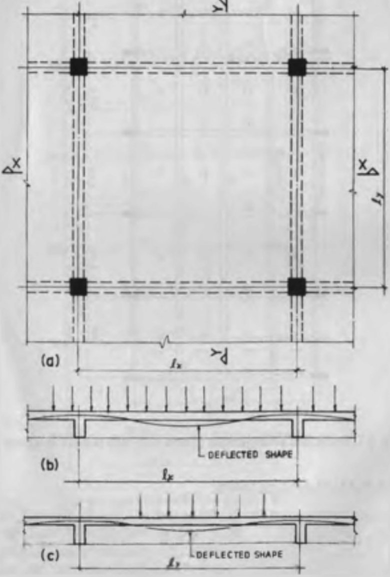


Fig. 3.8. Two-way slab panel in plan. (a) Plan of slab panel. (b) Section X-X; (c) Section Y-Y.

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.

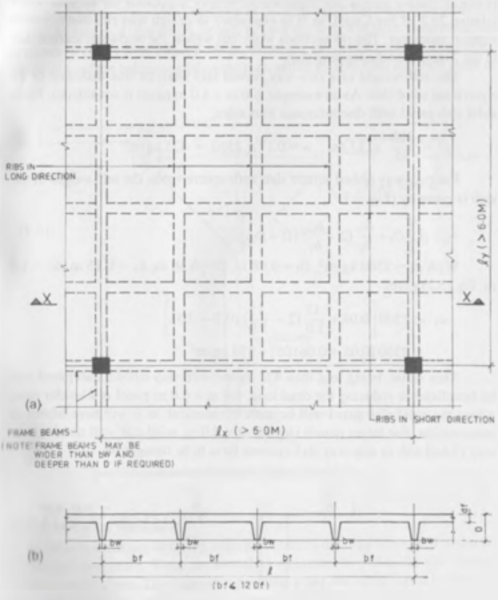


Fig. 3.9. Two-way ribbed slab panel in plan. (a) Plan; (b) Section X-X.

When the panel size is large or slab thickness works out large, two-way ribbed slab may be used to save on the self-weight of the slab-panel (Fig. 3.9). The construction systems as given in Figs. 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,

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

For two-way ribbed square slab with square voids, the self-weight of the slab is given by (Fig. 3.3)[17]

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

With $\rho_c = 2500$ kg/m³, $D_f = 0.08$ m, $D = 0.30$ m, $b_w = 0.15$ m, $b_f = 1.0$ m, Eq. (3.2) gives

$$w_s = 2500 \left[0.08 + \frac{.15}{1.0} \left(2 - \frac{.15}{1.0}\right) (0.3 - .08) \right]$$

$$= 2500\,[0.08 + 0.06105] = 353 \text{ kg/m}^2$$

This value, being less than 425 kg/m², 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 × 9.0 m, solid slab will not do, two-way ribbed slab or one-way slab systems have to be thought of.

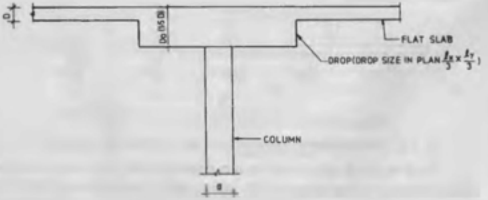


Fig. 3.10. Flat slab with drop (without capital).

Whether a slab panel will behave as a one-way or a two-way panel, solely depends on the panel dimensions in plan. When $l_y/l_x \leq 2$, 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 propensityof the behaviour of a structure, rather than to foist our own ideas on it.

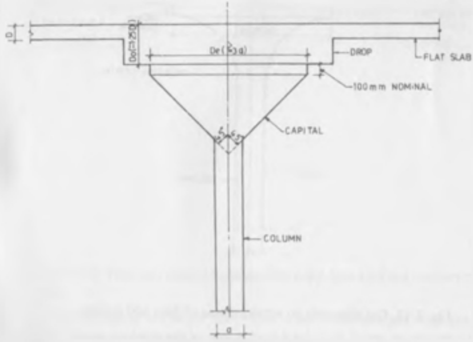


Fig. 3.11. Flat slab with drop with column capital.

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 to punch through the slab. In order to protect the structure against the punching effect, slab portions around columns are made 25% to 50% thicker than the 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 × 6.0 m to 9.0 m × 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.

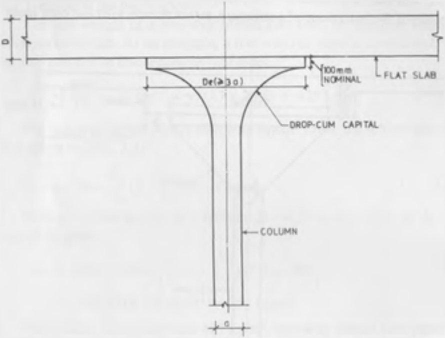


Fig. 3.12. Flat slab with an artistic shape of drop and capital.

The Code gives two methods of design of flat slab structures:

(i) Emperical method;
(ii) 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 horizontal shear 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.

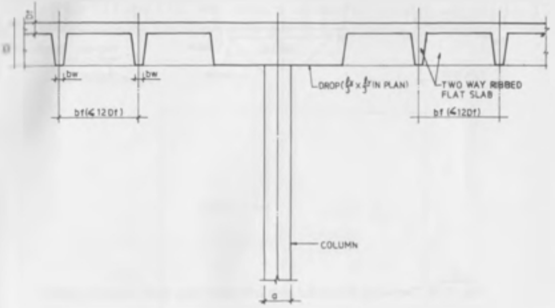


Fig. 3.13. Two-way ribbed flat slab with solid drop (without column capital).

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 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), where $l = \dfrac{l_x + l_y}{2}$

l_x = span in short direction
l_y = span in long direction

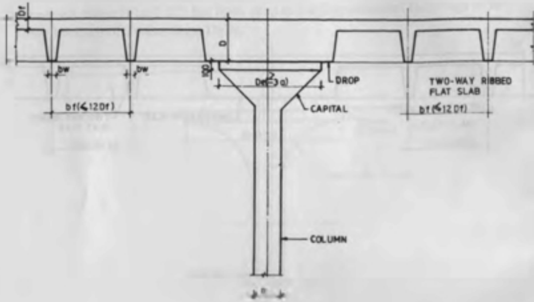


Fig. 3.14. Two-way ribbed flat slab with solid drop (with column capital).

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, where $l = \dfrac{l_x + l_y}{2}$

This system may be usefully provided for small spans in either direction, such as 6.0 m × 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 Figs. 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.

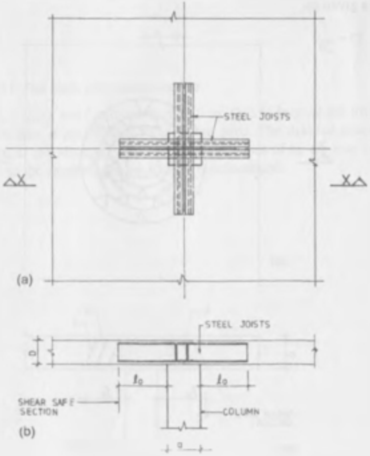


Fig. 3.15. Flat plate system with structural steel shear head (without cloumn capital). (a) Plan; (b) Section X-X.

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 solid slab 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 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 (b_f) 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,

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

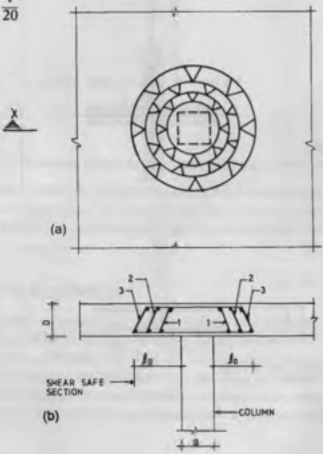


Fig. 3.16. Flat plate with shear head of steel bars (without column capital). (a) Plan; (b) Section X-X.

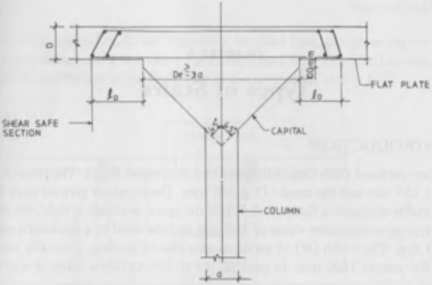


Fig. 3.17. Flat plate with column capital.

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.[17] Grids with large values of b_f, are easy to construct at site, as the shuttering in such cases is economical.

CHAPTER 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 (T) at 300 mm. These values give an easy and comfortable 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 structural engineer is to fix its waist slab thickness, steel bars and designs the supporting elements like beams and columns.

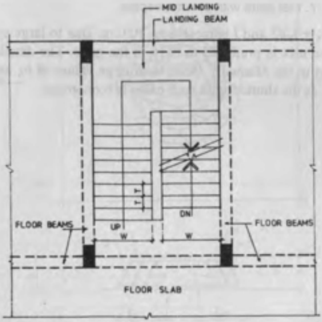


Fig. 4.1. Plan of two-flight stairs.

4.2 COMMON TYPES OF STAIRS

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

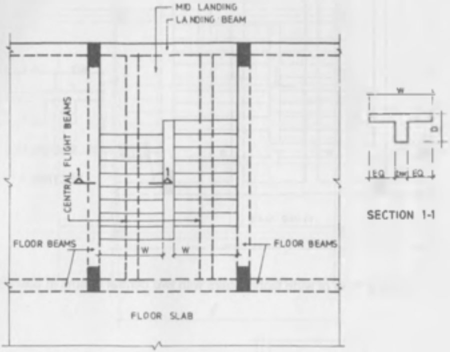


Fig. 4.2. Plan of two-flight stairs with central flight beams.

The waist slab works out, in most cases, to be 150 mm to 200 mm thick. In certain cases, waist slab works out to be about 300 mm thick, which does not appear elegant to the view. In such cases, a small amount of compression steel may be permitted at the mid-span section and the waist slab thickness can then be brought down to a value of 200 mm. This arrangement is valid for spans 4.0 m to 6.0 m. When the spans are longer than 6.0 m, central beams may be put to support flight slabs, which may be cantilevered out on either side (Fig. 4.2). For extra wide flights, two central beams may also be provided to get an economical structural design (Fig. 4.3). The flights with central beams do not look good to the view, where the space is restricted. In such situations, 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 (clause 32.1 of the Code) works out to be equal to the going plus 1.0 m or half the landing width (whichever is less) on either side. Generally, a staircase needs four columns in plan. Of these, two columns are a must for supporting the landing beam.

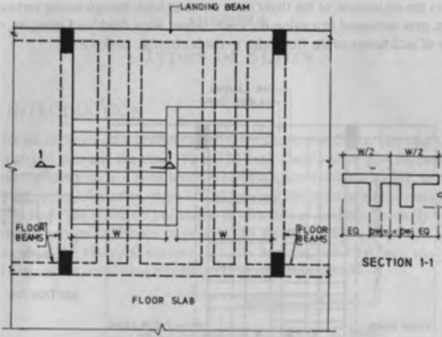


Fig. 4.3. Plan of wide flights stairs with two central beams in each flight.

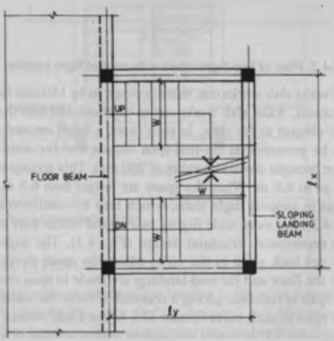


Fig. 4.4. A three-flight stairs.

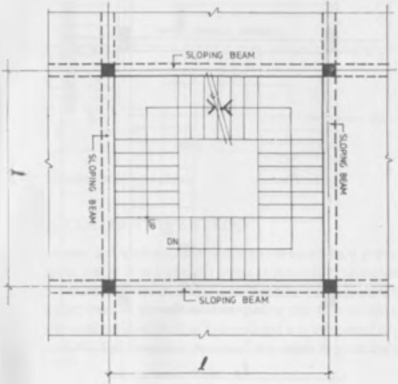


Fig. 4.5. A four-flight stairs in a square room in plan.

Three- or four-flight stairs are also planned with an open well in the central area (Figs. 4.4 and 4.5). In Fig. 4.4, the two long flights of span ly may be considered as main flights supported on ends by floor beam on one side and by the sloping landing beam on the other. The intermediate short flight of span ($l_x - W$), may be regarded as a suspended flight, supported on the two main flights. In a square grid $l \times l$, 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 landing loading being taken half its value in each direction. This is explained in clause 33.2. of the Code. When the staircase room is rectangular in plan ($l_y > l_x$), l_y-flights 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, the mid-landings can be supported by hangers suspended from a floor beam above, 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 of by the architect concerned.

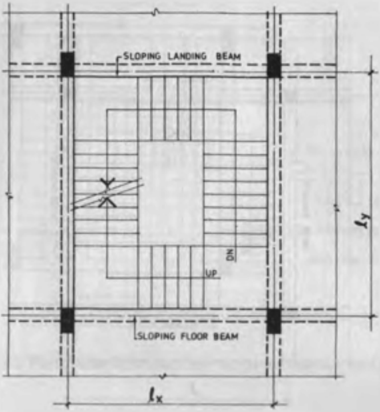


Fig. 4.6. A four-flight stairs in a rectangular room in plan.

4.3 CENTRAL-WALL TYPE STAIRS

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 either side of the central wall and the landing slabs too are cantilevered 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 oval or other artistic shapes can be made in the central wall, in order to save on the materials 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 floor slab. It is, thus, not advisable to separate out the central-wall stairs from the main building by an expansion joint.

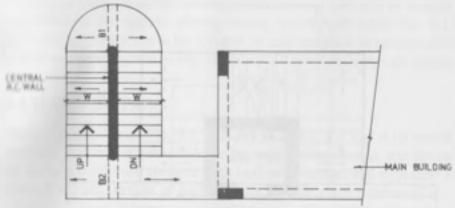


Fig. 4.7. Central-wall type stairs.

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 750 mm. 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 lowered and it is concreted to give the spiral stairs (Fig. 4.8). The structural design of this stairs is given by Jain and Jaikrishna Vol. I.[18]

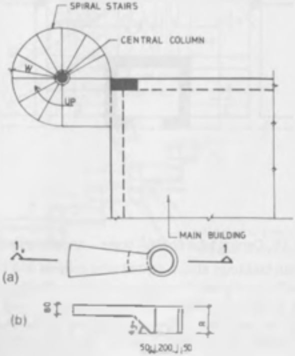


Fig. 4.8. Spiral stairs. (a) Plan of precast step; (b) Section 1-1.

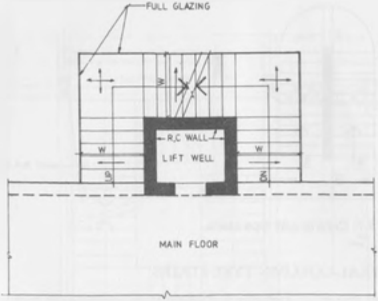


Fig. 4.9. Central tube column stairs.

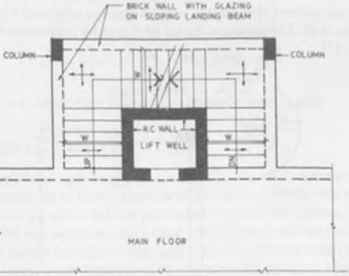


Fig. 4.10. Central tube column stairs – another alternative.

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 steps coming 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-way between the outside, sloping beams, and the lift walls (Fig. 4.10). The lift walls will work out to be 150 mm or 230 mm thick in reinforced concrete and these systems given in Figs. 4.9 and 4.10 work out very cost effective.

4.5 SLABLESS STAIRS

Stairs types given in Fig. 4.1, 4.4, 4.5, 4.6, 4.7, 4.9 and 4.10 can be made slabless, in that, the waist slab can be omitted (Fig. 4.11a), and the steps can be made in reinforced concrete with uniform thickness and with steel arrangement of closed stirrups in both the rise and the tread portions (Fig. 4.11b). There are many methods given for the design of the slabless stairs.[19,20] But the net result has been given,[21] in that, it can be designed in the way of any common stairs with the depth and the steel calculated as for any solid slab. Only the steel arrangement has to be put in loops in both the rise and the tread portions and also in landing slabs. The slabless cantilever steps are more effective as the depth available for resisting moment equals $(R + D)$ (Fig. 4.11b), thereby saving on the steel consumption. An example of slabless stairs is given below.

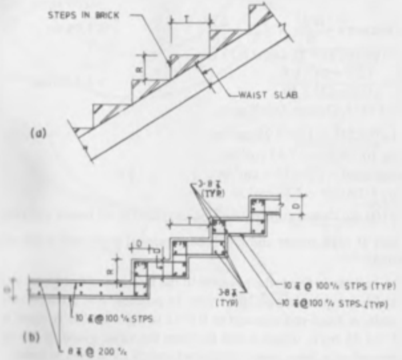


Fig. 4.11. Slabless stairs. (a) Common stairs (solid slab with steps in brick work); (b) slabless stairs.

4.5.1 Example of Slabless Stairs[20]

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

$$M = + \frac{W l^2}{8}$$

Referring to Fig. 4.11b,
$T = 0.25$ m, $R = 0.15$ m, $D = .15$ m, $L = 14 \times 0.25 = 3.5$ m
Selfweight of slabless stairs = P/T

$P = 0.15(.15 + .15) \times 1.0 \times 2500$	= 112.5 kg/m
$+ (.25 - .15) \times .15 \times 1.0 \times 2500$	= 37.5 kg/m
	150.0 kg/m
$\frac{P}{T} = \frac{150}{0.25}$	= 600 kg/m^2
W: Selfwight of slabless stairs	= 600 kg/m^2
Finish $.05 \times 2500$	= 125 kg/m^2
Plaster	= 25 kg/m^2
DL	= 750 kg/m^2
LL	= 500 kg/m^2
W	=1250 kg/m^2 or 1.25 t/m^2

Moment at centre $= \frac{W L^2}{8} = 1.25 \times \frac{3.5^2}{8}$ = 1.91 tm

With $b = 100$ cm, $D = 15$ cm, $d = 13$ cm,

$K = \frac{M u}{b d^2} = \frac{1.5 \times 1.91 \times 10^5}{100 \times 13^2 \times 10}$ = 1.7 N/mm^2

For M15, Fe415, Design Aids[58] give,

$$A_{st} = 0.558 \times 13 = 7.25 \text{ cm}^2/\text{m}$$

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

10/100 c/c closed stirrups shall be provided in all treads and risers with 3 ϕ 8 bars at each corner 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 27600 lb 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 $Wl^2/12$ in our example is equal to $1.25 \times 3.5^2/12 = 1.28$ tm/n, which is not far from the value given in the paper.[20] The author prefers to have simply supported ends for this type of stairs.

4.6 HELICOIDAL STAIRS

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 literature[22,23] 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 suggested in reference 22, is that the helicoidal stairs may be analysed as a beam circular in plan, for which the methods of Bergman[24] and Salvadori[25] are easily 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 helicoidal stairs has to be designed for vertical moment, vertical shear, torque and a horizontal moment. It is a space structure and it is mandatory for reasons of equilibrium, to design this structure with both ends fixed. An example of helicoidal stairs is given below to illustrate the above ideas.

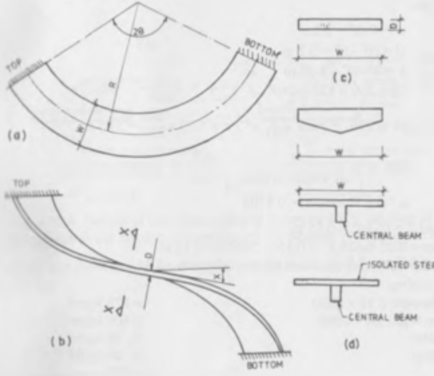


Fig. 4.12. Helicoidal stairs. (a) Plan of helicoidal stairs; (b) Elevation of stairs; (c) Section X-X; (d) Alternative sections of stairs.

In Indian Institute of Technology, Delhi, these stairs have been used in some places. A full circular helicoidal stairs with the central beam supporting isolated steps has been 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, a semi-circular helicoidal stairs has been provided with a cross-section of variable thickness, giving less thickness on the outside and more thickness in the centre, for a better view (Fig. 4.12d). Because of the complex nature of forces acting on sections of a helicoidal stairs, it is advisable not to use slabless stairs in these cases.

4.6.1. Example of Helicoidal Stairs

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

a. Data

b = 4'-0" = 1.2 m

H = 10' - 6" = 3.2 m

R = 4' - 6" = 1.35 m

$2\beta = 360 - 120 = 240°$

$$\text{arc length in Plan} = 2\pi R \frac{2\beta}{360} = 2\pi \times 1.35 \cdot \frac{240}{360} = 5.65 \text{ m}$$

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

$\alpha = 29.5°$, $\cos\alpha = 0.8704$

Tread T = 0'-10" = 0.25 m

Riser = 0.25 tan29.5° = 0.25 × .56637 = 0.15 m

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

b. *Loading*

Selfweight 0.15 × 2500		= 375 kg/m²
4 cm finish .04 × 2500		= 100 kg/m²
Plaster		= 25 kg/m²
Railing		= 25 kg/m²
	DL	= 525/cos α
		$= \frac{525}{0.8704}$ = 600 km/m²
	LL	= 300 kg/m²
	W	= 900 kg/m²
		= 0.9 t/m² of plan area
$W_1 = Wb = 0.9 \times 1.2$		= 1.08 t/m
$W_1 R^2 = 1.08 \times (1.35)^2$		= 2.0

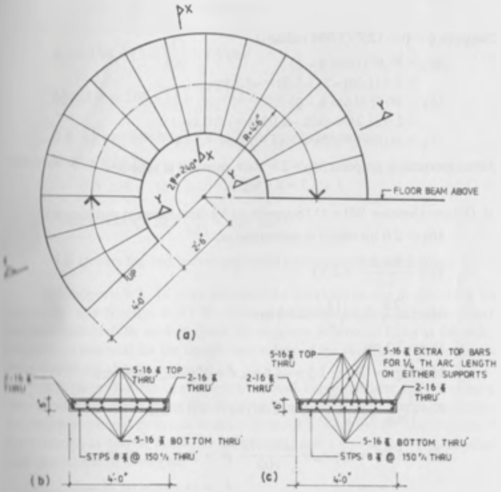


Fig. 4.12A. Details of helicoidal stairs of Example 4.6.1. (a) Plan of helicoidal stairs); (b) Section X-X (midspan); (c) Section Y-Y (support TS).

c. *Analysis as a fixed curved beam by Bergman's method*:[24] From Table 10.1 (Ref. 24), for

$$\frac{b}{h} = \frac{1.2}{.15} = 8$$

$$K = 0.64$$

From Fig. 10.4 (Ref. 24), for

$$K = 0.64 \text{ and } \beta = 120° , U = 1.28$$

Midspan $\phi = 0$

$$Mr_c = W_1R^2(U-1)$$

$$= 2.0\,(1.28 - 1.0) = 0.56 \text{ tm}$$

$Mt_c = 0$

$V_c = 0$

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

$$Mr_\phi = W_1R^2 (U\cos\phi - 1)$$
$$= 2.0\,(1.28(-.5) - 1.0) = -3.3 \text{ tm}$$
$$Mt_\phi = W_1R^2 (U\sin\phi - \phi)$$
$$= 2.0\,(1.28 \times .866 - 2.094) = -2.0 \text{ tm}$$
$$V_\phi = W_1R\phi = 1.08 \times 1.35 \times 2.094 = 3.1 \text{ t}$$

Horz. moment at supports $M_H = 2 \times$ Vert. moment at supports

$$2 \times 3.3 = 6.6 \text{ tm}$$

d. *Design*: (Section 120 × 15) Support: (i) Vertical moment and torque

$Mt_\phi =$ 2.0 tm which is equivalent to,

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

$$M_e = \pm\frac{2.0}{1.7}\left(1 + \frac{.15}{1.2}\right) = \pm 1.3 \text{ tm}$$

$M_r = -3.3$ tm

M design $= -3.3 \pm 1.3 = -4.6$ tm, $b = 120$ cm, $d = 12$ cm

$$K = \frac{1.5 \times M \times 10^5}{120 \times 12^2 \times 10} = 0.868\,M\,(\text{tm}) = 4.0 \text{ N/mm}^2$$

$$d^1/d = 3/12 = .25,\ A_{st} = \frac{120 \times 12}{100} \text{ pt} = 14.4 \text{ pt} = 19.87 \text{ cm}^2$$

$$A_{sc} = 14.4 \text{ pc} = 10.37 \text{ cm}^2$$

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

Bott. bars 5 nos. 16 bars = 10.05 cm^2

(ii) Shear = 3.1 + 2.7 = 5.8 t

$$T_v = \frac{1.5 \times 5.8 \times 1000}{120 \times 12 \times 10} = 0.60 \text{ N/mm}^2$$

For $p_t = 1.4$, $T_c = 0.67$ N/mm2 , Safe

Min $\frac{V_{us}}{d} = .0348\,b$ or d whichever is less

$= .0348 \times 12 = .42$ KN/cm

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

(iii) Horz. moment $M_H = 6.6$ tm, $b = 15$ cm, $d = 117$ cm

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

$$A_{st} = .144 \times 15 \times \frac{117}{100} = 2.5 \text{ cm}^2$$

$$\text{Min Ast} = .205 \times 15 \times \frac{117}{100} = 3.6 \text{ cm}^2$$

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

Midspan: $M_t = 0.56$ tm

$$K = .868 \times .56 = 0.49 \text{ N/mm}^2$$

$$A_{st} = .144 \times 120 \times \frac{12}{100} = 2.07 \text{ cm}^2$$

5 ϕ 16 bars top and bottom provided for convenience in detailing.

On supports 5 ϕ 16 extra bars shall be provided on top to give 10 ϕ 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[22] and Cusens,[23] for economy in design. It may be noted that Cusen's curves save considerable time and labour of a designer.

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. 4.13). 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 both the 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 small, plane frame analysis can be made to apply. There are excellent papers[26-31] which give different approaches for the analysis of this neat structure. The approach given here is summarized below.

i) 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.

4.14a). The two parts of the structure are A-B-C and A'-B'-C' (Fig.4.13). We then find the reaction R_B at the fictitious support.

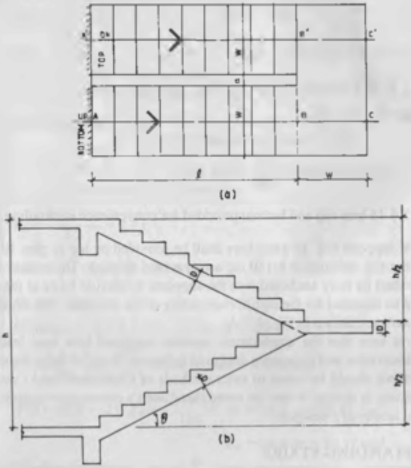


Fig. 4.13. Free-standing stairs. (a) Plan of free-standing stairs; (b) elevation of free-standing stairs.

ii) 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. 4.14b).

iii) 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. 4.13). This aspect causes horizontal moments in flights (Fig. 4.14c).

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 of a, the clear distance between the flights in plan is to be made as less as practicable, say 150 mm to 300 mm. When the distance a is required to be made substantial or a three-flight free-standing stairs are to be planned (Fig. 4.15), 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. 4.15) with a central subtended angle 2β can be considered for analysis of 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 free-standing stairs is given below.

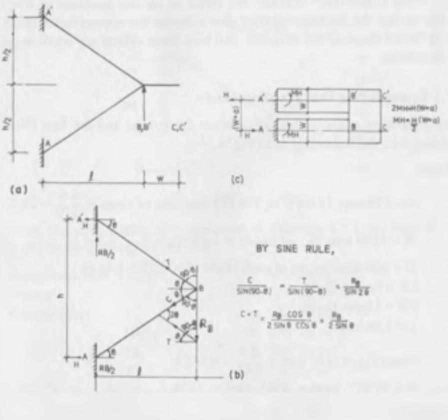


Fig. 4.14. Analysis of free-stadning stairs. (a) Fictitious support at line B-B′; (b) Truss action of flights; (c) Horizontal moments in flights.

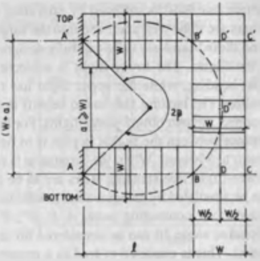


Fig. 4.15. Three-flight free-standing stairs.

Some authorities[26] consider the effect of various positions of live load on flights and the landing and they also consider the second-order effects of the deflected shape of the structure. But both these effects are small in practical structures.

4.7.1 Example of a Free-standing Stairs

A free-standing stairs is planned between the ground and the first floor of a building with the following data (Fig. 4.16).

a. *Data*

$R = 179$ mm, $H = 4.6$ m, $T = 275$ mm. No. of risers $= \frac{4.6}{179} = 25.7 = 26$

$W = 1200$ mm, No. of treads in each flight $= \frac{26}{2} + 1 = 14$

$D = 200$ mm, length of each flight $14 \times .275 = 3.85$ m

LF = Lower flight

UF = Upper flight

L = Landing

From Fig. 4.16b, $\tan\theta = \frac{2.3}{3.85} = 0.5974$

$\theta = 30.85°$, $\cos\theta = .8585$, $\sin\theta = .5128$

b. *Loading*

L. Selfweight 0.20×2500 = 500 kg/m^2

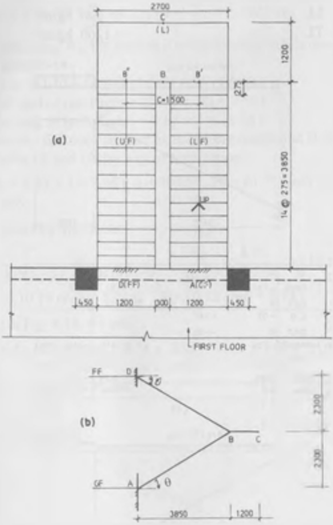
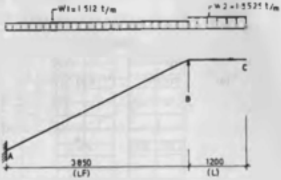


Fig. 4.16. Planning details of a free-stairs of Example 4.7.1. (a) Plan of stairs; (b) Skeletal elevation of stairs.

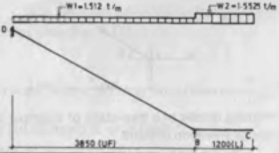
4 cm finish 0.04 × 2500	=	100 kg/m²
Plaster	=	25 kg/m²
Railing (say)	=	25 kg/m²
DL	=	650 kg/m²
LL	=	500 kg/m²
TL	=	1150 kg/m²
LF, UF : Selfweight .20 × 2500	=	500 kg/m²
4 cm finish	=	100 kg/m²
Plaster	=	100 kg/m²
Railing	=	25 kg/m²

$$
\begin{array}{lll}
DL & \dfrac{650}{\cos\theta} = \dfrac{650}{.8585} & = 760 \text{ kg/m}^2 \\
LL & & = \underline{500} \text{ kg/m}^2 \\
TL & & = \underline{1260} \text{ kg/m}^2
\end{array}
$$



DF	0		1.0	—	
FEMS	−1.87		+1.87	−1.12	
DIST	0		−.75		
C.O	−.37	+.38	0		
DIST	0	−.68			
FINAL MOMENTS	−2.24	+1.13	+1.12	−1.12	(tm)
SHEAR	2.91		2.5	1.86	
WT EFFECT	0.29		0.29	–	
FINAL SHEAR	3.2		2.62	1.86	(t)

(a)



FINAL MOMENTS	−2.24	+1.13	−1.12	−1.12	(tm)
FINAL SHEARS	3.2		2.62	1.86	(t)

(b)

Fig. 4.17. Bending analysis of free-standing stairs by moment distribution. (a) Analysis of flight A-B-C; (b) Analysis of flight D-B-C.

c. *Analysis*

i) Assume a fictitious support at B and analyse two continuous beams ABC and DBC by moment distribution as given in Fig. 4.17a, b.

$$W_1 = 1.26 \times 1.2 = 1.512 \text{ t/m}$$

$$W_2 = 1.15\,(W + \frac{a}{2}) = 1.15(1.2 + \frac{.3}{2}) = 1.5525 \text{ t/m}$$

In evaluating W_2, the portion a of 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 = 15.36 t

ii) Remove the fictitious support at B and the reaction at B is applied at B to be resisted by LF and UF by way of truss action.

$R_B = 4.48 \times 2 = 8.96$ t, $\theta = 30.85°$, $2\theta = 61.7°$, $\cos\theta = .8585$, $\sin 2\theta = .8805$

Fig. 4.18 gives by Sine Rule.

$$\frac{C}{\sin(90-\theta)} = \frac{T}{\sin(90-\theta)} = \frac{R_B}{\sin 2\theta} = \frac{8.96}{.8805} = 10.18 \text{ t}$$

$C = 10.18\cos\theta = 8.74$ t, $\quad T = 10.18\cos\theta = 8.74$ t

Referring to Fig. 4.18, we get,

At A : $H' = C\cos\theta = 8.74 \times .8585$	$= 7.50$ t
$V' = C\sin\theta = 8.74 \times .5128$	$= 4.48$ t
At D : $H'' = T\cos\theta$	$= 7.5$ t
$V'' = T\sin\theta$	$= 4.48$ t
Total $V' + V''$	$= 4.48 + 4.48 = 8.96$ t

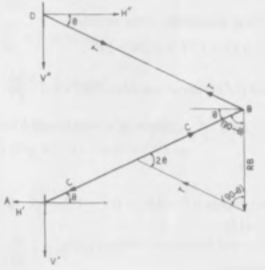


Fig. 4.18. Truss analysis of free-standing stairs by Sine Rule.

Total shear at supports = 3.2 + 3.2 = 6.4 t

Check : Total load = 8.96 + 6.4 = 15.36 t

which is the same as the total load of both the flights.

iii) In the truss system of step (ii), point B is not located at one point in plan. Point B is actually located at two points B' and B" in plan (Fig. 4.16). This causes horizontal moments in flights (LF and UF). Considering an overall equilibrium of the structure as a whole (Fig. 4.19),

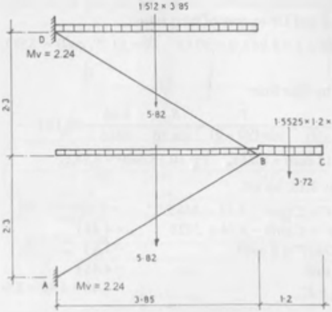


Fig. 4.19. Overall equlibrium of the structure.

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

or $V = 7.68$ t

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

$$+ 3.72\,(3.85 + \frac{1.2}{2}) - 2Mv = 22.4 + 16.6 = 39.0 - 4.48 = H \times 4.6$$

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

Horizontal moment in flights (LF or UF) = $H\frac{c}{2} = 7.5 \times \frac{1.5}{2} = 5.62$ tm

d. *Design* (M15, Fe415)

(i) *Bending of flights and landing in vertical plane*

A : (LF in compression)

$M_V = 2.24$ tm (Fig. 4.17a)

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

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

$= .072\ KP$

$= .072 \times 1 \times 8.74 = 0.63$ tm

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

Section 120×20 $b = 120$ cm $D = 20$ cm

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

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

$\frac{d'}{D} = \frac{2.4}{20} = .12 \approx .15$

For $A_{sc} \neq A_{st}$, using Manual[17] chart No. 7.7, we get

$\frac{d}{D} = .85$, Fe415, $X = 0$, $n = .3$, $\frac{r}{f_{ck}} = .2 \times 10^{-3}$

$r = .2 \times 15 \times 10^{-3} = 0.003$

$A_s = .003 \times 120 \times 20 = 7.2\ cm^2$, $A_{st} = 7.2\ cm^2$, $A_{sc} = 0$

Provide 10 Nos. $\bar{\phi}$ 10 bars, @ top give 7.85 cm²

Or, using Design Aids,[58]

$K = \frac{1.5 \times 2.87 \times 10^5}{120 \times 17.6^2 \times 10} = 1.16\ N/mm^2$

$A_{st} = .357 \times 120 \times \frac{17.6}{100} = 7.54\ cm^2$

Provide 10 Nos. $\bar{\phi}$ 10 bars, @ top = 7.85 cm²

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

$C = 8.74$ t

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

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

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

$r = .1 \times 10^{-3} \times 15 = .0015$

$A_s = .0015 \times 120 \times 20 = 3.6\ \text{cm}^2$

Provide 5 Nos. ϕ 10 bars, @ bottom = 3.92 cm²

B : $M_v = -1.12 - .63 = -1.75$ tm

$C = 8.74$ t

Provide 5 Nos. ϕ 10 bars, @ top (result as per A–B).

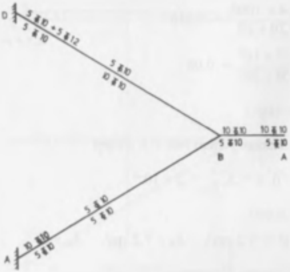


Fig. 4.20. Results of design showing top and bottom bars at critical sections.

Max shear = 3.2 t

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

$T_c = .35\ \text{N/mm}^2$ (considered as minimum)

$$\text{Min } \frac{V_{us}}{d} = .0348\ b \text{ or } d \text{ which ever is less}$$

$$= .0348 \times 18 = 0.626\ \text{KN/cm}$$

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)

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

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

Chart 7.7 of Manual[17] gives

$$\frac{r}{f_{ck}} = .25 \times 10^{-3}, X = 0$$

$$r = .25 \times 10^{-3} \times 15 = .00375$$

$$A_s = .00375 \times 120 \times 20 = 9.0 \text{ cm}^2$$

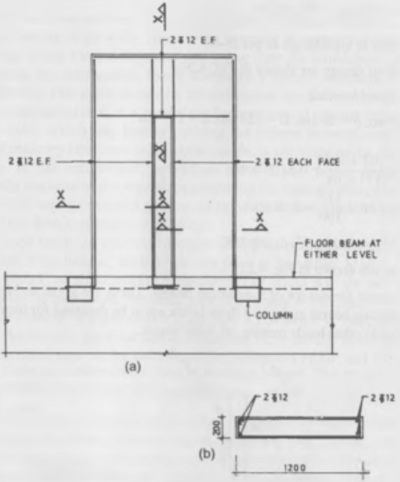


Fig.4.21. Plan of stairs showing extra bars for horizontal bending of flights. (a) Plan; (b) Section X-X.

5 ϕ 10 bars at top + 5 ϕ 12 bars at top

$3.92 + 5.65 = 9.57$ cm²

D–B : $M_v = +1.13$ tm $\quad T = -8.74$ tm

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

$$\frac{M_u}{f_{ck}\, bD^2} = 0.024$$

$$\frac{r}{f_{ck}} = .2 \times 10^{-3}$$

$$r = .003, \quad A_s = 7.2 \text{ cm}^2$$

10 $\bar{\phi}$ 10 bars at bottom give area of 7.85 cm²

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

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

The results of design are shown in Fig. 4.20.

(ii) *Horizontal bending*

$M_H = 5.62$ tm, $b = 20$ cm, $D = 120$ cm, $d = 117$ cm

$$K = \frac{1.5 \times 5.62 \times 10^5}{20 \times 117^2 \times 10} = 0.31 \text{ N/mm}^2$$

$$A_{st} = 0.09 \times 20 \times \frac{117}{100} = 2.10 \text{ cm}^2$$

2 $\bar{\phi}$ 12 E.F. = 2.26 cm², nearly OK.

These bars are shown in Fig. 4.21.

(iii) *Reactions for design of supporting beams.* These are given in Fig. 4.22. The supporting beams at the two floor levels are to be designed for these loads in addition to other loads coming on these beams.

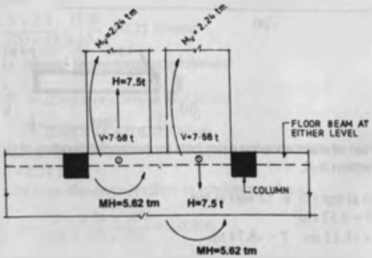


Fig. 4.22. Reactions due to stairs on the supporting beams at either level.

CHAPTER 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[12] and SP:20[32] 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 north India 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 × 125 mm × 75 mm, in Madhya Pradesh (Bhopal area) it is 200 mm × 100 mm × 75 mm, while in the rest of the country, the brick size is 230 × mm 115 × 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 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° to 60° 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.[12] The local pressure on a brick wall may be exceeded by 50%, this aspect is used to fix the size of a bed block given at the ends 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.[11]

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/cm^2, 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/cm^2 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. 5.1a, a part floor plan of a building is shown and it is desired to design the internal wall W1. In Fig. 5.1b, a section of wall W1 is given through the height of the building, which is a three storeyed building.

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

Roof

Self weight of slab 0.15 m thick = 0.15 × 2500			= 375 kg/m^2
Average 0.10 m thick lime terracing = .10 × 2000			= 200 kg/m^2
Water proofing (say)			= 50 kg/m^2
Brick tiles on top	= .05 × 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

Type Floor

Self weight of a slab .15 m thick = 0.15 × 2500		= 375 kg/m^2
Floor finish 50 mm thick = .05 × 2500		= 125 kg/m^2
Ceiling plaster		= 25 kg/m
	DL	= 575 kg/m^2
Live load for residential use	LL	= 200 kg/m^2
	TL	= 725 kg/m^2

b) 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

Load from roof slab S1 $= \frac{w\, l_x}{4}\left(2 - \frac{l_x}{l_y}\right)$

$= 900\, \frac{4.0}{4}\left(2 - \frac{4.0}{6.0}\right)$ = 1200 kg/m

Load from roof slab S2 $= 900\, \frac{5}{4}\left(2 - \frac{5}{6}\right)$ = 1313 kg/m

Self weight of wall = .23 × 3.0 × 1.0 × 1900 = 1311 kg/m

P_1 = 3824 kg/m

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 W1. So, for length of wall less openings = 6.0 – 2 × 1.0 = 4.0 m

Pressure in masonry wall taking

openings into account $p = \frac{3824}{23 \times 100} \times \frac{6.0}{4.0}$ = 2.49 kg/cm^2

Level 2-2

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

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

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

Self weight of wall = 1311 kg/m

P_2 = 3336 kg/m

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

$p = \frac{7160}{23 \times 100} \times \frac{6}{4}$ = 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

p $$p = \frac{10758}{23 \times 100} \times \frac{6}{4} = 7.02 \text{ kg/cm}^2$$

c) **Allowable pressure on masonry wall**

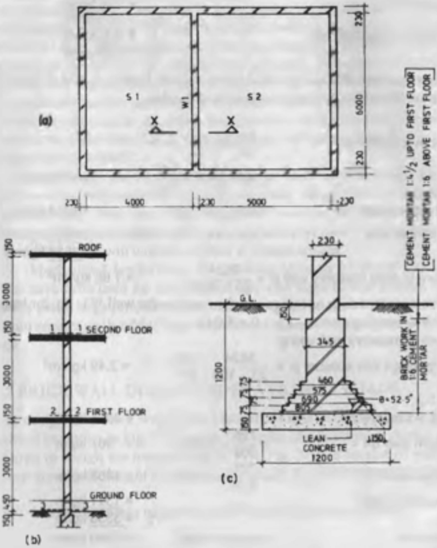


Fig. 5.1. Load bearing brick wall under vertical load. (a) Part plan of building; (b) Section X-X. (c) Details of footing for brick wall W1.

With the crushing strength of bricks to be 100 kg/cm2 and with 1 : 6 cement mortar IS: 1905[12] gives,

$$p_a = 8.1 \times Cr \text{ kg/cm}^2$$

where, C_r = reduction factor due to slenderness of wall

$$\frac{h}{t} = \frac{0.75\,H}{t} = .75\,\frac{(3.15 + .45 + .15)}{0.23} = 12.2$$

Cr = 0.75

$p_a = 8.1 \times .75 = 6.08$ kg/cm^2
H = storey height
h = effective height of wall
t = 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,

$$p_a = 9.6 \times 0.75 = 7.2 \text{ kg/cm}^2 > 7.02 \text{ kg/cm}^2$$

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

Level 2.2

$$p = 4.67 \text{ kg/m}^2$$

For 1 : 6 cement mortar

$$p_a = 8.1 \times \text{Cr}$$

$$\frac{h}{a} = \frac{.75 \times 3.15}{.23} = 10.27$$

$$C_r = .85$$

$$p_a = 8.1 \times .85 = 6.89 \text{ kg/m}^2 > 4.67 \text{ kg/m}^2$$

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.

d) Wall Footing

The load at level 3-3: ΣP = 10.758 kg/m
≈ 11.0 t/m

With a safe bearing capacity of soil at 10 t/m^2 with a foundation depth of 1.2 m below ground,

$B = \frac{11.0}{10} = 1.1$ 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° to 60° at all levels.

Pressure in brick work in foundation (on 345 mm thick wall) = $\frac{11000}{34.5 \times 100}$
= 3.19kg/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/cm^2 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[12]. Fine sand is used only in mortars used for plastering 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.

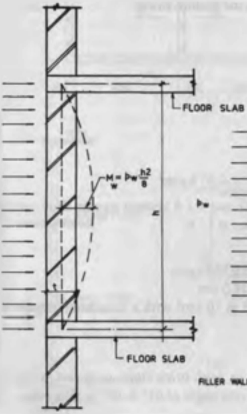


Fig. 5.2. Wind force on an external brick wall.

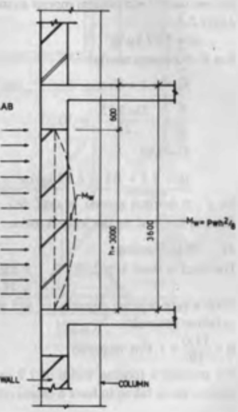


Fig. 5.3. Wind pressure on filler brick walls.

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[12] permits tensile stress of 1.0 kg/cm^2 in masonry which may be increased by 33.3% when wind effect is included.

Referring to Fig.5.2, $p_w = 150 \times .75 = 112.5$ kg/m^2

$h = 4.0$ m, $t = 23$ cm

$$M_w = 112.5 \times \frac{4.0^2}{8} \qquad = 225 \text{ kgm}$$

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

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

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

$f = f_d \pm f_t = 2.17 \pm 2.55 = 4.72, -0.38$ 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

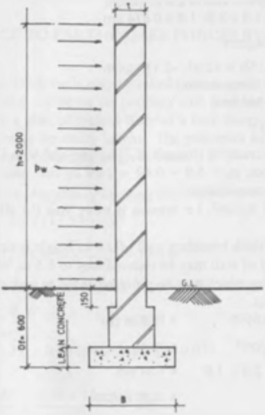


Fig. 5.4. Wind pressure on boundary wall.

When the external walls are used as filler walls in a framed building, the wall can be checked as follows (Fig. 5.3).

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

$$f_t = \pm\frac{127 \times 100}{8817} = \pm 1.44 \text{ kg/cm}^2$$

p = load of self-weight of wall of $h/2$ height

$= .23 \times 1.5 \times 1.0 \times 1.9 = 0.66$ t/m

$f_d = \frac{660}{23 \times 100} = 0.29$ kg/cm^2

$f = 0.29 \pm 1.44 = 1.73, -1.15$ kg/cm^2

The tensile stress 1.15 kg/cm^2 less 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. 5.4).

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

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

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

$= .23 \times 1.0 \times 2.0 \times 1.9 = 0.874$ t/m

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

$f = f_1 + f_d = 0.38 \pm 2.55 = +2.93, -2.17$ kg/cm^2

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

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

$\frac{h}{t} = \frac{2 \times 2.15}{.23} = 18.7$

For bricks of crushing strength of 75 kg/cm^2 and with 1 : 6 cement mortar, IS: 1905[12] gives, $p_a = 5.9 \times 0.49 = 2.89$ kg/cm^2 nearly 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.

With, $z = 100 \times (34.5)^2/6 = 19{,}838$ cm^3

$f_1 = \pm\frac{225 \times 100}{19838}$ kg/cm^2 $= \pm 1.13$

$p = = .345 \times 1.0 \cdot 2.0 \times 1.9 = 1.31$ t/m

$f_d = \frac{1310}{345 \times 100} = 0.38$ kg/cm^2

$f = .38 \pm 1.13 = +1.51, -0.75$ kg/cm^2

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. 5.5).

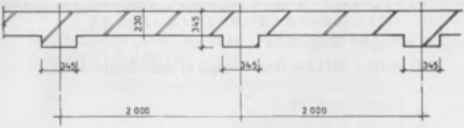


Fig. 5.5. Plan of long boundary wall with pilasters.

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 earthquake 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.

a. *Wind force on long face of building* (IS: 875)[2]

$$p_u = 150 \times .75 = 112.5 \text{ kg/cm}^2$$

$$P_w = 10.0(12.45 + .9) \times \frac{112.5}{1000} = 15.0 \text{ t}$$

b. *Earthquake base shear* (V_B) IS: 1893[9] *gives,*

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

$$\alpha_h = \beta.I.\alpha_0$$

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

$$K = 1.0$$

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

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

$$V_B = 1.0 \times .075 \times 1.0 \times W = 0.075\,W$$

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

$$W_T = D_L \text{ on roof} + \text{weight of half height of walls} + W_t \text{ of parapet wall}$$
$$= .75 \times 10.0 \times 5.0 + .23\,(.9 + 1.5)\,(20 + 20) \times 1.9$$
$$= 37.5 + 42.0 = 79.5 \text{ t}$$

$$W_f = (D_L + .25LL) \text{ on floor} + \text{weight of storey height of walls}$$
$$= (.525 + .25 \times .2)\,10 \times 5 + .23 \times 3.3 \times 40 \times 1.9$$
$$= 28.8 + 52.4 = 81.2 \text{ t}$$

$$W = 79.2 + 3 \times 81.2 = 322.8 \text{ t}$$

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

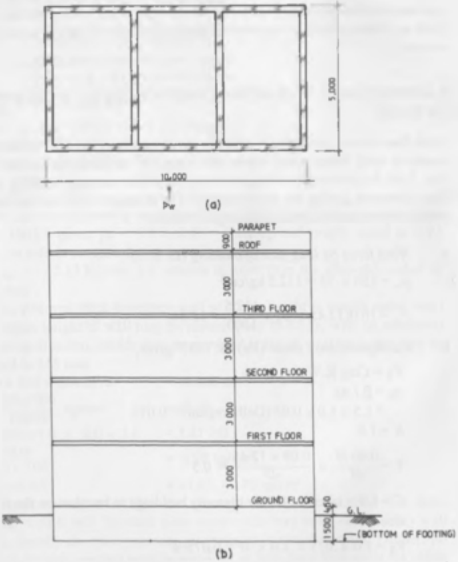


Fig. 5.6. Typical floor plan of a 4-storeyed building. (a) Plan of a typical floor; (b) Elevation of building.

So, earthquake governs the design.

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

$$Q_i = V_B \frac{W_i h_i^2}{\sum (w_i h_i^2)} \; ; \; V_B = 24.2 \text{ t}$$

Table 5.1 Calculations for Q_i acting at floor levels.

Level	W_i (t)	h_i (m)	$W_i h_i 2$	$\frac{W_i \cdot h_i^2}{\Sigma W_i h_i^2}$	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
Base of footing		0	0 –	–	
Σ	323.1	–	32387	1.00	24.2

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 = $0.23(20 + 20) = 9.2 \text{ m}^2$

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

$= 0.263 \text{ kg/cm}^2$

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

Stresses due to OTM $f_t = \pm \frac{M}{z}$

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

$$z = (10.0 \times \frac{5.0^2}{6} - 9.54 \times \frac{4.54^2}{6}) + 2 \times .23 \times \frac{5.0^2}{6}$$

$= (41.67 - 32.77) + 1.9$

$= 8.9 + 1.9 = 10.8 \text{ m}^3$

$$f_t = \pm \frac{280}{10.8} = \pm 25.9 \text{ t/m}^2 = \pm 2.59 \text{ kg/cm}^2$$

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,

$$f_d = \frac{0.23 \times 4.95 \times 1.0 \times 1.9 \times 1000}{23 \times 100} = 0.94 \text{ kg/cm}^2$$

$$f = f_d + f_t = 0.94 \pm 2.59 = 3.63, -1.65 \text{ kg/m}^2$$

the tensile stress is 1.65 kg/m^2 which is more than the allowable value of 1.0 × 1.33 = 1.33 kg/cm^2.

If these are load bearing walls, the

$$f_d = \frac{323.1}{40 \times 0.23} = 35.1 \text{ t/m}^2$$

$$= 3.51 \text{ kg/m}^2$$

$$f = f_d + f_t = 3.51 \pm 2.59 = +6.1\ ,\ +0.92 \text{ kg/m}^2$$

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 resist earthquake 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 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.

CHAPTER 6

Framed Buildings under Vertical Loads

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 may act 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% to 20%. It is, thus, important to note that dead load is the most significant of all loads. Dead and live loads are both gravity or vertical loads and these produce similar 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 vertical 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.

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, reinforced 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 × 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.

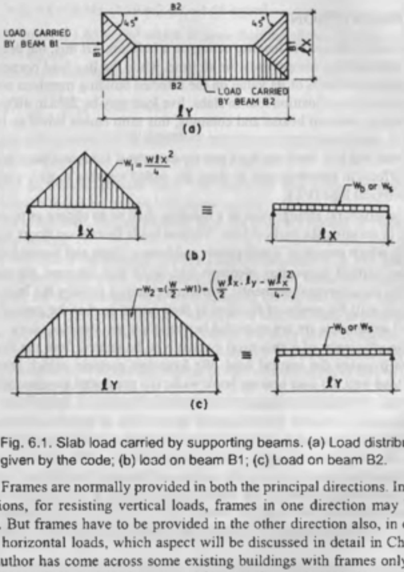


Fig. 6.1. Slab load carried by supporting beams. (a) Load distribution as given by the code; (b) load on beam B1; (c) Load on beam B2.

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.

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:

(i) 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 the Code. Referring to Fig. 6.1,

for beam B1 : $W_b = W \dfrac{lx}{3}$ (6.1)

For beam B2 : $W_b = W \dfrac{lx}{6} [3 - (\dfrac{lx}{ly})^2]$ (6.2)

where W = slab loading in t/m^2

By using these values of W_b, we get beam shears, which are more than the 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),

for beam B1 : $W_s = W \dfrac{lx}{4}$ (6.3)

for beam B2 : $W_s = W \dfrac{lx}{4} (2 - \dfrac{lx}{ly})$ (6.4)

With these values of W_s, 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 also column loads all will work out exactly.

(ii) In the analysis, moment of inertia of beams shall be calculated taking the effect of T-action of slab into account. Its necessity has been nicely explained by Jain and Jaikrishna (p.2, Vol. II).[5] In many computer programs, only rectangular section of beam is considered and the T-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.

(iii) 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.

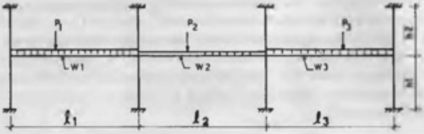


Fig. 6.2. Substitute frame method for a floor beam.

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 span, 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 and shears in beams and in the design results, we get steel areas at the supports and the midspan sections of spans with V_{us}/d-values for fixing stirrups in beams. 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 away from the column face and this aspect has been considered in the computer program in use.

6.4 INTERACTION AT JUNCTION OF REINFORCED CONCRETE ELEMENTS

In the earlier practice,[3] 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, 230 mm × 230 mm and the beam size is large, say, 230mm × 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. 6.3a 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 $wl^2/24$ 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. 6.3b gives a junction of a beam B1, resting on the supporting beam B2 at its ends. The beam B1 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 B1.

Fig. 6.3c 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 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. 6.3a, b are subsidiary interactions.

Fig. 6.3d gives the column-footing junction. The columns in buildings

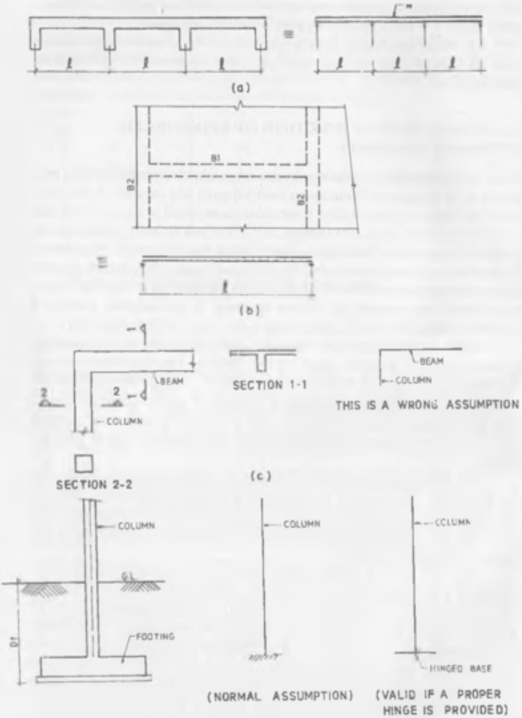


Fig. 6.3. Interaction of RC elements at junctions. (a) Slab and beam junctions; (b) Plan of beam tobeam junction; (c) Column-beam junction; (d) Column footing junctions.

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.

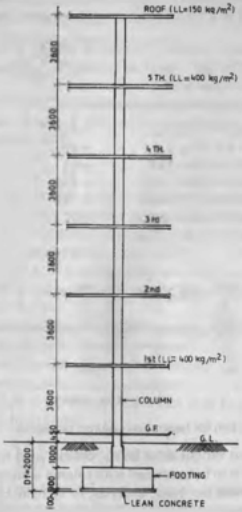


Fig. 6.4. Elevation of an example column.

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.[2] 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 × 6.0 m = 36.0 m². The following data are given from the frame analysis.

Roof: TL = 40.0 t, LL = 36 × 0.15 = 5.4 t, DL = 40.0 – 5.4 = 34.6 t

Typ. floor: TL = 52.9 t, LL = 36 × 0.4 = 14.4 t, DL = 52.9 – 14.4 = 38.5 t (1st to 5th)

The design column loads are given as follows:

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

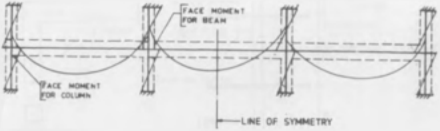


Fig. 6.5. Face correction for beam and column moments.

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,

$$e_{min} = \frac{l}{500} + \frac{(b \text{ or } D)}{30} \quad (\nless 2.0 \text{ cm}) \qquad (6.5)$$

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[34] and its commentary[35] explain that the minimum eccentricity moments shall be taken about one axis at a time and these should not be applied about both the axes 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,

$$M_{a,b} = \frac{K\,P\,b}{2000}\left(\frac{l_{ef}}{b}\right)^2 \qquad (6.6)$$

where l_{ef} = effective height of column (= 1.2 l)

l = clear height of column in a given storey

P = column load

$$k = \frac{P_{uz} - P_u}{P_{uz} - P_b} \quad (\leq 1)$$

= 0.5 for heavily loaded column ($P_u/f_{ck}bD > 0.4$)

= 1.0 for lightly loaded column ($P_u/f_{ck}bD \leq 0.4$)

$P_{uz} = 0.45\, f_{ck}\, A_c + 0.75\, f_y . A_{sc}$

P_u = ultimate column load (= 1.5 P)

P_b = axial load corresponding to condition of maximum compressive strain of 0.0035 in concrete and tensile strain of 0.002 in outermost layer of tension steel.

f_{ck} = characteristic compresive strength of concrete

f_y = characteristic strength of steel

A_c = net concrete area of column section

A_{sc} = area of steel in column section

When $l_{ef}/b \leq 12$, slenderness effect is to be neglected. However, when $l_{ef}/b = 12$, Eq. (6.6) gives

$$M_{a,b} = \frac{k\,P\,b}{2000} \times (12)^2 = 0.072\ k.P.b \qquad (6.7)$$

This value of the additional moment due to slenderness with $l_{ef}/b = 12$ is also substantial, but it is allowed by the Code to be neglected. For $k = 0.5$, $P = 274$ t, $b = 0.3$ m, Eq. (6.7) gives

$$M_{a,b} = 0.072 \times 0.5 \times 274 \times 0.3 = 2.96 \text{ tm}$$

when $lef/b = 6$, Eq. (6.6) gives,

$$M_{a,b} = kPb(6)^2/2000 = 0.018\ kPb \qquad (6.8)$$

For $k = 0.5$, $P = 274$ t, $b = 0.3$ m

$$M_{a,b} = 0.018 \times 0.5 \times 274 \times 0.3 = 0.74 \text{ tm}$$

which is small enough to be neglected. Strictly speaking,

$M_{a,b} = 0$ only, 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.,[36] that hanger load should be increased by 33.33% to take care of the 'flutter' phenomenon. Further, tensile stress in concrete is to 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 super-structure, then column loads can be calculated by the tributary area (T.A.) method.[37] On each floor, dead and live loads are calculated for a column and live load reduction factors are applied to get the design load at each 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 not considered. 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 floor loads are calculated as follows:

Roof slab loading:	DL	= 750 kg/m²
	LL	= 150 kg/m²
Typical floor slab loading:	DL	= 625 kg/m²
	LL	= 400 kg/m²

Roof : DL : slab 36 × 0.75 = 27.0 (t) LL : 36.0 × 0.15 = 5.4 t
(0.23 × 0.45 × 1.0 × 2.5)
= 0.26 t/m beam 0.26 (6 + 6) = 3.1 t
parapet = –
(0.45×.45×3.6×2.5) Self wt. of col. = 1.8 t
DL = 31.9 t LL = 5.4 t

Typical floor : DL : slab 36 × 0.625 = 22.5 t LL : 36 0.4 = 14.4 t
beams = 3.1 t
(.115×1.0×3.0×1.9) 115 tk. walls .65 (6+6) = 7.8 t
Self wt. of column = 1.8 t
DL = 35.2 t LL = 14.4 t

Level	DL + *k*.LL	= TL × 1.05	= *P*(*t*)
Roof	31.9 + 1.0 × 5.4	= 37.3 × 1.05	= 39
5th fl.	67.1 + 0.9 × 19.8	= 84.9 × 1.05	= 89
4th fl.	102.3 + 0.8 × 34.2	= 129.7 × 1.05	= 136

3rd fl.	137.5 + 0.7 × 48.6	= 171.5 × 1.05	= 180
2nd fl.	172.7 + 0.6 × 63.0	= 210.5 × 1.05	= 221
1st fl.	207.9 + 0.6 × 77.4	= 254.3 × 1.05	= 267
GF	267 t		

Column moments at each floor level can be calculated by using Table IX 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 (whichever are greater) are used in column design.

6.7 AN EXAMPLE BUILDING

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 115 mm 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/m².

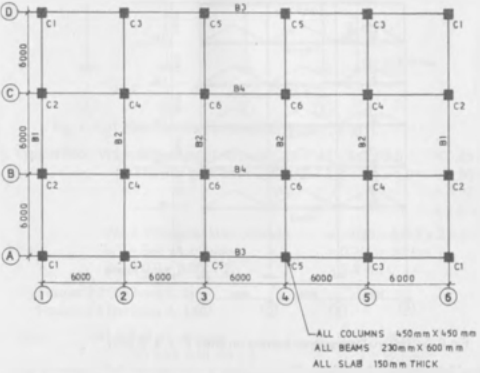


Fig. 6.6. Plan of the example building of six storeys.

General Loadings

Roof slab:	Self weight 0.15 × 2500		= 375 kg/m^2
	Lime terracing 0.10 × 2000		= 200 "
	Water proofing		= 50 "
	Ceiling plaster		= 25 "
	Brick lining		= 100 "
		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 "
	Plaster		= 25 "
	Light partitions		= 100 "
		DL	= 625 kg/m^2
		LL	= 400 "
		TL	=1025 kg/m^2

We wish to design this building by the following four methods and compare the column loads and moments in the base storey.

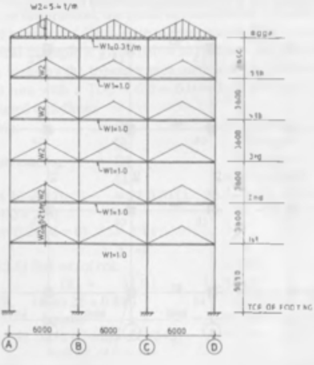


Fig.6.7a. Internal transverse frames on lines 2, 3, 4, 5 (B2).

Method I (*Sections* 6.2 *and* 6.5). The building is divided into full frames in the two principal directions. F1 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:

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

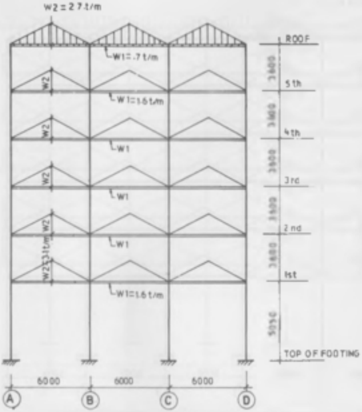


Fig. 6.7(b). End transverse frames on lines 1, 6 (B1).

Typical floor: W1 = Selfweight of rib-beam .23 × .45 × 1.0 × 2.5 = 0.26 t/m
115 mm thick brick wall .115 × 3.0 × 1.0 × 1.9 = 0.66 t/m
W_1 = 0.92 t/m
≈ 1.0 t/m

W2 = Trinagular load ordinate = 1.025 × 6.0/2 × 2 = 6.2 t/m
Roof: W1 = Sefl wt. of beam = 0.26 or .30 t/m
W2 = 0.9 × 6.0/2 × 2 = 5.4 t/m

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

Floor: W1 = Self wt. of beam = .26 t/m
230 thick wall .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	= .26 t/m
	parapet .23 × 1.0 × 1.0 × 1.9	= .44 t/m
		.70 t/m
	W2 = .9 × 3.0	= 2.7 t/m

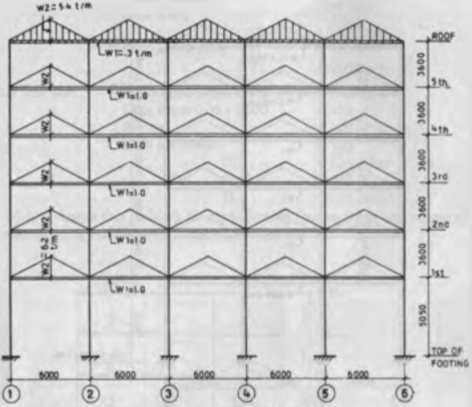


Fig.6.7(c). Longitudinal frames on lines B and C (B4).

Computer analysis of the four frames F1 to F4 is done and column loads and moments for columns C1 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 the roof and floor levels. Beams B1 to B4 (Fig. 6.6) are analysed by computer at the roof, typical and the first floor levels separately under the loading W_1, W_2 as found in Method I. The column loads for colums C1 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 (*Section* 6.6). The column loads are found by the T.A. method and the column moments are calculated by using Table IX of IS:456-1964,[3] considering the effect of slab flange on the moment of inertia of beams.

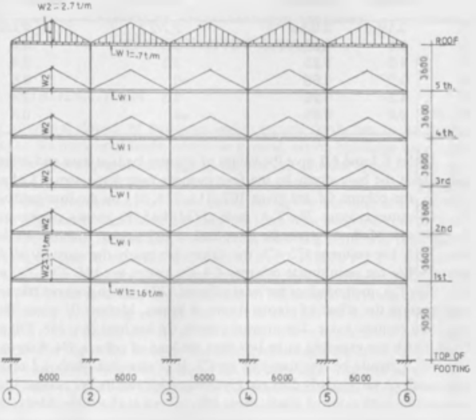


Fig. 6.7(d). End longitudinal frames on lines A and D (B3).

Table 6.1. Comparison of column loads at base by different methods of analysis.

Column Mark	Column load at base (*t*)			
	Method I	Method II	Method III	Method IV
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

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 Method II. The column loads and moments at base are found.

Table 6.2. Comparison of column moments in the base storey by diferent methods of analysis.

Column	Column moments in base storey (t_m) mark			
	Method I	Method II	Method III	Method IV
C1	3.1/4.8	3.8/3.8	2.7/2.7	2.8/2.8
C2	3.6	5.2	3.3	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

Tables 6.1 and 6.2 give the values of column loads at base and column moments in the base storey by the four methods considered above. Column loads for the column C1 are given 107, 111, 115, 111 by the four methods. This is a corner 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 111 t. For columns C2, C3, the values are nearly the same by all the methods. For the penultimate columns C4, the values are 284, 275, 270 and 280 t. The T.A. method gives the least value of 270 t, which does not take into consideration the effect of elastic shears of beams. Method IV gives 280 t which is a realistic value. The internal column C6 has load 284, 268, 270 and 271 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 II and 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 6.2 compares the column moment in the base storey for columns C1 to C6. In internal columns C4, C6 moments are negligibly small and the minimum eccentricity moments will govern the design. For corner columns C1 and external facade columns C2, C3, C5, Methods III and IVgive realistic values of column moments as both those metheds 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.

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.

CHAPTER 7

Framed Buildings under Horizontal Loads

7.1. INTRODUCTION

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 the strict 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 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.

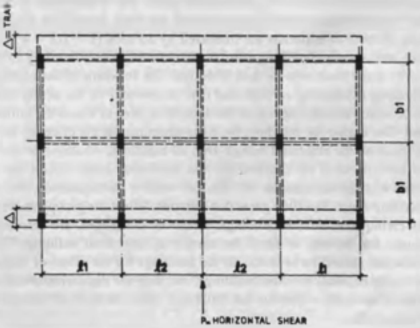


Fig. 7.1. Floor diaphragm under translation.

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).[9,11]

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 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.

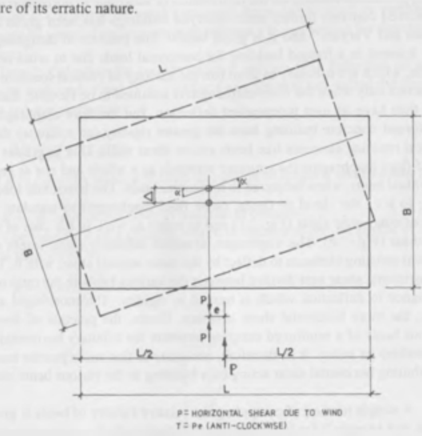


Fig. 7.2. Plan of floor diaphragm under translation and rotation. P = horizontal shear due to wind; $T = P_e$ (anti-clockwise).

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[38] 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 a reinforced 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 their resistance to deflection which is termed as rigidity. The more rigid an 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 by Cross and Morgan[39] 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 are available in literature[40,41,42] 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.[43]

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 earthquake and locate its point of application with reference to any bent in the direction of the force.

Step 2: Calculate the rigidity of each column. The deflection of a column with both ends fixed (Fig. 7.3) for a unit shear at top, called the flexibility of a column is given by

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

where $K = I/h$ = the relative stiffnes of column of height h and moment of inertia I

E = modulus of elasticity of material of column.

The rigidity R of a column, which is defined as the reciprocal of flexibility is then given by,

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

Fig. 7.3. Rigidity of a column.

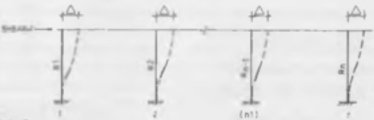


Fig. 7.4. System in parallel.

Step 3: 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

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

For a system 'in series' (Fig. 7.5), where the applied shear remains the same in all the constituents, the flexibility of a bent equals the sum of flexibilities of the constituent columns. This gives,

$$\Delta = \Delta_1 + \Delta_2 + \ldots + \Delta_n \tag{7.4}$$

The rigidity of such a bent is given by,

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

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. 7.6 gives an example for calculating the rigidity of a bent employing parallel and series combinations. Refering to Fig. 7.6, the following parallel combinations give,

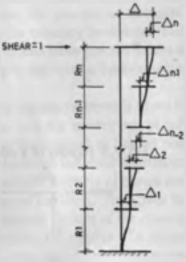


Fig. 7.5. A system in series.

$$R_{gh} = 10 + 10 = 20$$

$$R_{abc} = 8 + 16 + 16 = 40$$

$$R_{def} = 20 + 20 + 10 = 50$$

$$R_{abcdef} = 40 + 50 = 90$$

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

$$R = \frac{1}{[1/90 + 1/20]} = \frac{180}{11} = 16.36$$

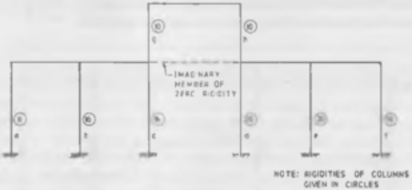


Fig. 7.6. Calculating rigidity of a bent.

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 centre of rigidity is then known.

Step 4: 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

$$p_y = P_y \frac{R_y}{\Sigma R_y} = P_y r_y \qquad (7.6)$$

where $r_y = R_y/\Sigma R_y$ = relative rigidity of a bent of rigidity R_y

A similar formula in the x-direction can be written as

$$p_x = P_x \frac{R_x}{\Sigma R_x} = P_x r_x \qquad (7.7)$$

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

Step 5: 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),

$$V_y = P_y \frac{R_y}{\Sigma R_y}\left(1 - \frac{e\,x\,\Sigma R_y}{J_p}\right) \qquad (7.8)$$

where $J_p = \Sigma\,(R_x\,y^2) + \Sigma\,(R_y\,x^2)$

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

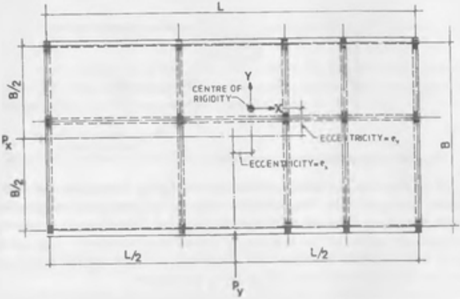


Fig. 7.7. Floor diaphragm under eccentric horizontal shear.

Denoting,

$$F_y = \left(1 - e\,x\,\frac{\Sigma R_y}{J_p}\right) \tag{7.10}$$

Eq. (7.8) gives with the help of Eq. (7.6),

$$V_y = p_y\,F_y \tag{7.11}$$

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 ($e = 0$), then $F_y = 1.0$ and Eq. (7.8) reduces to Eq. (7.6). Similarly, for bents in x direction,

$$V_x = P_x\,\frac{R_x}{\Sigma R_x}\left(1 + e\,y\,\frac{\Sigma R_x}{J_p}\right) \tag{7.12}$$

$$F_x = \left(1 + e\,y\,\frac{\Sigma R_x}{J_p}\right) \tag{7.13}$$

$$V_x = p_x\,F_x \tag{7.14}$$

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 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,

$$F_y \text{ or } F_x \geq 1.0 \tag{7.15}$$

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.

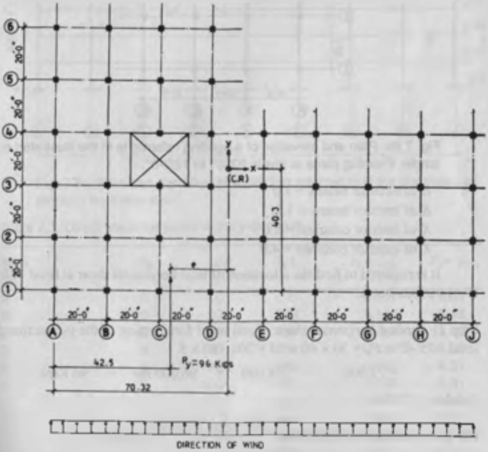


Fig. 7.8a. Plan and elevantion of a building referred to in the illustrated example. Framing plans at levels 0'-0", 10'-0" to 80'-0".

An illustrtive example, taken from Ref. 33 is given to illustrate the procedure of allocation analysis.

7.2.1. Illustrative Example

Fig. 7.8 gives line plans, front elevation and side elevation of a framed building with wind loads acting at floor levels calculated on the basis of 30 psf. Stiffness (K) of members are given as follows:

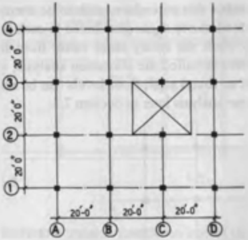


Fig. 7.8b. Plan and elevation of a building referred to in the illustrated example. Framing plans at levels 90'-0" to 120'-0".

K of exterior beams = 1.0
K of interior beams = 1.5
K of interior columns = 8.0
K of exterior columns = 4.0

It is required to find the allocation of total horizontal shear at level +80'-0" to various bents.

Step 1: *Applied horizontal shear.* Total wind force acting in the y-direction at level +75'-0" = $P_y = 30 \times 40 \times 60 + 30\times 160 \times 5$

$$= 72{,}000 \quad + \quad 24{,}000 \quad = \quad 96{,}000 \text{ lbs} \quad = \quad 96 \text{ Kips}$$

(1.0 kip= 1000 lbs)

$$\text{acting at a distance from bent A} = \frac{72 \times 30 + 24 \times 80}{96} = 42.5 \text{ ft}$$

$P_x = 0.$

There exist nine bents A to J to resist P_y = 96 Kips.

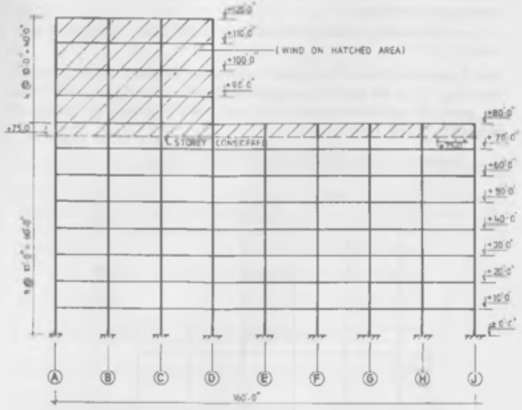


Fig. 7.8c. Plan and elevation of a building referred to in the illustrated example – front elevation.

Table 7.1. Direct shear for bents in the y-direction.

Bent	R_y	d_1	$R_y d_1$	$r_y = \frac{R_y}{\Sigma R_y}$	$P_y = p_y r_y$
A	6	0	0	0.097	9.31
B	10	20	200	0.161	15.46
C	10	40	400	0.161	15.46
D	8	60	480	0.129	12.38
E	6	80	480	0.097	9.31
F	6	100	600	0.097	9.31
G	6	120	720	0.097	9.31
H	6	140	840	0.097	9.31
J	4	160	640	0.064	6.15
Σ	62	–	4360	1.000	96.00

Step 2: *Rigidity of columns.* For columns of constant height and assuming

beams to be very stiff, Eq. (7.2) indicates that the rigidity of a column is proportional to its stiffness.

Let the rigidity of an exterior column = 1, then the rigidityof an interior column = 8/4 = 2.

Step 3: *Rigidity of bents.* The rigidity of a bent in the storey under consideration (Fig. 7.8) is the sum of the rigidities of the constitutent columns. Table 7.1 gives the rigidities of bents A to J and Table 7.2 gives the rigidities of bents 1 to 6. From Table 7.1,

$$\text{distance of centre of rigidity from bent A} = \frac{\Sigma\,(R_y\,d)}{\Sigma\,R_y}$$

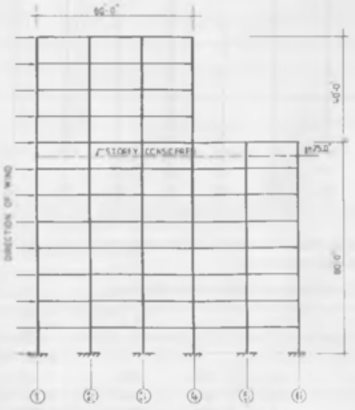


Fig. 7.8d. Plan and elevation of a building referred to in the illustrated example – side elevation.

$= 4360/62 = 70.32$ ft. From Table 7.2,

distance of centre of rigidity from bent $I = \Sigma(R_x d)/\Sigma R_x = 2500/62 = 40.3$ ft

The eccentricity of P_y works out to be (Fig.7.8a), $e = 70.32 - 42.50 = 28.82$ ft.

Step 4: *Direct shear on each bent assuming $e_r = 0$.* The direct shear p_y on each bent is calculated by Eq. (7.6) and it is given in Table 7.1.

Table 7.2. Centre of rigidity of bents in the x-direction.

Bent	R_x	d_2	$R_x\ d_2$
1	9	0	0
2	16	20	320
3	16	40	340
4	11	60	660
5	6	80	480
6	4	100	400
Σ	62	–	2500

Step 5: *Design shear on each bent when e* = 28.82 ft. The direct shears on bents ascertained in step 4 are multiplied by F_y to correct for the eccentricity e = 28.82 ft. The computations for J_p based on Eq. (7.9) are shown in Table 7.3, which gives

$$J_p = 146993 + 49193 = 196186 \text{ lbft}$$

Eq. (7.10) gives,

$$F_y = 1 - \frac{28.82\,(x)\,62}{196186} = 1 - 0.0091\,x$$

Eq. (7.11) gives design shear on each bent and it is given in Table 7.4.

Table 7.3. Calculation of J_p

Bents in the y-direction				Bents in the x-direction			
Bent	R_y	x	$R_y x^2$	Bent	R_x	y	$R_x y^2$
A	6	–70.32	29669	1	9	–40.3	14617
B	10	–50.32	25321	2	16	–20.3	65.93
C	10	–30.32	9193	3	16	–0.3	1
D	8	–10.32	852	4	11	+19.7	4269
E	6	+9.68	562	5	6	+39.7	9457
F	6	+29.68	5285	6	4	+59.7	14256
G	6	+49.68	14809				
H	6	+69.68	29132				
J	4	+89.68	32170				
Σ	62	–	146993	Σ	62	–	49193

7.2.2. Alternative Methods

The same example with the actual values of stiffness of beams and columns, has been solved by the precise methods referred to earlier and now listed as under:

i) Wilbur's Method [40]
ii) Method of joint coefficients by Smith[41]
iii) Method of *D*-values by Muto[42]

Table 7.4. Design shear for bents in *y*-direction.

Bent	Direct shear (Kips)	x (ft)	.0091 x	F_y= 1–.0091 x	Shear corrected for eccentricity (Kips)	Design shear (Kips)
A	9.31	–70.32	–.64	1.64	15.3	15.3
B	15.46	–50.32	–.46	1.46	22.4	22.4
C	15.46	–30.32	–.28	1.28	19.7	19.7
D	12.38	–10.32	–.09	1.09	13.5	13.5
E	9.31	+9.68	+.09	0.91	8.5	9.31
F	9.31	+29.68	+.27	0.73	6.8	9.31
G	9.31	+49.68	+.45	0.55	5.2	9.31
H	9.31	+69.68	+.63	0.37	3.5	9.31
J	6.15	+89.68	+.82	0.18	1.1	6.15
Σ	69.00	–	–	–	96.00	–

Table 7.5. Comparison of design shear for bents by different methods.

Bent	Design shear by different methods (1000 lb)				
	Proposed method	Wilbur's method	Method of *D*-values	Method of joint coefficients	Tributary shear method
A	15.3	15.70	16.30	16.30	13.50
B	22.4	21.40	22.80	22.20	27.00
C	19.7	15.40	16.20	16.20	27.00
D	13.5	13.90	14.30	13.20	15.00
E	9.31	9.00	9.50	10.60	3.00
F	9.31	9.00	9.50	10.60	3.00
G	9.31	9.00	9.50	10.60	3.00
H	9.31	9.00	9.50	10.60	3.00
J	6.15	6.00	6.20	6.70	1.50

The results of these methods are compared in Table 7.5, which also shows the tributary bent shears calculated on the basis of exposed area tributary to each bent multiplied by the wind pressure. It is seen from Table 7.5 that the proposed method of rigidities gives bent shears quite compatible with those given by the precise methods except in the case of bent C where the variation is as high as 20% from the method of *D*-values, which is attributed to the absence of a beam in one of its spans. For regula· arrangement of beams, as is normally required in earthquake resistant structures, it follows that the proposed method is expected to give acceptable results. The results obtained from the precise methods which depart from the main assumption of this method and take into account the actual stiffness of beams, vary among themselves to the extent of 8%. However, horizontal loads on a structure are attributed to wind or earthquake which are quite inexact by their very nature and there is, therefore, an element of arbitrariness in their evaluation. Hence, as it is concerned only with the distribution of the total horizontal shear to the various bents, it does not make any particular method safer than the rest. The method of rigidities has the advantage of simplicity over others, involves less labour and is less liable to numerical errors. The tributary shear method which is often employed in practice due to the minimum effort involved, gives a maximum variation of 314% from the method of D-values and it is, thus totally unreliable, as it does not take into account the rigidity of the resisting bents and it neglects torsion altogether.

In a discussion of this method,[45] a comparison of the method with a computer program HIRISE based on reference 43 is given and it is reproduced in Table 7.6 It shows that excepting bent C. the results are acceptable.

Table 7.6. Comparison of shear by author's proposed method and computer program HIRISE.

Bent	Bent shear by different methods (Kips)			
	Factor F_y	Authors proposed	Program HIRISE	Variation %
A	1.64	15.30K	16.35K	−8%
B	1.46	22.40	21.62	+4%
C	1.28	19.70	13.29	+48%
D	1.09	13.50	12.41	+9%
E	0.91	8.5	11.12	−24%
F	0.73	6.8	8.84	−23%
G	0.55	5.2	6.58	−21%
H	0.37	3.5	4.30	−19%
J	0.18	1.1	1.28	−14%

When the floor diaphragm is under an eccentric shear, the calculated eccentricity is to be increased by 50% as presecribed by clause 4.2.4 of IS: 1893.[9] This provision of the code increases the cost of such buildings. So it is economical to have buildings with symmetrical floor plans in earthquake-prone areas.

7.3. FRAME ANALYSIS

A frame with horizontal loads at floor levels can now be analysed by a computer program, giving exact values of beam and column moments. Earlier, approximate methods like portal or cantilever or factor methods, which assume point of contraflexure at mid-point of members, are now obsolete. A manual method called the 'Method of Multiples'[46,47] may be used to give comparable results, when a computer program is not available. For quick but approximate results, one may use the 'Method of Rigidities' with points of contraflexure at mid-point of members. This method is useful in taking into account the various column sizes in plan. An example is given below (Fig. 7.9) and it is solved by the following methods and results can be compared by examining Figs. 7.10-7.13:

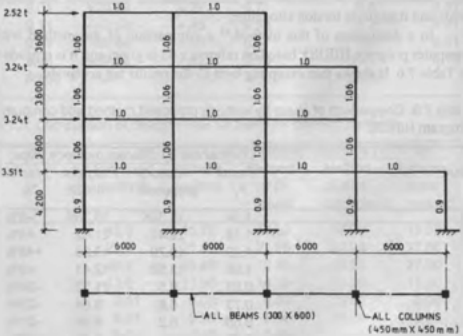


Fig. 7.9. Example of frame analysis. (*k*-values written near centres of members.)

i) Computer program (Fig. 7.10).
ii) Method of multiples (Fig. 7.11).
iii) Method of rigidities (Fig. 7.12).
iv) Portal method (Fig. 7.13).

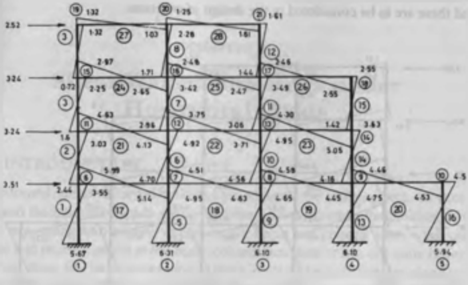


Fig. 7.10. Results of analysis by computer program.

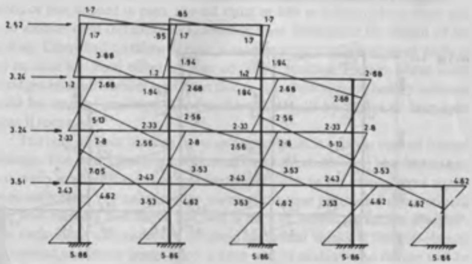


Fig. 7.11. Results of analysis by the method of multiples.

The moments due to horizontal loads are to be combined with those of the vertical load moments and the members are to be designed with a load factor of 1.2 instead of 1.5. Further reversal of moments due to horizontal loads shall also be considered. Shears in beams due to horizontal loads are constant along the beam spans, while the shear diagram under vertical load is triangular. This difference in shear diagrams is also important to be considered in the design of shear stirrups. Also, axial loads in columns due to earthquake or wind loads are also of an alternating nature in sign, i.e. compressive or tensile and these are to be considered in the design of columns.

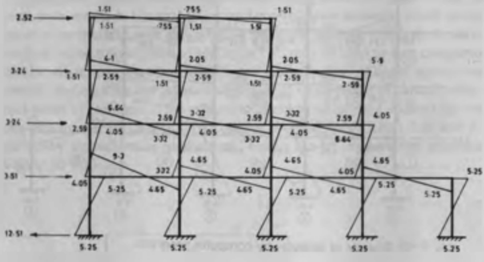


Fig. 7.12. Results of analysis by the method of rigidities.

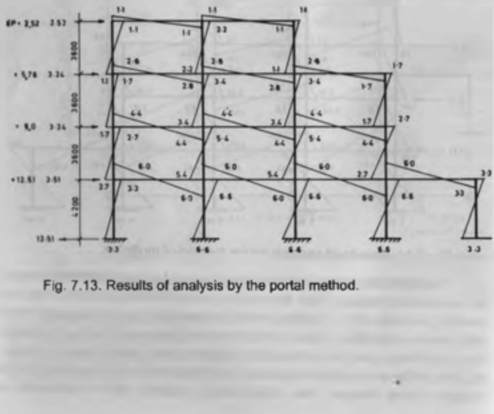


Fig. 7.13. Results of analysis by the portal method.

CHAPTER 8

Shear-Walled Buildings under Horizontal 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'.

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 asumed 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[51] 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 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 more horizontal 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 (Δx) 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),

$$\Delta_x = \frac{V_x\, h^3}{3\,E\,I_y} + \frac{V_x\, h'\, h^2}{2\,E\,I_y} + \frac{1.2\,V_x\, h}{A_y\, G}$$

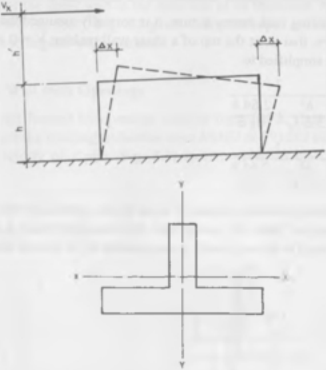


Fig. 8.1. Deflection of cantilever shear wall subject to lateral load.

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

$$R_x = \frac{V_x}{\Delta x} = \frac{1}{\dfrac{h^3}{3E\,I_y} + \dfrac{h'\,h^2}{2E\,I_y} + \dfrac{2.64\,h}{A_y\,E}} \tag{8.1}$$

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

$$R_y = \frac{V_y}{\Delta y} = \frac{1}{\dfrac{h^3}{3E\,I_x} + \dfrac{h'\,h^2}{2E\,I_x} + \dfrac{2.64\,h}{A_x\,E}} \tag{8.2}$$

The notation used above is explained below:

E = modulus of elasticity of material of shear wall
G = shear modulus of material of shear wall
I = moment of inertia of shear wall about the axis of bending
A = area of web about the axis of bending
V = aplied shear in a given direction
Δ = deflection due to applied shear in a given direction
h = height of shear wall element
h' = height of applied shear above the top of shear wall element

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 to

$$R_x = \frac{1}{\dfrac{h^3}{3\,E\,I_y} + \dfrac{2.64\,h}{A_y\,E}} \tag{8.3}$$

$$R_y = \frac{1}{\dfrac{h^3}{3\,E\,I_x} + \dfrac{2.64\,h}{A_x\,E}} \tag{8.4}$$

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.

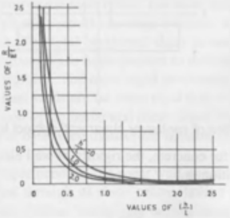


Fig. 8.2. Rigidity of a cantilever wall element fixed at base.

8.2.3 Solid Rectangular Shear Walls

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

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

$$I_x = \frac{1}{12} L t^3 \approx 0$$

$$A_y = A_x = Lt$$

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),

$$R = \frac{E t}{4\left(\frac{h}{L}\right)^3 + 6\left(\frac{h'}{L}\right)\left(\frac{h}{L}\right)^2 + 2.64\left(\frac{h}{L}\right)} \tag{8.5}$$

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 $h^3/3EI$ to $h^3/12EI$ in Eqs. (8.3) and (8.4). The rigidity of a pier (Fig. 8.3) is then given in the direction of its length,

$$R = \frac{E t}{\left(\frac{h}{L}\right)^3 + 2.64\left(\frac{h}{L}\right)} \tag{8.6}$$

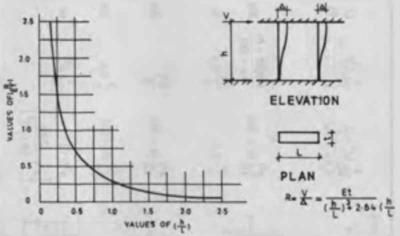
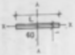
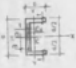
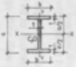
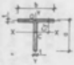
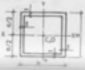
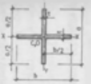
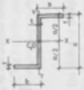
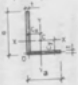
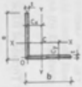


Fig. 8.3. Rigidity of a wall element fixed at both ends.

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

Table 8.1. Rigidity characteristics of various types of shear walls.

Case	Shape of shear wall	I_x	I_y	A_y	A_x	R_x	R_y	R_{xy} or R_{yx}	J	Position of centroid C (C_x, C_y)	Position of shear centre O
1.		$\frac{1}{12}Lt^3 \approx 0$	$\frac{1}{12}L^3t$	Lt	Lt	$\frac{1}{\frac{h^3}{3EI_y}+\frac{2.64h}{A_yE}}$	≈ 0	0	≈ 0	$C_x = C_y = 0$	$m = 0$
2.		$\frac{a^2t}{12}(a+6b)$	$\frac{b^3t(2a+b)}{3(a+2b)}$	$2bt$	at	$\frac{1}{\frac{h^3}{3EI_y}+\frac{2.64h}{A_yE}}$	$\frac{1}{\frac{h^3}{3EI_x}+\frac{2.64h}{A_xE}}$	0	$\frac{Ra^2}{2}$	$C_x = \frac{b^2}{a+2b}$ $C_y = 0$	$m = \frac{3b^2}{a+6b}$
3.		$\frac{a^2t}{12}(a+6b)$	$\frac{b^3t}{6}$	$2bt$	at	-"-	-"-	0	$\frac{Ra^2}{2}$	$C_x = C_y = 0$	$m = 0$
4.		$a^3t/12\left[1+\frac{3ab}{(a+b)^2}\right]$	$\frac{1}{12}b^3t$	bt	at	$\frac{1}{\frac{h^3}{3EI_y}+\frac{2.64h}{A_yE}}$	$\frac{1}{\frac{h^3}{3EI_x}+\frac{2.64h}{A_xE}}$	0	≈ 0	$C_x = 0$ $C_y = \frac{a^2}{2(a+b)}$	$m = C_y$
5.		$\frac{a^2t}{6}(a+3b)$	$\frac{b^2t}{6}(3a+b)$	$2bt$	$2at$	-"-	-"-	0	$\frac{a^2b^2tE}{1.1h(a+b)}$	$C_x = C_y = 0$	$m = 0$
6.		$\frac{1}{12}a^3t$	$\frac{1}{12}b^3t$	bt	at	-"-	-"-	0	≈ 0	$C_x = C_y = 0$	$m = 0$
7.		$\frac{a^2t}{12}(a+6b)$	$\frac{2}{3}b^3t$	$2bt$	at	-"-	-"-	0	$R_f\frac{a^2}{2}$	$C_x = C_y = 0$	$m = 0$
8.		$\frac{5}{24}a^3t$	$\frac{5}{24}a^3t$	at	at	$\frac{1}{\frac{h^3}{3EI_y}+\frac{2.64h}{A_yE}}$	$\frac{1}{\frac{h^3}{3EI_x}+\frac{2.64h}{A_xE}}$	$R_{xy} = 0.6R_y$ $Ryx = 0.6R_x$	≈ 0	$C_x = a/4$ $C_y = a/4$	$m_x = C_x$ $m_y = C_y$
9.		$\frac{a^3t(a+4b)}{12(a+b)}$	$\frac{b^3t(4a+b)}{12(a+b)}$	bt	at	-"-	-"-	$R_{xy} = \frac{3b^2R_y}{a(a+4b)}$ $R_{yx} = \frac{3a^2R_x}{b(b+4a)}$	≈ 0	$C_x = \frac{b^2}{2(a+b)}$ $C_y = \frac{a^2}{2(a+b)}$	$m_x = C_x$ $m_y = C_y$

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]

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 R_x 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]

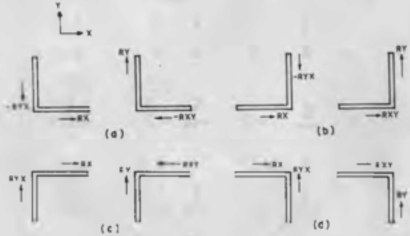


Fig. 8.4. Direction of R_{xy} and R_{yx} for various dispositions of the angle wall element. (a) Angle in position 1; (b) Angle in position 2; (c) Angle in position 3; (d) Angle in position 4.

8.2.6 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,

$$J=\frac{T}{\theta} \tag{8.7}$$

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,

$$J_u = \frac{E\,t^3\,\Sigma\,0}{6.6\,h} \tag{8.8}$$

where $\Sigma 0$ equals the perimeter of the section of shear wall of height h and thicknes 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. 8.5 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,

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

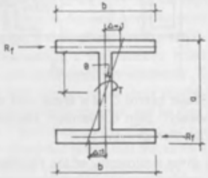


Fig. 8.5. Torsional rigidity of a shear wall of open section based on non-uniform torsion theory (R_f = rigidity of wall element $b \times t$).

which is produced by a torque

$$T = R_f \times a$$

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,

$$J_n = \frac{T}{\theta} = R_f \times \frac{a^2}{2} \tag{8.9}$$

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

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

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.

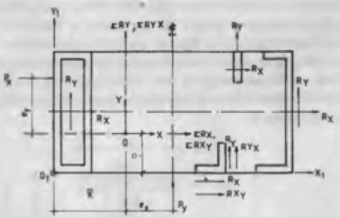


Fig. 8.6. Shear centre O of a shear wall structure (P_x, P_y are not to act simultaneously). Sign convention: clockwise moments +ve in the increasing direction of x and y.

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]

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 coordinates 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

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

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

where $(\bar{x}, \bar{y})$ 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 $\bar{x}$ and $\bar{y}$ to locate the shear centre of the structure, taking proper signs of all the forces involved.

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

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

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

8.2.9 Evaluation of Applied Horizontal Loads

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 $-P_y.ex$ 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 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,

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

$$S_y = -T_p \frac{(x \, R_y - x \, R_{xy})}{J_p} \tag{8.24}$$

$$T = T_p \frac{J}{J_p} \tag{8.25}$$

where, $J_p = \Sigma(y^2 R_x) + \Sigma(x^2 R_y) - \Sigma(xy \, R_{xy}) - \Sigma(xy \, R_{yx}) + \Sigma J$ (8.26)

and ΣJ = summation of torsional rigidities of all shear wall elements

When $R_{yx} = R_{xy} = 0$ for all shear walls, the above equations simplify to,

$$S_x = T_p \frac{y R_x}{J_p} \tag{8.27}$$

$$S_y = -T_p \frac{x R_y}{J_p} \tag{8.28}$$

$$T = T_p \frac{J}{J_p} \tag{8.29}$$

$$J_p = \Sigma (y^2 R_x) + \Sigma (x^2 R_y) + \Sigma J \tag{8.30}$$

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. 8.7a)

$$N_x = \pm \frac{T}{a} \tag{8.31}$$

For closed sections (Fig. 8.7b)

$$N_x = \pm \frac{T}{2a} \tag{8.32}$$

$$N_y = \pm \frac{T}{2b} \tag{8.33}$$

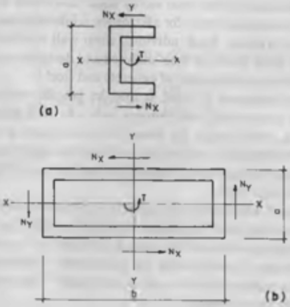


Fig. 8.7. Flange forces in wall sections under torsion. (a) Open section; (b) Closed section.

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,

$$Q_x = F_x + S_x \tag{8.34}$$

$$Q_y = F_y + S_y \tag{8.35}$$

where F_x, F_y and S_x, S_y are to be determined from the relevant equations 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 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.

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 its 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.

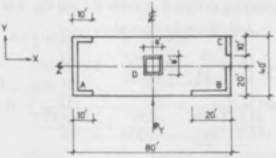


Fig. 8.8. Plan of a shear wall structure of the illustrative Example 8.2.12. (Wall thickness = 6" everywhere.)

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

$$A_x = at = 40 \times 0.5 = 20.0 \text{ sft}$$

$$A_y = 2bt = 2 \times 10 \times 0.5 = 10.0 \text{ sft}$$

$$I_x = \frac{a^2 t}{12}(a + 6b) = \frac{1}{12} \times (40)^2 \times 0.5\,(40 + 60) = 6670 \text{ ft}^4$$

$$I_y = \frac{b^3 t}{3}\frac{(2a + b)}{(a + 2b)} = \frac{1}{3} \times (10)^3 \times 0.5\,\frac{(80 + 10)}{(40 + 20)} = 250 \text{ ft}^4$$

$$R_x = \frac{1}{\dfrac{h^3}{3I_y} + \dfrac{2.64h}{A_y}} = \frac{1}{\dfrac{(15)^3}{3 \times 250} + \dfrac{2.64 \times 15}{10}} = 0.118$$

$$R_y = \frac{1}{\dfrac{h^3}{3I_x} + \dfrac{2.64h}{A_x}} = \frac{1}{\dfrac{(15)^3}{3 \times 6670} + \dfrac{2.64 \times 15}{20}} = 0.465$$

$$R_{xy} = R_{yx} = 0$$

$$J = R_f \frac{a^2}{2}$$

For $\dfrac{h'}{L} = 0$, $\dfrac{h}{L} = 1{,}5$, Fig. 8.2 gives $R_f = 0.03$.

Then, $J = 0.03 \times (\dfrac{40^2}{2}) = 24$

$$m = \frac{3b^2}{(a+6b)} = \frac{3 \times (10)^2}{(40+60)} = 3.0 \text{ ft.}$$

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

$$A_x = A_y = at = 20 \times 0.5 = 10.0 \text{ ft}^2$$

$$I_x = I_y = \frac{5}{24} a^3 t = \frac{5}{24} \times (20)^3 \times .5 = 835 \text{ ft}^4$$

$$R_x = R_y = \frac{1}{\frac{h^3}{3I_x} + \frac{2.64h}{At}} = \frac{1}{\frac{(15)^3}{3 \times 835} + \frac{2.64 \times 15}{10}} = 0.188$$

$$R_{xy} = +0.6R_x = +0.6 \times .188 = +0.113$$

$$R_{yx} = -0.6R_x = -0.113$$

$$J = 0$$

$$m_x = m_y = \frac{a}{4} = \frac{20}{4} = 5.0 \text{ ft}$$

Shear Wall C

For $\frac{h'}{L} = 0$, $\frac{h}{L} = \frac{15}{10} = 1.5$, 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,

$$R_x = 0$$

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

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

$$A_x = A_y = 2at = 2 \times 8.0 \times .5 = 8.0 \text{ ft}^2$$

$$I_x = I_y = \frac{2}{3} a^3 t = \frac{2}{3} \times (8)^3 \times 0.5 = 171.0 \text{ ft}^4$$

$$R_x = R_y = \frac{1}{\frac{h^3}{3I_x} + \frac{2.6h}{A_x}} = \frac{1}{\frac{(15)^3}{3 \times 171} + \frac{2.64 \times 15}{8}} = 0.087$$

$$R_{xy} = R_{yx} = 0$$

$$J = \frac{a^2 b^2 t}{1.1h(a+b)} = \frac{64 \times 64 \times 0.5}{1.1 \times 15(8+8)} = 7.8$$

$$m = 0$$

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.

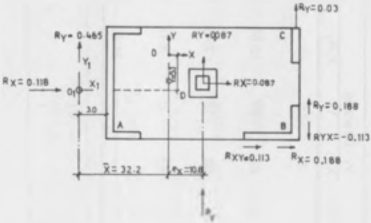


Fig. 8.9. Summary of rigidities of shear walls and location of the shear centre O of the structure.

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

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

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 $(\bar{x}, \bar{y})$ is given by Eqs. (8.11) and (8.12), the various quantities required in the above equations, being computed with the help of Table 8.3. This leads to the following equations:

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

which on solution give,

$$\bar{x} = 32.2 \text{ ft}$$
$$\bar{y} = 5.1 \text{ ft}$$

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.

Shear wall	X_1 - Direction						Y_1 - 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

Table 8.4. Calculation of direct shear on walls.

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	0.580 P_y	0.042P_y
B	0.188	0.188	0.113	−0.113	0.074	−0.0128	−0.0212	−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

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

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

$J_p = \Sigma(Y^2 . R_X) + \Sigma(X^2 . R_Y) - \Sigma(Y . Rx_y) - \Sigma(XY . R_{yx}) + \Sigma J$

$= 123.82 + 1056.7 - (-145) - 145 + 31.8 = 1212.3$

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

Table 8.6. Effect of torsion on walls.

Shear wall	Torsional shear												Force in flanges			
	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=−0.0089 P_yJ	N_x	N_y
–	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
A	0.465	0.118	−32.2	−5.1	−0.60	−15.0	0	0	0	0	$0.0053P_y$	$-0.133P_y$	24	$-0.214P_y$	$\pm0.0054P_y$	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	0
C	0.030	0	50.8	9.9	0	1.52	0	0	0	0	0	$0.014P_y$	0	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	$-0.07P_y$	$\pm0.0044P_y$	$\pm0.0044P_y$
Σ	0.770	0.393	–	–	–	–	0.113	−0.113	–	–	0	0	31.8	–	0	0

$$\frac{T_P}{J_P} = \frac{10.8}{1212.3} P_y = -0.0089\, P_y$$

$$e_x = (40 + 3) - 32.2 = 10.8 \text{ ft}$$

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_x, 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	0.037	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 horizontal 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. 8.11). Each section of wall has to be designed 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.

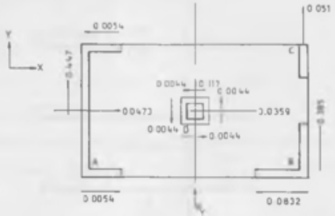


Fig. 8.10. Final allocation of applied shear Py to shear walls given as fraction of it.

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 ($L \times t$). For the following data,

$L = 10.0$ m, $P = 200.0$ t, $f_{ck} = 150$ kg/cm^2

$t = 0.3$ m, $M = 200.0$ tm, $f_y = 4150$ kg/cm^2

$H = 20.0$ t

$$\frac{P_u}{f_{ck}\, bD} = \frac{1.2 \times 200 \times 1000}{150 \times 1000 \times 30} = 0.053$$

$$\frac{M_u}{f_{ck}\, b\, D^2} = \frac{1.2 \times 200 \times 10^5}{150 \times 30 \times (1000)^2} = 0.005$$

Assuming $d'/D = 12/1000 = 0.012 \approx 0.05$, (Fig.8.12) and with $A_s/2$ on each face,

$p/f_{ck} = 0.02, \quad p = .02 \times 15 = 0.3$

$A_s = 0.3 \times 30 \times \frac{1000}{100} = 90$ cm^2

Provided by 12 ϕ 32 bars = 96.51 cm^2

6 ϕ 32 bars shall be provided at either end of the shear wall. In the rest

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

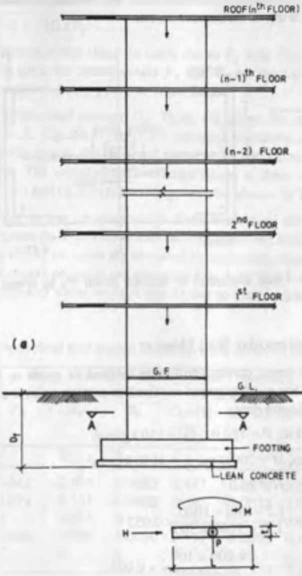


Fig.8.11. Solid rectangular shear wall. (a) Elevation of shear wall; (b) Section A-A.

$\frac{P}{L}$ = 20.0 t/m, M_{vL} = 2.0 tm/m, l = 3.6 m

$\frac{l_{ef}}{b} = \frac{1.2 \times 3.6}{0.3} = 14.4 > 12$,

M_a, 30 = $kP \times .3 \times (14.4)^2/2000 = 0.031\, kP$

$= .031 \times 1 \times 20 = 0.6$ tm/m

$\frac{P}{L} = 20.0$ tm/m, M30 $= 2.0 + .6 = 2.6$ tm/m

$$\frac{P_u}{f_{ck}bD} = \frac{1.5 \times 20.0 \times 1000}{150 \times 100 \times 30} = 0.067$$

$$\frac{M_u}{f_{ck}bD^2} = \frac{1.5 \times 2.6 \times 10^5}{150 \times 100 \times (30)^2} = 0.029$$

$d'/D = \frac{5}{30} = .167 = 0.20,\quad A_s/2$ on each face,

$p/f_{ck} = 0, \quad p = 0$

Minimum steel as for reinforced concerete walls shall be provided.

Vertical steel $= 0.4\%$, $A_{sv} = 0.4 \times 30 \times \frac{100}{100} \quad = 12.0$ cm²/m

$\frac{A_{sv}}{2} = \frac{12.0}{2} \quad = 6.0$ cm²/m

Provide ϕ 12/150 c/c E.F $\quad = 7.54$ cm²/m

Horizontal steel $= 0.2\%$, $A_{sh} = 0.2 \times 30 \times \frac{100}{100} = 6.0$ cm²/m

$A_{sh}/2 = \frac{6.0}{2} \quad = 3.0$ cm²/m

Provide ϕ 10/200 c/c E.F. $\quad =3.93$ cm²/m

Fig 8.12 gives the steel arrangement in the solid wall as designed above. All vertical bars shall be tied by small links ϕ 8/200 c/c, as one does for column bars.

Check on shear

$$T_v = \frac{1.2 \times 20.0 \times 1000}{1000 \times 30 \times 10} = 0.08 \ \text{N/mm}^2$$

$T_c = 0.35$ N/mm² (taken as minium)

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. 8.13). The vertical load (P) is shared equally by the two solid elements if equal, otherwise 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, $a = \frac{1}{2}(L + l')$

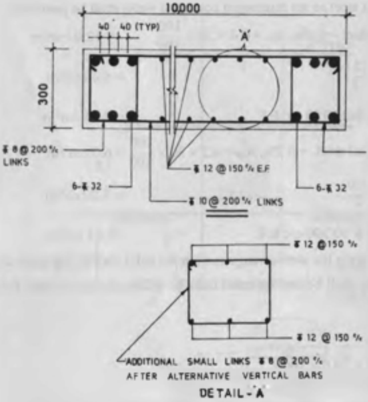


Fig. 8.12. Steel arrangement in solid shear wall.

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 shear equal in magnitude to C or T and moment equal to $T.l'/2$.

With the following data,

L=10.0 m $\qquad \frac{L - l'}{2} = 3.5\text{m} \qquad M = 200.0$ tm

$l' = 3.0$ m $\quad a = \frac{1}{2}(10 + 3) = 6.5\text{m}, \quad P = 200.0$ t

$t = 0.3$ m $\quad \frac{P}{2} = 100.0$ t

$$C = T = \frac{M}{a} = \frac{200}{6.5} = 30.8 \text{ t}$$

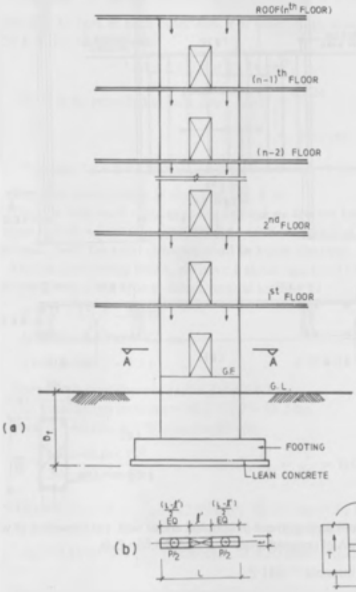


Fig. 8.13. Coupled shear wall. (a) Elevation of coupled shear wall; (b) Section A-A.

For a solid element of length = 3.5 m, $t = 0.3$ m,

Max. compresssion = 100 + 30.8 = 130.8 t = 131 t
Min. eccentricity moment = .02 P

$$= .02 \times 131 = 2.62 \text{ tm}$$

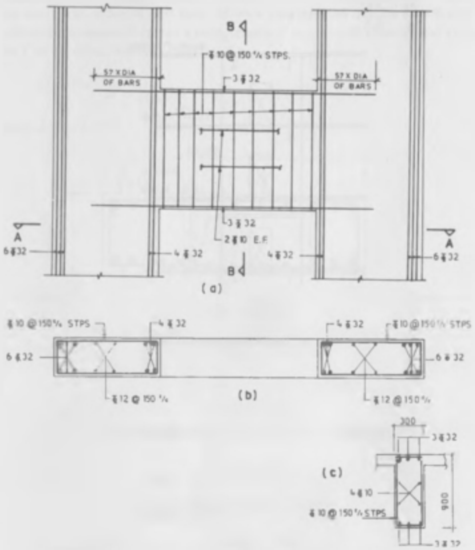


Fig. 8.14. Steel arrangement in coupled shear wall. (a) Elevation of wall; (b) Section A-A of coupled shear wall; (c) Section B-B.

Slenderness moment = .031 P

$$= .031 \times 131 = 4.06 \text{ tm}$$

For $P = 131$ t, M30 = 2.62 + 4.06 = 6.68 = 6.7 tm

$$\frac{P_u}{f_{ck} bD} = \frac{1.2 \times 131 \times 1000}{150 \times 350 \times 30} = 0.1$$

$$\frac{M_u}{f_{ck}bD^2} = \frac{1.2 \times 6.7 \times 105}{150 \times 350 \times (30)^2} = 0.017 \simeq .02$$

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 ϕ 32 bars at each outer end. For inner ends, steel area cut-off = ϕ 12/150 E.F. for 3.0 m length

$$= 7.54 \times 2 \times 3.0 = 45.24 \text{ cm}^2$$

Steel to be provided at each inner edge $= \dfrac{45.24}{2} \times 1.25$

$= 28.3 \text{ cm}^2$

Provided by 4 $\bar{\phi}$ 32 bars $= 32.17 \text{ cm}^2$

The steel arrangement is shown in Fig. 8.14.

Tension side wall element is not critical, as the net load is compressive and equal to 100 – 31 = 69 t. But, because of the reversal of direction of horizontal loads, both the 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

$$w = .7 + 1.5 = 2.2 \text{ t/m},$$

$$M = 2.2 \times 3.0^2/12 = 1.7 \text{ tm}$$

$$S = 2.2 \times 3.0/2 = 3.3 \text{ t}$$

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,

$$k = \frac{1.2 \times 48.2 \times 10^5}{30 \times (85)^2 \times 10} = 2.7 \text{ N/mm}^2, \quad \frac{d'}{d} = \frac{5}{85} = 0.05$$

$$A_{st} = 0.90 \times 30 \times \frac{85}{100} = 22.95 \text{ cm}^2, \ 3\bar{\phi}\ 32 \text{ Top} = 24.12 \text{ cm}^2$$

$$A_{sc} = 0.18 \times 30 \times \frac{85}{100} = 4.59 \text{ cm}^2, \ 2\ \bar{\phi}\ 20 \text{ Bott.} = 6.28 \text{ cm}^2$$

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

$$T_v = \frac{1.2 \times 34.3 \times 1000}{30 \times 85 \times 10} = 1.61 \text{N/mm}^2$$

For $p_t = \frac{24.12 \times 100}{30 \times 85} = 0.95, \quad T_c = 0.6 \text{ N/mm}^2$

$V_{us}/d = (0.161 - 0.06) \times 30 = 3.03$ kN/cm

Provided by two legged stirrups Φ 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

Provided by 4 Φ 10 bars = 3.14 cm^2

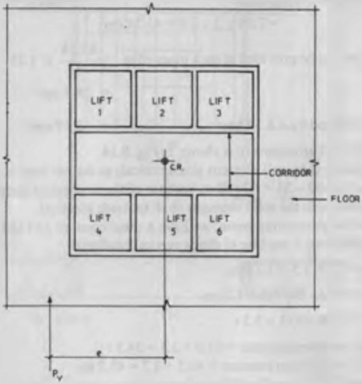


Fig 8.15. Plan of eccentric core in a tall building.

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 concentric 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 Nasseta[59] have suggested to design the existing frames in the building for the effect of the torsion in plan, while the core will be designed only for the full horizontal shear (Fig.8.16). It will relieve the core of the torsional effects. The extra horizontal forces on frames H_1, H_2, etc. are given by the equations,

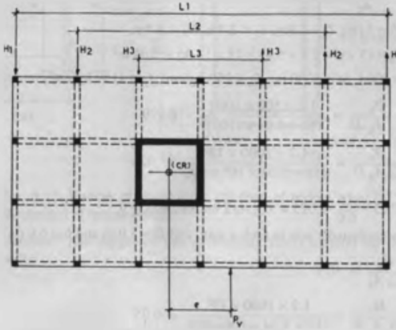


Fig 8.16. Plan of eccentric core in a tall building.

$$H_1L_1 + H_2L_2 + H_3L_3, + \ldots \text{ (etc.)} = P_ye \tag{8.36}$$

$$\frac{H_1}{L_1} = \frac{H_2}{L_2} = \frac{H_3}{L_3} = \ldots \text{ (etc.)} \tag{8.37}$$

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 horizontl loads H_1, H_2, 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 $\dfrac{M_u}{f_{ck}A_g D}$, where,

A_g = concrete area of the hollow core.

For a rectangular hollow column (Fig.8.17),

$$Ag = (bD - b_1 D_1)$$

$$p = \frac{A_s}{A_g} \times 100$$

With, $b = 3.0\text{m}, D = 6.0\text{m}, b_1 = 2.4\text{m}, D_1 = 5.4\text{m}.$

$$A_g = (3 \times 6 - 2.4 \times 5.4) = 18 - 12.96 = 5.04 \text{ m}^2$$

With $P = 500$ t, $M = 1000$ tm, $f_{ck} = 150$ kg/cm^2, $f_y = 4150$ kg/cm^2,

$$\frac{P_u}{f_{ck} A_g .D} = \frac{1.2 \times 500 \times 1000}{150 \times 5.04 \times (100)^2} = 0.079$$

$$\frac{M_u}{f_{ck} A_g D} = \frac{1.2 \times 1000 \times 10^5}{150 \times 5.04 \times 10^4 \times 600} = .026$$

With, $d^1/D = \dfrac{.15}{6.0} = .025 \approx .05$, $A_s/2$ on E.F, $p/f_{ck} = 0$

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

$$\frac{P_u}{f_{ck} A_g} = .079$$

$$\frac{M_u}{f_{ck} A_g D} = \frac{1.2 \times 1000 \times 10^5}{150 \times 5.04 \times 10^4 \times 300} = 0.05$$

with $d^1/D = \dfrac{.15}{3.0} = 05$, $A_s/2$ on E.F.

$$p/f_{ck} = .015$$

$$p = .015 \times 15 = .225$$

$$A_s = .225 \times 5.04 \times \frac{10^4}{100} = 113.4 \text{ cm}^2$$

16 $\bar{\phi}$ 32 bars give = 128.68 cm^2

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 rules as written earlier, apply here too.

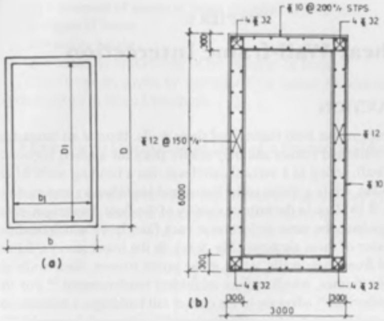


Fig. 8.17. Design of shear core. (a) Plan of hollow core; (b) Steel arrangement of hollow core.

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

CHAPTER 9

Shear Wall-frame Interaction

9.1 INTRODUCTION

When a tall structure has both frames and shear walls, there is an interaction between shear walls and frames and they jointly share the applied horizontal shear. A shear wall, acting as a vertical cantilever, has a bending mode of deflection (Fig. 9.1a), while a frame under horizontal loads has a shear mode of deflection (Fig. 9.1b) Due to the infinite rigidity of the floor diaphragm, shear walls and frames have the same deflection at each floor level, which modifies the final behaviour of these elements (Fig, 9.1c). In the lower storeys, frames relieve the load from shear walls, while in the upper storeys, shear walls get supported by the frames, which require additional reinforcement.[64] For this reason, some authorities[36] advocate in the case of tall buildings, a minium column reinforcement of 1.25% of gross column area in place of the usual 0.8%, in the upper most storeys.

The shear wall frame interaction problem is basically the problem of 'allocation analysis', by which the share of the horizontal load resisted by each vertical resisting element is to be found, after which the frame and shear wall analyses are to be done as explained in Chapters 7 and 8, respectively.

9.2 Allocation Analysis

The shear wall frame interaction has been tackled by Khan and Sbarounis,[48] Rosman,[49] Parme,[50] Mcloed,[61] Som and Narasimhan[62] among others. These methods are best tackled by computers. Herein, an approximate method given by Salvadori[25] is explained.

The deflection (Δ_f) at the top of a frame of n storeys and m bays, under horizontal load W is given by,

$$\Delta_f = \frac{Wh^3}{24(m+1)EI_1}\left[n^2 + \beta\left(1 + 2n.\overline{n-1}\right)\right] \tag{9.1}$$

where, n = number of storeys of frame

m = number of bays of frame

W = storey horizontal load on frame

$\beta = \frac{l}{h}\,\frac{I_1}{I_2}$

I_1 = moment of inertia of column

I_2 = moment of inertia of beam regarded as T-section

l = span of beam

h = storey height

E = modulus of elasticity of the material of frame

This formula, given by Salvadori,[25] is useful for calculating drift (i.e. top deflection) in framed buildings.

9.2.1 Example of Calculating of Drift of a Framed Building by Salvadori[25]

A 13-storeyed building has four-bay frames at a spacing of 20′ c/c. Calculate the drift at the top under a wind pressure of 30 psf (Fig. 9.2).

With. $n = 13$, $m = 4$, $h = 12$ ft., $l = 20$ ft., $b = 20'$, $p = 30$ psf.

$W = p.b.h = 30 \times 20 \times 12 = 7200$ tbs.

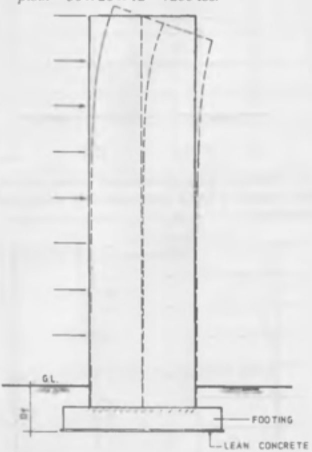


Fig. 9.1a. Behaviour of shear walls and frames under horizontal loads. Cantileverl beam action of shear wall under horizontal loads (bending mode of deflected shape).

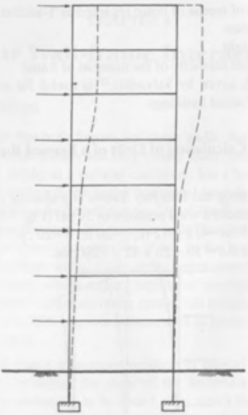


Fig. 9.1b. Behaviour of shear walls and frames under horizontal loads Deflected shape of a frame under horizontal loads (shear mode of deflected shape).

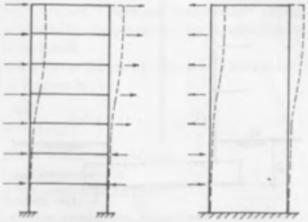


Fig. 9.1c. Behaviour of shear walls and frames under horizontal loads. Interaction of shear wall and frame.

Assuming column size at 1 ft. × 2 ft. and beam size at 1 ft. × 2.00 ft., we have

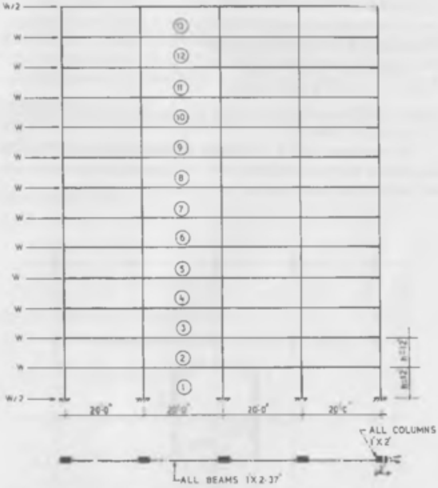


Fig. 9.2. Example of frame deflection.

$$I_1 = \frac{1}{12} \times 1 \times (2)^3 = 0.667 \text{ ft}^4$$

$$\frac{I_1}{h} = \frac{.667}{12} = 0.0555 \text{ ft}^3$$

$$I_2 = \frac{1}{12} \times 1 \times (2.00)^3 \times 1.66 \text{ (T-effect)} = 1.109 \text{ ft}^4$$

$$\frac{I_2}{l} = \frac{1.109}{20} = 0.0555 \text{ ft}^3$$

$$\beta = \frac{I_1/h}{I_2/l} = 1.0$$

$E = 3.0 \times 10^6$ psi

Eq. (9.1) gives the value of drift,

$$\Delta f = \frac{7200 \times (12 \times 12)^3}{24(4+1) \times 3.0 \times 10^6 \times .667 \times (12)^4} [(13)^2 + 1.0 (1 + 2 \times 13 . \overline{13 - 1})]$$

$$= \frac{7200 \times (12)^3 \times (12)^3 \times 482}{24 \times 5 \times 3 \times 10^6 \times .667 \times (12)^4} = 2.08 \text{ in}$$

IS:1893–1975,[9] clause 4.2.3 gives that drift should not exceed 0.004H = .004 × 13 × 12 × 12 = 7.488 in.

The calculated drift is well within the prescribed limit. When the drift value exceeds the maximum prescribed value, it will be necessary to provide shear walls, which will restrain the deflection of the building.

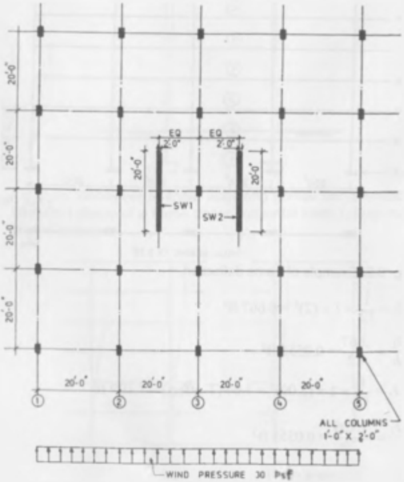


Fig. 9.3. Plan of building with shear walls and frames.

9.2.2 Example of Shear Wall-frame Interaction by Salvadori[25]

Fig. 9.3 gives a plan of building with two shear walls (2 ft. thick and 20 ft. long) and five frames. It has the same number of storeys and bays as given in Ex. 9.2.1. We are required to find the share of wind force resisted by shear walls and frames and also the deflection at the top of the building (i.e. drift) under a wind pressure of 30 psf.

a. *Allocation analysis*

(i) Let the frames carry the entire wind load,

W_f = wind load per frame per ft height

$= 30 \times 20 = 600$ lb/ft

Example 9.2.1 gives

$\Delta_f = 2.08$ in

k_f = stiffness of frame

$$= \frac{w_f}{\Delta_f} = \frac{600}{2.08} = 289 \text{ lb/ft/in}$$

(ii) Let shear walls resist the entire wind force.

W_s = wind load per shear wall per ft. height of building

$$= 30 \times \frac{80}{2} = 1200 \text{ lb/ft}$$

$$\delta_s = \frac{wl^4}{24EI} \text{ (as a guided vertical cantilever beam)}$$

$$= \frac{(1200/12)(13 \times 12 \times 12)^4}{24 \times 3.0 \times 10^6 \times (\frac{1}{12} \times 2 \times 20^3 \times 12^4)}$$

$$= \frac{1200 \times (13)^4 \times (12)^4 \times (12)^4}{12 \times 24 \times 3.0 \times 10^6 \times \frac{1}{12} \times 2\,(20)^3 \times (12)^4} = 0.617 \text{ in}$$

k_s = stiffness of shear wall

$$= \frac{W_s}{\delta_s} = \frac{1200}{0.617} = 1945 \text{ lb/ft/in}$$

(iii) Shear wall frame interaction. There are two shear walls and five frames in the building (Fig. 9.3).

$\Sigma k_s = 2 \times 1945 \quad = 3890$

$\Sigma k_f = 5 \times 289 \quad = 1445$

$\Sigma k \quad = \underline{5335}$

$$\frac{\Sigma k_s}{\Sigma k} = \frac{3890}{5335} \times 100 = 73\%$$

$$\frac{\Sigma k_f}{\Sigma k} = \frac{1445}{5335} \times 100 = 27\%$$

It shows that shear walls resist 73% of the wind shear and the frames 27% of it. Frames have to be designed for not less than 25% of the total wind shear.[11]

b. *Drift Calculations*. Actual wind load on a shear wall

$$w = 30 \times \frac{80}{2} \times 0.73 = 1200 \times .73 = 876 \text{ tb/ft height}$$

Drift = deflection at top of shear wall

$$\frac{wl^4}{24EI}$$

$$= \frac{876}{1200} \times 0.617 = 0.45 \text{ in}$$

Actual wind load on a frame

$$w_f = \frac{1}{5}(30\times80 - 876\times2)$$

$$= \frac{1}{5}(2400 - 1752) = 129.6 \text{ tb/ft height}$$

$$W = 129.6 \times 12 = 1555.2 \text{ tbs (per storey)}$$

drift = deflection at top of frame

$$= \frac{1555.2}{7200} \times 2.08 = 0.449 = 0.45 \text{ in}$$

The drift of the building has been reduced from 2.08 in to 0.45 in by the introduction of the two shear walls.

Scheuller[14] also gives a similar approach for solving the problem of the shear wall-frame interaction. This approximate approach can be used for a preliminary analysis. For the final analysis, computer solutions with the help of the specialists in the field shall have to be used.

CHAPTER 10

Space Structures

10.1 INTRODUCTION

Sloping roofs are, generally, provided for buildings in hilly areas of high altitude, in order that snow does not get accumulated on top. In farm houses too, sloping roofs are preferred by architects, as these easily merge with the surroundings of trees, etc.

Sloping roof slabs may rest on load bearing walls or on reinforced concrete frames. The design of sloping slab panels is to be done in the same way as for level slab panels, with the only change that the dead load is to be divided by cos θ where θ is the angle of the sloping slab with the horizontal. For covering large areas,the sloping slabs may assume shape of a pyramidal roof on square or rectangular plans or even of more complicated shapes on hexagonal or octagonal plans. These roofs may be analysed as space trusses.

10.2 PYRAMIDAL ROOF ON A SQUARE PLAN

Fig. 10.1 gives details of a pyramidal roof on a square plan. It consists of four sloping triangular slab panels, cast monolithic, resting on beams B1, which rest on four columns C1. The triangular slab panels with three sides hinged or fixed can be analysed by tables given on page 175 of Design Handbook by Shaker El-Behairy.[63] Alternatively, a triangular slab panel can be designed as a circular slab of diameter of the inscribed circle, which can be found by the following formulae. For an isoceles triangle of base b and height h (Fig. 10.2a), the diameter d of the inscribed circle is given by,[30]

$$d = \frac{2bh}{b + (b^2 + 4h^2)^{1/2}} \tag{10.1}$$

For a triangle of sides a, b, c the diameter d of the inscribed circle (Fig.10.2b) is given by,[30]

$$d = \frac{2b}{(a+b+c)}\left[a^2 - \left(\frac{a^2+b^2-c^2}{2b}\right)^2\right]^{1/2} \tag{10.2}$$

For an isosceles triangle with $a = c$, Eq. (10.2) reduces to,

$$d = \frac{b}{(2a + b)} (4a^2 - b^2)^{1/2} \tag{10.3}$$

Eqs. (10.1) and (10.3) will lead to the same result.

For a circular slab with hinged support on the periphery, the moment at the centre, under a uniform loading w, is given by,

$$M = + \frac{wd^2}{16} \tag{10.4}$$

For a circular slab with fixed support on the periphery, the moments at the supports and the centre, under uniform loading w, are given as (Fig. 10. 2a),

$$-M = -\frac{wh^2}{30} \tag{10.5}$$

$$+M = +\frac{wd^2}{30} \tag{10.6}$$

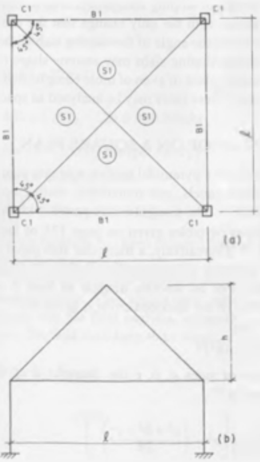


Fig. 10.1. Pyramidal roof on square plant. (a) Plan; (b) Elevation.

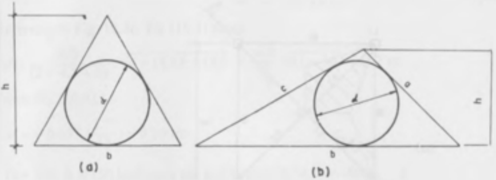


Fig. 10. 2. Diameter of inscribed circle for triangular slab panels. (a) Isosceles triangle; (b) triangle of sides *a*, *b*, *c*.

The tables given by Shaker El-Behairy give results for various loading cases and are more useful. In sloping slab panels, straight bars on top and bottom, bothways are generally used, for ease in placement and for top and bottom bars requirement at corners.

The four sloping slab panels S1 rest on four horizontal beams B1 and this system is in equilibrium and it gives only vertical load on columns C1 (Fig. 10.1). The beams B1 are under bending and axial tension. When the square panel is large, one may provide sloping beams B2, along the sloping edges as shown in Fig .10.3a.

The beam B2 may be assumed to rest on column C1 and at an imaginary support at centre 0, where the reaction of the beam B2 is V. As in reality, there is no support at the centre 0, 4V load acts downwards at 0, which is in equlibrium by compression C in the sloping beams B2 (Fig. 10.3b). From Fig. 10.3b, it is given by considering space truss action,

$$4C \cos\alpha = 4V$$

or, $$C = \frac{V}{\cos\alpha} \tag{10.7}$$

where, $$\tan\alpha = \frac{l.\sqrt{2}}{2h} \tag{10.8}$$

The horizontal components of C at the centre point 0 get cancelled.

The beam B1 spans between columns C1, with the maximum moment at the centre of span and the maximum shear at the supports. The compression C in beam B2 can be resolved in the two perpendicular directions at the point C1 to give tension T in beams B1. From Fig. 10.3 (a,b),

$$T = C\sin\alpha \cos 45° = 0.707C \sin\alpha \tag{10.9}$$

$C\cos\alpha$ goes directly to the column C1, adding to its load. The four beams B1, in plan, form a sort of a closed square ring under tension and bars at the corners shall be well-lapped, so that tension can pass on all along the ring.

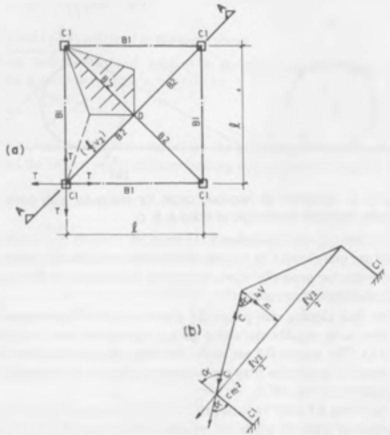


Fig. 10.3. Pyramidal roof on square plan with sloping beams. (a) Plan of roof; (b) Section A-A through a diagonal.

10.2.1 Numerical Example

Let $l = 6.0$ m, $h = 1.5$ m (Fig. 10.4)

a. Slab S1

$$
\begin{aligned}
w: \text{self weight } 0.10 \times 2500 &= 250 \text{ kg/m}^2 \\
\text{Mangalore tiles on top} &= 125 \text{ kg/m}^2 \\
\text{Water profing on top} &= 50 \text{ kg/m}^2 \\
\text{Ceiling plaster} &= 25 \text{ kg/m}^2 \\
M &= 400/\cos\theta \\
&= \frac{400}{\cos 26.570} = 500 \text{ kg/m}^2
\end{aligned}
$$

$$LL = \underline{75 \text{ kg.m}^2}$$

$$W = \underline{575 \text{ kg/m}^2}$$

Referring to Fig. 10.4c, Eq. (10.3) gives,

$$d = \frac{6.0}{(2 \times 4.5 + 6)} \sqrt{4 \times (4.5)^2 - (6)^2} = \frac{6.0}{10} \overset{1}{\sqrt{81 - 36}} = 2.7 \text{ m}$$

From Eq. (10.4),

$$M = +\,0.575\,(\frac{2.7^2}{16}) = 0.26 \text{ tm}$$

$D = 100$, $\bar{\phi}$ 8/150 bothways top and bottom to be provided.

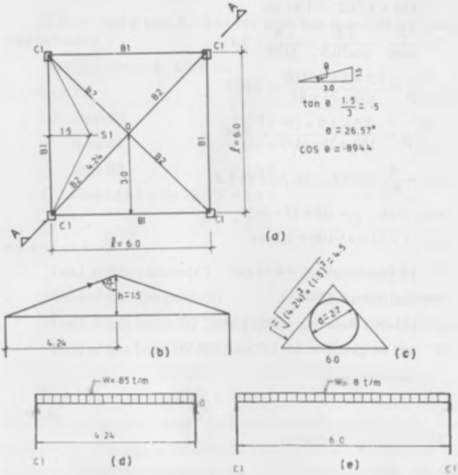


Fig. 10.4.Pyramidal roof of a numerical example. (a) Plan; (b) Section A-A; (c) Diameter of inscribed circle; (d) Long section of beam B2; (e) Long section of beam B1.

Total load of four slab panels $S_1 = 0.575 \times 6.0 \times 6.0 = 20.7$ t

Load on beam B1 $= \dfrac{.575 \times 1.5}{2} \times 6.0 = 0.45 \times 6.0 = 2.7$ t

Load on four beams B1 $= 2.7 \times 4 = 10.8$ t

Load on four beams B2 $= 20.7 - 10.8 = 9.9 = 10$ t

Load on a beam B2 $= 10/4 = 2.5$ t

b. *Beam* B2 (23 × 45). Referring to Fig. 10.4d.

$W =$ self weight $.23 \times .45 \times 1.0 \times 2.5 = .26$ t/m

from slab S1 2.5/4.24 $= 0.59$ t/m

$= \overline{\underline{0.85}}$ t/m

$M = 0.85 \times 4.24^2/8 = 1.9$ tm

$V = 0.85 \times 4.24/2 = 1.81$ tm

$$C = \frac{V}{\cos\alpha} = \frac{1.8}{\cos 70.5} = \frac{1.8}{.3333} = 5.4 \text{ t}$$

$$\frac{P_u}{f_{ck}bD} = \frac{1.3 \times 5.4 \times 1000}{150 \times 23 \times 45} = 0.052$$

$$\frac{M_u}{f_{ck}bD^2} = \frac{1.5 \times 1.9 \times 10^5}{150 \times 23 \times (45)^2} = 0.04$$

$$d'/D = \frac{4}{45} = 0.089 \approx .10, \text{ As/2 on E.F.,}$$

$p/f_{ck} = .02, \quad p = .02 \times 15 = 0.3$

$A_s = .3 \times 23 \times 45/100 = 3.1$ cm^2

2 ϕ 12 *T* and *B* give $A_s = 4.52$ cm^2

Nominal stirrups 8/200 c/c

c. *Beam* B1 (23×60). Referring to Fig. 10.4e.

$W =$ self weight $.23 \times .6 \times 1.0 \times 2.5 = 0.35$ t/m

from slab S_1 $.575 \times 1.5/2$ $= 0.45$ t/m

W $= \overline{\underline{0.80}}$ t/m

$$M = 0.8 \times \frac{6.0^2}{8} = 3.6 \text{ tm}$$

$$S = .8 \times \frac{6.0}{2} = 2.4 \text{ t}$$

$T = .707\ C\sin\alpha$

$$= 707 \times 5.4 \times \sin 70.5^0 = 3.6 \text{ t}$$

$$\frac{P_u}{f_{ck}bD} = \frac{-1.5 \times 3.6 \times 1000}{150 \times 23 \times 60} = -.026$$

$$\frac{Mu}{f_{ck}bD^2} = \frac{1.5 \times 3.6 \times 10^5}{150 \times 25 \times (60)^2} = 0.04$$

$$\frac{d'}{D} = \frac{4}{60} = .067 \approx .10, \text{ As/2 on E.F.}$$

$$\frac{p}{f_{ck}} = .035, \text{ p} = .035 \times 15 = 0.525$$

$$A_s = .525 \times 23 \times \frac{60}{100} = 7.2 \text{ cm}^2$$

2 $\bar{\phi}$ 16 T and *B* give $A_s = 8.04$ cm² in all four beams B1 with proper lapping at corners.

Nominal stirrups $\bar{\phi}$ 8/200 c/c

d. *Column* C1

P : from B1	= 2.4 t
from B1	= 2.4 t
from B2	= 1.8 t
*C*cosα (5.4 × cos 70.5°)	= 1.8 t
P	= $\overline{8.4}$ t

e. *Check on loads*

Load on four columns C1	= 4 × 8.4 = 33.6 t
Load of four slab panels S1	= 20.7 t
Load of four beams B2 .26×4.24×4	= 4.4 t
Load of four beam B1 .35×6×4	= 8.4 t
	$\overline{33.5}$ t ≈ 33.6 t, OK

10.3 PYRAMIDAL ROOF ON A RECTANGULAR PLAN

Fig. 10.5 gives the plan and elevations of the proposed roof structure. The slab panels S1 are one-way spanning, while S2 are triangular panels, which may be designed as circular slabs with hinged periphery. The beams B1 and B2 may be taken as supported on an imaginary support at 0. The sum of reactions at 0 of beams B1 and two B2 beams may be designated by *V*, which is then applied as load at 0, to be supported by space truss action at the joint 0. It is assumed that load $V/2$ will be taken by one beam B2 and B1 and the rest of load $V/2$ will be taken by another beam B2 and B1. Referring to Fig. 10.5d, we get by the Sine Rule

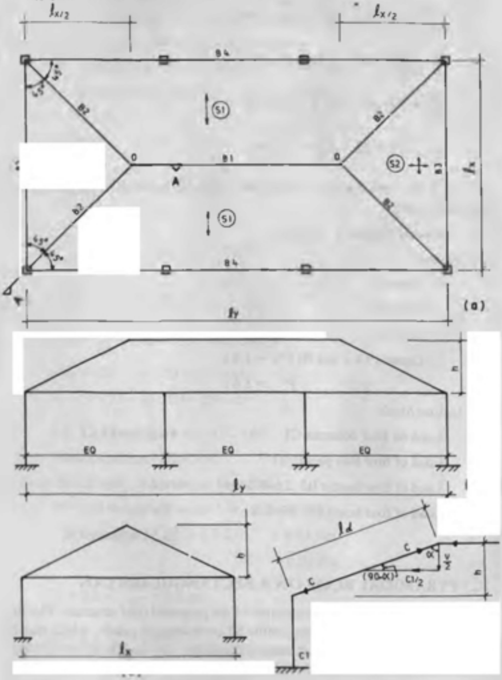


Fig. 10.5. Pyramidal roof on a rectangular plan. (a) Plan of roof; (b) Elevation of roof; (c) Side view of roof; (d) Section of A-A along the diagonal.

$$\frac{C}{\sin 90^\circ} = \frac{C/2}{\sin\alpha} = \frac{V/2}{\sin(90-\alpha)}$$

$$C = \frac{V}{2\cos\alpha} \tag{10.10}$$

$$C_1 = V\tan\alpha \tag{10.11}$$

where, $\cos\alpha = \dfrac{h}{ld}$ (10.12)

Beams B1 and B2 are all under bending combined with axial compression. Now as in section 10.2, the columns loads will be increased by $C\cos\alpha$ and tension T in beams B3 and B4 shall be,

$$T = C\sin\alpha \cos 45^\circ = 0.707\, C\sin\alpha \tag{10.13}$$

We may understand this system better, by following a numerical example.

10.3.1 Numerical Example

Let $l_x = 8$ m, $l_y = 16.0$ m, $h = 2.0$ m (Fig. 106a)
a. *Slab design*

S1: W = self wt. $0.12 \times 2500 = 300$ kg/m²

Mangalore tiles $.05 \times 2000$	$= 100$ kg/m²
Water proofing	$= 50$ kg/m²
Ceilling plaster	$= 25$ kg/m²
DL	$= \overline{475} / \cos\theta$
	$= 475/\cos 26.565^\circ = 531$ kg/m²
	$= 75$ kg/m²
W	$= \overline{606}$ kg/m²
(say)	$= 610$ kg/m²

Referring to Fig 10.6b,

$M = \pm 0.61 \times 4.0^2/10 = \pm 0.98$ tm

$D = 120$mm, ϕ 8/100 c/c top and bottom, capacity = 1.052 tm

Distribution steel ϕ 8/200 c/c top and bottom.

S₂: Triangular slab with a, b, c in the inclined plane as given in Fig. 10.6c:
$a = c = 6.0$m, $b = 8.0$m
Eq. (10.3) gives

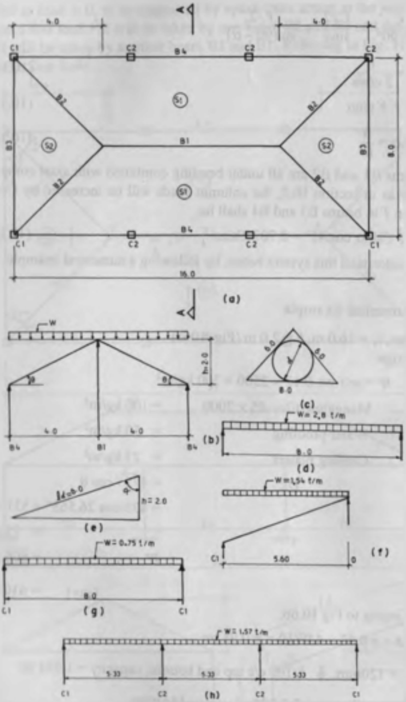


Fig. 10.6. Details of numerical example of 10.3.1. (a) Plan of roof; (b) section A-A (design of slab S1); (c) Triangular slab S2 with dimensions of sides *a*, *b*, *c* in the inclined plane; (d) Design of beam B1; (e) Evaluation of angle α; (f) Design of beam B2; (g) Design of beam B3; (h) design of beam B4.

$$d = \frac{8.0}{(12+8)}\sqrt{4\times(6)^2-(8)^2} = \frac{8}{20}\sqrt{144-64} = 3.6\text{ m}$$

$$M = \frac{+wd^2}{16} = +0.61\times\frac{3.6^2}{16} = +0.49$$

$D = 120$ mm ϕ 8/150 c/c both ways T and B

Capacities = 0.738, .674 (tm)

b. *Beam design*

B1 (23 × 60) Referring to Fig. 10.6d,

W: self wt. $.23\times.6\times1.0\times2.5$	$= 0.35$ t/m
Slab S1 $0.61\times4/2\times2$	$= 2.44$ t/m
	$= \underline{2.79} = 2.8$ t/m

$M = 2.8\times8.0^2/8$	$= 22.4$ tm
$S = 2.8\times8.0/2$	$= 11.2$ t
$C_1 = V\tan\alpha = 20.0\times2.828$	$= 56.6$ t

$V = 20$t will be made clear after B2.

Referring to Fig. 10.6e,

$\cos\alpha = 2/6 = 1/3 = .3333$, $\alpha = 70.5°$

$\tan\alpha = 2.828$, $\sin\alpha = .9426$

$$\frac{P_u}{f_{ck}\,bD} = \frac{1.5\times56.6\times1000}{150\times23\times60} = 0.41$$

$$\frac{M_u}{f_{ck}\,bD^2} = \frac{1.5\times22.4\times10^5}{150\times23\times(60)^2} = 0.27$$

$d'/D = 4/60 = .067 \approx .10$, $\quad A_s/2$ on E.F.

$p/f_{ck} = .20$, $\quad p = .20\times15 = 3.0$

$$A_s = 3.0\times23\times\frac{60}{100} = 41.4\text{ cm}^2$$

4 25 ϕ T and B give $A_s = 39.26$ cm², nearly OK.

$$T_v = \frac{1.5\times11.2\times1000}{23\times56\times10} = 1.3\ \text{N/mm}^2$$

For $p_t = 1.5$, $\qquad T_c = .68$ N/mm²

$V_{us}/d = (.13 - .068)\times23 = 1.426$ kN/cm

Stirrups ϕ 8/200 c/c $= 1.815$ kN/cm

B2 (23 × 60): Referring to Fig. 10.6f

W: Self-weight	= .35 t/m
slab S1 .61 × 2/2	= .61 t/m
slab S2 3.3/5.66	= .58 t/m
	1.54 t/m

$M = 1.54 \times 5.66^2/8 = 6.2$ tm

$S = 1.54 \times 5.66/2 = 4.4$ t

$$C = \frac{V}{2}\frac{1}{\cos\alpha} = \frac{20.0}{2} \times \frac{1}{1/3} = 30.0 \text{ t}$$

$V = 11.2 + 4.4 \times 2 = 20.0$ t (used above for design of B1)

$$\frac{P_u}{f_{ck}\,bD} = \frac{1.5 \times 30 \times 1000}{150 \times 23 \times 60} = .22$$

$$\frac{M_u}{f_{ck}\,bD^2} = \frac{1.5 \times 6.2 \times 10^5}{150 \times 23 \times (60)^2} = .075$$

$d'/D = .10, \quad p/f_{ck} = .02, \quad p = .3, A_s = 0.3 \times 23 \times \frac{60}{100} = 4.14$ cm²

2 ϕ 12 T and B give $A_s = 4.52$ cm², nominal stps. ϕ 8/200 c/c.

Additional load on column = $C \cos\alpha$

$= 30 \times .33 = 10.0$ t

Tension (T) in beams B3 and B4 is given by,

$T = .707\ C.\sin\alpha$

$= 0.707 \times 30.0 \times .9426 = 20.0$ t

B3 (23 × 60): Referring to Fig. 10.6g,

$W = .35 + .4$ (slab) $= 0.75$ t/m

$M = .75 \times 8.0^2/8 = 6.0$ t/m

$S = .75 \times \frac{8.0}{2} = 3.0$ t

$T = 20.0$ t

$$\frac{P_u}{f_{ck}\,bD^2} = -\frac{1.5 \times 20 \times 1000}{150 \times 23 \times 60} = -0.145$$

$$\frac{M_u}{f_{ck}bD^2} = \frac{1.5 \times 6.0 \times 10^5}{150 \times 23 \times (60)^2} = 0.07$$

$d'/D = .10, \quad p/f_{ck} = .09, p = .09 \times 15 = 1.35$

$$A_s = 1.35 \times 23 \times \frac{60}{100} = 18.63 \text{ cm}^2$$

$2 \bar{\phi}$ 25 T and B give $A_s = 19.63$ cm^2

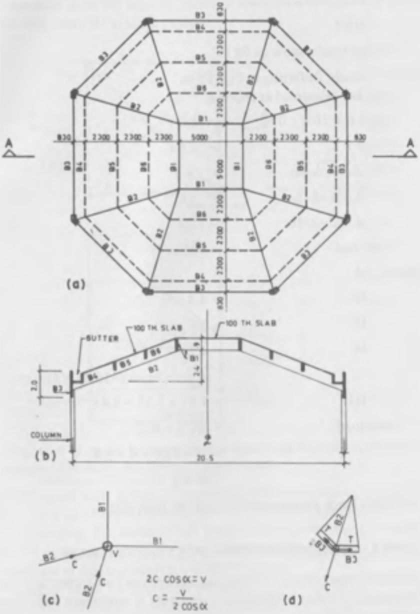


Fig. 10.7. Pyramidal roof on an octagonal base. (a) Plan; (b) Section A-A; (c) Joint V; (d) Joint B3.

Nominal strrups at ϕ 8/200 c/c

B4 (23 × 60): Referring to Fig. 10.6h

$W = .35 + 1.22$ (slab) $= 1.57$ t/m

$M = \pm\ 1.57 \times (5.33)2/10 = \pm\ 4.5$ t/m

$S\ = 1.57 \times 5.33/2 = 4.2$ t

$T = 20.0$ t

Design results same as for B3

c. *Check on loads*: Referring to Fig. 10.6a

Loads are calculated as follows:

Slab 8.0 × 16.0 × 0.61	= 78.0 t
B1 0.3 × 8	= 2.8 t
4 × B2 .35×4.×6	= 8.4
2 × B3 .35×2×8	= 5.6
2 × B4 .35×2×16	= 11.2
Total load	= $\overline{\underline{106}}$ t

Columns loads

C1: B2	= 4.4
B3	= 3.0
B4	= 4.2
	$\overline{\underline{11.6}} + C\cos\alpha\ (=10.0) = 21.6$ t
C2 : B4	= 1.57 × 5.33 = 8.4 t
Total load	= 4C1 + 4C2
	= 4 × 21.6 + 4 × 8.4
	= 86.4 + 33.6 = 120 t ≈ 106 t

Column loads are assessed 12% on the high side

10. 4 PYRAMIDAL ROOF ON A POLYGONAL BASE

For providing interesting roofs over auditoriums or conference halls, pyramidal roofs may also be considered. For covering a conference hall of approximately 20.0m × 20.0m in plan, a novel type of roof was suggested. It was designed on the basis of a space truss and it is built and it is working satisfactorily. Fig. 10.7 gives its plan and a central section. The slab thickness has been kept at 100mm. Beams B4, B5, B6 are continuous ring beams resting on sloping beams B2. Beams B1 are supported on corners of beams B2, which are under axial compression together with bending. The outer ring beam B3 is monolithic with columns. It is under axial tension together with bending. In reality, the upper two columns are placed about 2.0m away from the column positions shown in Fig. 10.7, as they were interfering with the stage function. Heavy brackets from the receded columns have been brought out, to support the outer ring beam B3 at these places.

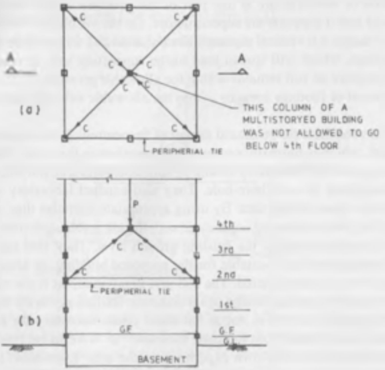


Fig. 10.8. Scheme for supporting a column of multistorey building above ground floor level. (a) Plan; (b) Section A-A.

The space truss action of members has been utilized to support a column, which was otherwise coming in the centre of an entrance hall in a multistoreyed building. The entrance hall has a height of four storeys, out of which, upper two storeys are utilised in the space truss height (Fig. 10.8). It gives a majestic look to the main entrance of the building.

General ideas for design of non-planar roofs are given by Terrington and Turner [64] and a strict solution for a pyramidal roof is given by Kalra.[65]

CHAPTER 11

Foundations

11.1 INTRODUCTION

Foundation or sub-structure is that part of the structure which lies buried in the ground and it supports the superstructure, i.e. the visible part of the building under usage. All vertical elements like columns and walls will be provided with footings, which will spread load to the supporting soil, in such a way, that the pressure on soil remains within the allowable pressure of soil and also the settlement of footings remains within the allowable value of settlement of soil.

A pre-requisite for structural design of foundations is the soil investigation report, which is normally prepared by specialists in this field. They carry out investigations of the site by means of bore-holes and take out soil samples at various depths in each bore-hole. They also conduct laboratory tests and find out the values of soil data. By using appropriate formulae they work out values of the net safe bearing capacity of soil, the allowable settlement and the depth of foundation below the existing ground level. They also suggest the type of foundations most suitable for the proposed building, in keeping with the type of the soil encountered. The soil investigation report is the most vital input for a proper and safe design of foundations. Earlier, say in the fifties, the soil investigation was not in vogue but these values necessary for design of foundations were taken from the code IS:1904,[66] or were to be fixed by the engineer depending on his own experience in the area. Even now, for small buildings, no soil investigation is undertaken. But for all important projects, the soil investigation is now regarded as a must.

Many types of soils are found in the sub-strata. Alluvial soil consisting predominantly of sandy soil is quite wide-spread. Clayey soils are also found in certain areas. Black cotton soil is a particular type of expansive clayey soil, which needs special care. Lastly, rocks are also found in certain areas. Footings on rocks work out quite economical, as these require less areas in plan. Sandy or non-cohesive soils have safe bearing capacities of 10 t/m^2 to 20 t/m^2, when dry. When, however, the water table is high and it is above or near the base of footings, the above values will be halved. For cohesive or clayey soils, the safe bearing capacity varies from 5 t/m^2 to 15 t/m^2. For rocky soil, the safe bearing capacity varies from 45 t/m^2 to 90 t/m^2 or even more. The values given above are only indicative and the final value which is to be be adopted for design, will be given by the soil investigation report. The best soil for a foundation is rock. Next to rock, it is sandy soil, where, the soil settlement takes place quickly. The clayey soil is not much relied on, as in clay; settlement keeps on taking place for a long time. Black cotton soil is an expansive clayey soil and special care has to be taken to design foundation elements. The worst soil for foundation support is the filled-up soil, which is to be avoided or it is to be scooped out and replaced by lean concrete.

The minium depth of foundation below the existing ground is 0.6 m. It is important to bury fully the footings in the ground. If the footing depth is 0.30 m, with 100 mm. of lean concrete and 150 mm. of earth cover on top of footing, the foundation depth (D_f) works out (Fig.11.1)

$$D_f = 0.15 + 0.30 + 0.1 = 0.55 \text{ or } 0.60 \text{ m.}$$

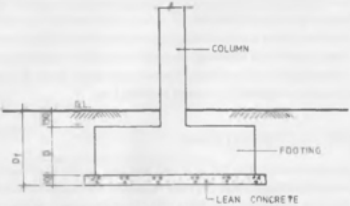


Fig. 11.1. Minimum foundation depth.

But in practice, the foundation depth is kept at 0.9m or even more. For buildings of six storeys or more, it may be kept at 1.5 m to 2.0 m. For tall isolated structures, like water towers, the foundation depth may be at 3.0 m also. The above values for depth of foundation of buildings has only intuition as the basis. The important thing to consider is to put the footing base on the natural soil and not on any filled-up soil. All footing trenches shall be carefully inspected before putting lean concrete, in order to ensure that the supporting soil is the natural soil and not a filled up one. If a fill is encountered, one may go deep further or the soil is scooped out and replaced by lean concrete. Normally all footings are kept at one level or at two or three levels, if the building is long and the ground is sloping.

Foundations can be of two types: (i) Shallow; (ii) Deep.

Shallow foundations consist of the common isolated footings, combined or strip footings or rafts, where the earth is excavated to form footing trenches and then reinforced concrete footings are cast. Most of the buildings require only shallow foundations. In areas of high water table, dewatering methods have to be used to keep the trenches dry. When the footings are cast and cured and sufficient top load of the structure comes on the footings, then dewatering is stopped. When the buildings are tall, raft foundations are provided to support the columns loads. When, however, large rafts are required in case of poor soil capacity or heavy column loads pile foundations have to be provided, which come under *deep foundations*. Piles are long vertical columns in the soil, which transfer the column load to the soil strata below by direct bearing at base and also by the skin friction with the surrounding soil, all along the pile depth in the soil. Pile foundations have a settlement of 12 mm or so, while the raft foundations have large settlement of 50 mm or even more. Pile foundations work out costlier than a raft foundation for a given case as piling contractors are not as many in the field as the general contractors, who can do the raft foundation as well as any other shallow foundation. Further, pile driving involves a lot of heavy machinery, which needs more capital investment. For tall buildings, piles are preferred to rafts, in order mainly to restrict settlement. Under-reamed piles are short-length piles suitable for black cotton soils, as these piles are capable of resisting tension due to the construction of one or more bulbs. Normal piles of uniform diameter can resist loads in compression only and their tension resistance is not to be relied on.

11.2 SHALLOW FOUNDATIONS

Soil investigation report gives the net safe bearing capacity of soil, which is calculated from the ultimate soil bearing capacity, by using a factor of safety of 3.0 to 2.5[66]. The practice of taking 10% extra load for the self weight of footing is now obsolete, as the weight of the footing only displaces the weight of the earth excavated . The column or wall load without any extra increase should be considered for calculating footing area. This is made clear in the clause 5.2 of IS: 1904-1978.[67]

The most economical footing type is an isolated column footing. It can be made of uniform depth or sloping depth or stepped depth. The design of isolated footings has been given in Manual.[17] Bowles[68] has advocated use of isolated footings, even when one footing just touches another one. But in some old texts, one comes across a statement, that when isolated footings occupy more than 50% of the floor area of building, a raft foundation should be used. This becomes a costly prosposition. When the isolated footings are put side by side, bulbs of pressure will interact (Fig. 11.2), this effect will slightly reduce the factor of safety of 3.0 to 2.5 to, say, a value of 2.8 to 2.3, which is still quite adequate. It is therefore, recomended to have isolated footings for columns, wherever possible.

Reinforced concrete columns and the brick walls may be given their own footings. But when the foundation depth is large, plinth beams may be provided to support brick walls, by which, the brick work below plinth can be largely saved. In earth quake-prone areas, footings are required to be tied in the two principal directions. Plinth beams, then, can serve the action of earthquake ties as well.

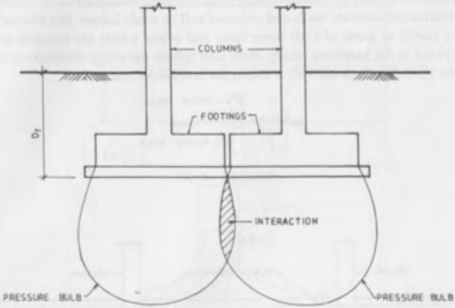


Fig. 11.2. Interaction of pressure bulbs.

In the current practice, plinth beams are being increasingly provided in buildings. Plinth beams can be designed for column base moments under both the vertical and the horizontal loads, so that footings can then be designed for column axial loads only (Fig. 11.3).

When the adjoining isolated footings overlap, a combined footing for the adjoining two columns is the next economical solution. Combined footings with beams either in the long or in the cross direction (Fig. 11.4A, B) are clear in their behaviour, but a combined footing with a slab of uniform depth (Fig. 11.4C) is not clear in its structural action, unless we assume hidden beams FB3 in the cross direction. This is made clear by Hilal.[69]

A strip foundation is the extension of a combined footing for a row of columns. The central beam is to be provided with cantilever slab on either side of it (Fig.11.5). For the design of the longitudinal beam FB1, the system in Fig.11.5b is considered, which is under equilibrium with column loads from above and the soil reaction from below. The moments and shears at each section are calculated by simple statics for all sections from left side to the centre and also from right side to the centre. Further, an alternate bending moment diagram equal to $\pm Pl/10$ is also to be considered. This approach of strip footing design is given by Reynolds.[70] When these columns form part of a basement retaining wall, the reinforced concrete wall acts as the longitudinal beam FB1 and it leads to an economical design Fig. 11.6. The two alternative sections A-A, given in Fig. 11.6b, c offer two arrangements. Alternative I saves materials but it is complicated for water proofing treatement, while alternative II is easy for waterproofing purposes but it consumes more materials as reinforced concrete walls and columns will be made longer. But alternative II is useful in areas of high water table and where toilets are required to be provided in the basement storey. Base slab in both the above alternatives connects all the footings and this is useful for earthquake resistance.

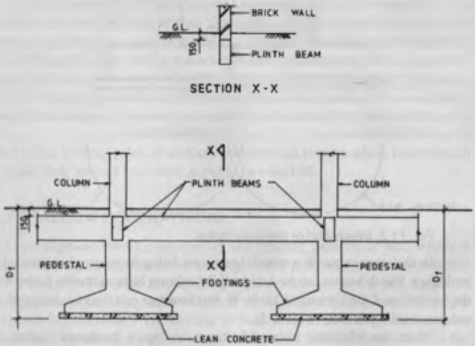


Fig. 11.3. Use of plinth beams.

When the strip footings overlap, it leads to a raft foundation. Initially, one tries to provide a few small-area rafts as this leads to economy. But a situation may arise, when one single raft covering all the columns of building may have to be provided. This is a costly proposition, but it has to be adopted, as other less costly alternatives have already been examined and rejected for the various difficulties. A slab type raft is commonly adopted (Fig. 11.7b), for ease in design and construction, but it consumes more steel. Beam-slab type raft (Fig. 11.7c) saves steel bul requires more depth of foundation. Fig. 11.7d gives a section of cellular raft foundation, in which the weight of raft foundation is reduced. This type of raft is useful in areas, where the safe bearing capacity of soil is small, say 5 t/m^2 to $10t/m^2$. It is also called a buoyant foundation and its design is given by Faber and Mead.[71] Annular raft foundation with cut-outs in plan is also used for isolated tall structures or for buildings in areas with high safe bearing capacity of soil (Fig. 11.8). Annular rafts are useful for columns at the property edge, for which a reference may be made to a paper by Varyani.[72]

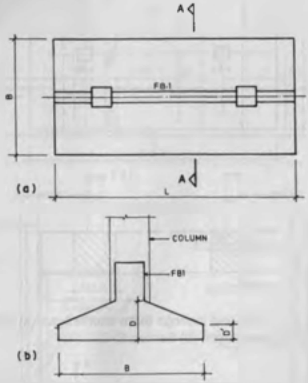


Fig. 11.4A. Combined footings three alternatives. (a) Plan of combined footing with one longitudinal footing beam FB1; (b) Section A-A.

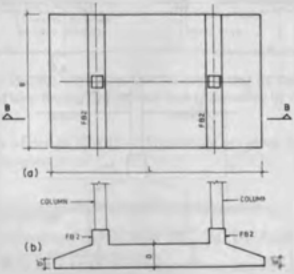


Fig. 11.4B. Combined footings with three alternatives. (a) Plan of combined footing with two cross beams at column points; (b) Section B-B.

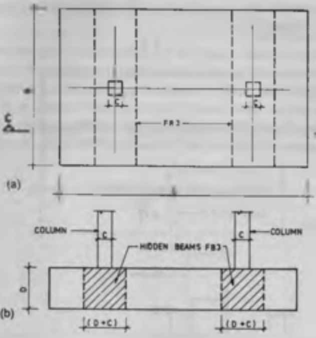


Fig. 11.4C. Combined footings three alternatives. (a) Plan of combined footing of uniform depth; (b) Section C-C.

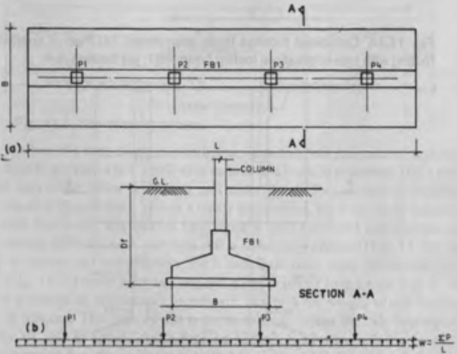


Fig. 11.5. Strip footings with central beam FBI. (a) Plan of strip footing; (b) section A-A and longitudinal action of beam FB1.

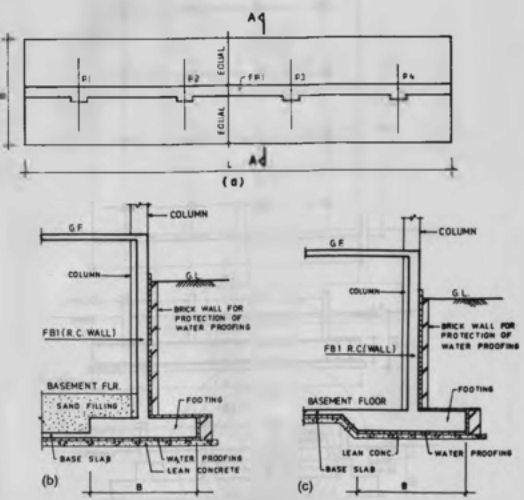


Fig. 11.6. Strip footings with basement RC wall acting as central beam FBI. (a) Plan of strip footing; (b) Section A-A (Alternative II); (c) Section A-A (alternative I).

A few example of design of shallow foundations are given below to illustrate the above principles.

11.2.1 Example of a Combined Footing (Three Alternatives)

i) *Alternative with longitudional footing beam FB1*. Fig. 11.9A gives the plan of a combined footing with the following data:

$P_1 = 100$ t, $P_2 = 150$ t, $\Sigma P = 250$ t, $p_a = 15$ t/m^2

Column size 450 mm × 450 mm

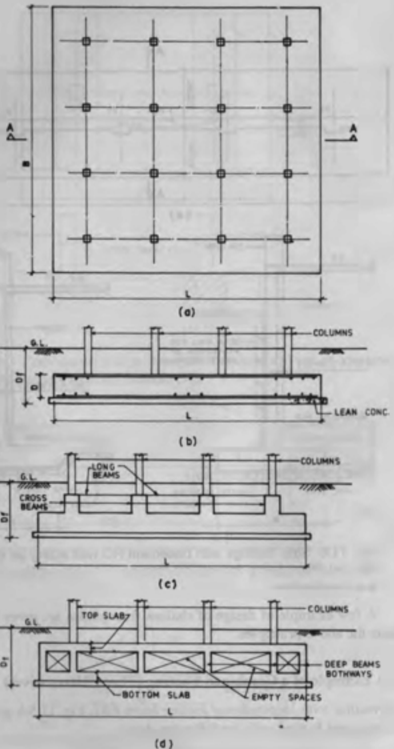


Fig. 11.7. Raft foundation – different types. (a) Plan; (b) Solid slab type raft; (c) Slab-beam type raft; (d) Cellular raft or bouyant foundation.

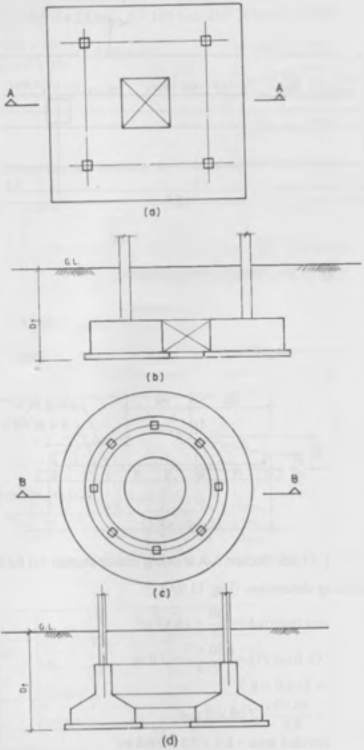


Fig. 11.8. Annular raft foundation. (a) Plan with soil capacity high; (b) Section A-A. (c) Annular raft for a tower; (d) Section B-B.

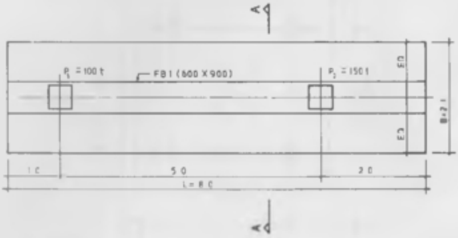


Fig. 11.9A. Plan of combined footing.

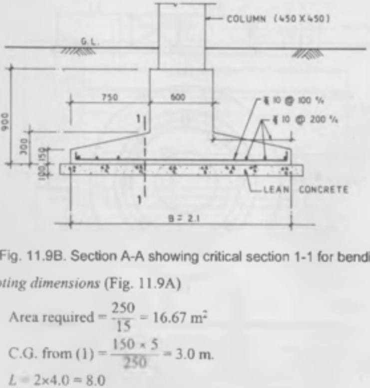


Fig. 11.9B. Section A-A showing critical section 1-1 for bending of slab.

a) *Footing dimensions* (Fig. 11.9A)

$$\text{Area required} = \frac{250}{15} = 16.67 \text{ m}^2$$

$$\text{C.G. from (1)} = \frac{150 \times 5}{250} = 3.0 \text{ m.}$$

$$L = 2 \times 4.0 = 8.0$$

$$B = \frac{16.67}{8.0} = 2.08 \approx 2.1 \text{ m.}$$

$$\text{Provided area} = 8.0 \times 2.1 = 16.8 \text{ m}^2$$

b. *Slab design* Referring to Fig 11.9B.

$M_{1\text{-}1} = 15\times .75^2/2 = 4.22$ tm, $b = 100$ cm, $D = 30$ cm, $d = 25$cm,

$$K = \frac{1.5\times 4.22\times 10^5}{100\times (25)^2\times 10} = 1.0 \text{ N/mm}^2$$

$$A_{st} = 0.303\times 25 = 7.57 \text{ cm}^2\text{/m}$$

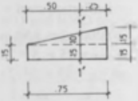
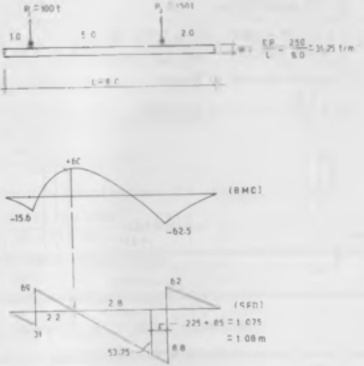


Fig. 11.9C. Critical Section for shear 1′– 1′ for shear design of slab.



M	−15.6	+60	62.5	(tm)
K = .0346M	0.54	2.08	2.16	(N/mm²)
As = 51 Pt	10.51 (MIN)	33.2	31.1 38.8	(cm²)
As	4 4 (19.63)	8 4	4 8 (39.2)	ALL ϕ 25 BARS

Fig. 11.9D. Design of footing beam FB1.

10/100c/c = 7.85 cm²/m

Distribution steel $= .12 \times \dfrac{(30 + 15)}{2} = 2.7$ cm²/m

10/200 c/c = 3.93 cm²/m

$S_{\text{Y-Y}} = 15 \times .5 = 7.5$ t/m

$M_{\text{Y-Y}} = 15 \times .5^2/2 = 1.875$ tm/m

$$T_v = \frac{V_u}{bd} \pm \frac{M_u}{bd^2} \cdot \tan\beta$$

Referring to Fig. 11.9C, we have $\tan\beta = 15/75 = 0.2$, $b = 100$ cm;

$D = 25$ cm, $d = 20$ cm.

$$T_v = \frac{1.5 \times 7.5 \times 1000}{100 \times 20 \times 10} - \frac{1.5 \times 1.875 \times 10^5}{100 \times 20^2 \times 10} \times 0.2$$

$$= 0.56 - 0.14 = 0.42 \text{ N/mm}^2$$

$$p_t = \frac{7.85}{20} = .39, \quad T_c = .42 \text{ N/mm}^2$$

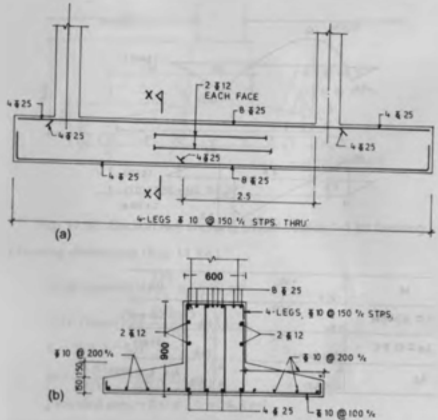


Fig. 11.9E. Detailing of footing beam FB1. (a) Plant; (b) Section X-X

Therefore, $D = 30$ cm to 15 cm, is safe in shear, with a botton mesh of steel ϕ 10/100 in small direction and ϕ 10/200 along the long direction.

c. *Footing beam* (FB1) *design*. Fig 11.9D gives SFD and BMD together with the steel required at the various critical sections and the bars to be provided. With $b = 60$ cm; $D = 90$ cm; $d = 85$ cm;

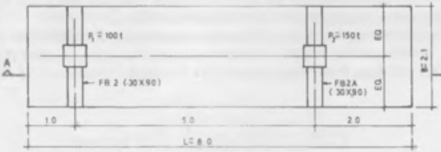


Fig. 11.10A. Plan of combined footing.

$$k = \frac{1.5 \times M\,(\text{tm}) \times 10^5}{60 \times (85)^2 \times 10} = .0346\ \text{M (tm)}$$

$$A_s = p_t \cdot \frac{60 \times 85}{100} = 51\ \text{pt (cm}^2\text{)}$$

Max shear at the critical section = $(2.8 - 1.08) \times 31.25 = 53.75$ t

$$T_v = \frac{1.5 \times 53.75 \times 1000}{60 \times 85 \times 10} = 1.58\ \text{N/mm}^2$$

With $p_t = 0.77$, $T_c = 0.54$ N/mm^2

$$\frac{V_{us}}{d} = (.158 - .054) \times 60 = 6.24\ \text{KN/cm}$$

4 legs stirrups 10/150 = $2 \times 3.781 = 7.562$ KN/cm, thro'.

Side face steel = $.1 \times 60 \times 90/100 = 5.4$ cm^2 given by $4\phi 12 = 4.52$ cm^2. Fig. 11.9E gives the detailing of the beam FB1.

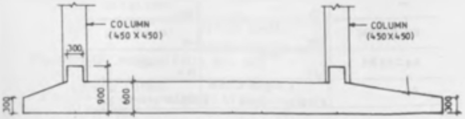


Fig. 11.10B. Section A–A longitudinal section of footing.

d. *Concrete Quantity*

FB1: $.6 \times .9 \times 8.0 \qquad = 4.32$ m^3

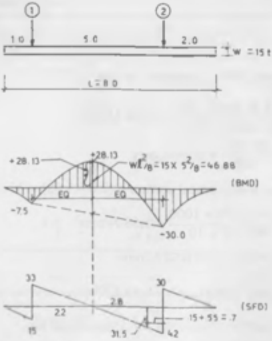
$$\text{Slab: } \frac{3 + .15}{2}(2.1 + .6) \times 8 = 2.7 \text{ m}^3$$

$$\overline{7.02} \text{ m}^3$$

ii) *Alternative with footing cross beams* (2 nos FB2)

a) *Footing dimensions*: Plan dimensions 8.0 × 2.1 m are kept as before. Fig. 11.10A gives the plan with the two cross beams FB2 and FB2A and Fig. 11.10B gives a longitudinal section through the footing.

b) *Slab design*: Fig. 11.10C gives the analysis and design of slab of $D = 60$ cm with variable steel as shown in the above figure. For check in shear (Fig. 11.10C), the shear at the critical section = 31.5 t, with $b = 100$ cm, $d = 55$ cm.



M	-7.5 +28.13	-30 (tm)
K = .0496M	0.37 1.4	1.49 (N/mm²)
As = 55 pt	6.27 24.31	26.4 (cm²)
As	ϕ 20 @ 240 c/c = 13.09 ϕ 20 @ 120 c/c = 26.18	ϕ 20 @ 120 c/c = 26.18 (cm²) D = 60

Fig. 11.10C. Design of slab.

$$T_v = \frac{1.5 \times 31.5 \times 1000}{100 \times 55 \times 10} = 0.86 \text{ N/mm}^2$$

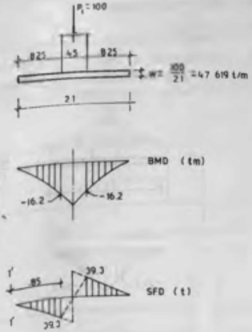
With Φ 20/120 c/c $\quad A_s = 26.18\ \text{cm}^2, p_t = \dfrac{26.18}{55} = 0.476$

$T_c = .45\ \text{N/mm}^2 \times 1.2$ (age factor)

$= 0.54\ \text{N/mm}^2$

i.e. $D = 60$ cm fails in shear. We must provide shear stirrups.

c. *Cross beams design* (FB2 and FB2A). Fig. 11.10D gives the SFD and BMD of the beam FB2 and its design. with $b = 30$ cm, $D = 90$ cm, $d = 85$ cm:



M	-16.2	-16.2	(tm)
K = .0692 M	1.12	1.12	(N/mm²)
As = 25.5 pt	8.75	8.75	(cm²)
As		2 φ 16 TOP NOMINAL 2 φ 25 BOTTOM = 9.81 cm²	

Fig. 11.10D Design of FB2 (.30 × .90).

$$k = \frac{1.5 \times M \times 10^5}{30 \times (85)^2 \times 10} = 0.0692\ \text{M (tm)}.$$

With $M = 16.2$ tm, $k = 1.12\ \text{N/mm}^2$

$A_s = 8.75\ \text{cm}^2$

$2\ \Phi$ 25 bottom and $2\ \Phi$ 16 top bars.

Shear is safe at the critical section as it lies outside the footing. Nominal V_{us}/d = 0.0348 × 30 = 1.044 KN/cm.

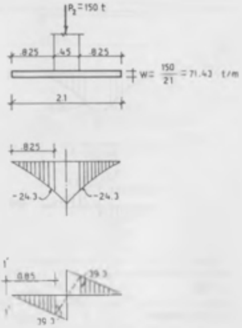
Minimum stirrups in footing beams ϕ 10/200 c/c thro.

$$\text{Side face steel} = .1 \times 30 \times \frac{90}{100} = 2.7\ \text{cm}^2,\ 4\ \phi\ 10\ \text{bars} = 3.14\ \text{cm}^2.$$

Fig. 11.10E gives the SFD and BMD of the beam FB2A. Max moment at bottom = 24.3 tm

k = .0692 × 24.3 = 1.68 N/mm².

A_s = 25.5 × p_t = 14.0 cm².



M	-24.3		-24.3	(tm)
K = .0692 M	1.68		1.68	(N/mm²)
As = 25.5 Pt	14.0		14.0	(cm²)
As		2 ϕ 16 3 ϕ 25	TOP BOTTOM	

Fig. 11.10E. Design of FB2A (.30 × .90).

3 $\bar{\phi}$ 25 bars at bottom, 2 $\bar{\phi}$ 16 top

Min, shear stirrups 10/ 200 C/c and side face steel 4 ϕ 10 bars.

d. *Concrete quantity*

$= 2 \times .3 \times .9 \times 2.1$	$= 1.34\ m^3$
$.6 \times 4.7 \times 2.1$	$= 5.922\ m^3$
$\dfrac{(6 + .3)}{2} \times 2.1(3 - .3)$	$= 2.5515\ m^3$
	$\underline{9.6075}\ m^3$

ii) *Alternative with slab type footing with hidden beams*

a) *Footing dimensions.* Fig.11.11A gives the plan (8.0 m × 2.1 m) of the combined footing with hidden cross beams FB3 and FB3A , of width $b = D + C$.

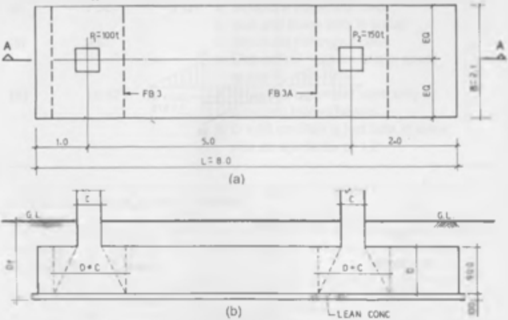


Fig. 11.11A. Plan and section of slab type combined footing. (a) Plan; (b) Section A-A.

b) *Slab design.* We have seen in alternative (ii) that $D = 60$ cm fails in shear. So Fig. 11.11B gives the analysis and design of slab with $D = 90$ cm. It needs $\bar{\phi}$ 16/120 top and bottom with distribution steel $\bar{\phi}$ 12/100 top and bottom.

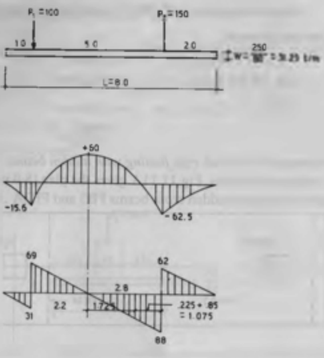
S = Shear force at the critical section = 31.25 × 1.725 = 53.9 t

$$S/B = 53.9/2.1 = 25.67\ t/m$$

$$T_v = 1.5 \times \frac{25.67}{85} = 0.45\ N/mm^2$$

With $p_t = .2$, min $T_c = .35 \times 1.2$ (age factor) = 0.42, nearly equal to 0.45 N/mm², so safe.

c) *Design of hidden beams*. Fig 11.10D gives SFD and BMD also for the hidden cross beams, FB3 with



M	-15.6 / +60	-62.6	(tm)
M/B	-7.4 / +28.6	-25.8	(tm/m)
K= 0208M	.15 / .59	.62	(N/mm²)
As =85Pt	10.6' (MIN) / 15.3	15.4	(cm²/m)
As	8 16 @120 % = 16.75	8 16 @120 % = 16.75 cm²/m	D=0.90

Fig. 11.11B. Analysis in the longitudinal direction.

$b = D + C = 0.9 + 0.45 = 1.35$ m, $D = 90$ cm, $d = 85$ cm, $M = 16.2$ tm.

$$k = \frac{1.5 \times 16.2 \times 105^5}{135 \times (85)^2 \times 10} = 0.25 \text{ N/mm}^2$$

$$A_{st} = .085 \times 135 \times \frac{85}{100} = 9.75 \text{ cm}^2$$

3 ϕ 25 bars at bottom gives = 14.73 cm^2

Nominal stirrups ϕ10/200 C/C.

Fig. 11.10E, gives SFD and BMD also for the hidden cross beams FB3A. For $M = 24.3$ tm, $k = .375$ N/mm^2, $p_t = 0.114$,

$A_{st} = 13.08$ cm^2 Same steel as for FB3 provided.

d. *Concrete quantity* = 8.0 × 2.1 × 0.9 = 15.12 m^3. Table 11.1 gives a comparison of these three alternatives for the given combined footing. From the above work, it is clear that alternative (i) with the central longitudinal beam shall be adopted as far as practicable. It is the most efficient and structurally correct solution.

Table 11.1. Comparison of three alternatives of a combind footing.

Altern-ative	Concrete quantity (m^3)	Remarks
(i)	7.02	a. Structural behaviour clear b. slab and beam safe in shear
(ii)	9.61	a. Structural behaviour clear. b. Slab with $D = 60$ cm fails in shear as per IS: 456-1978
(iii)	15.12	a. Structural behaviour clear only by providing hidden beams b. $D = 90$ cm Slab is just safe in shear with an age factor of 1.2

11.2.2 Example of a Strip Footing

Fig. 11.12A, gives the longitutional section of a strip footing with the following data:

$\Sigma P = 450$ t, $p_a = 15$ t/m^2

Area required $= \frac{450}{15} = 30.0$ m^2

C.G. of load from (1) $= \frac{1}{450} \times (100 \times 0 + 80 \times 6.0 + 120 \times 10.0 + 150 \times 16.0)$

$= 9.07$ m

$L = 2(9.07 + 0.93) = 20.0$

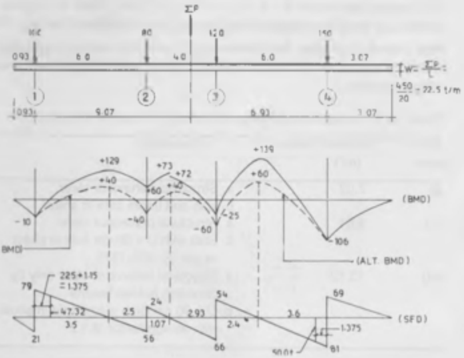
$B = \frac{30}{20} = 1.5$ m

Fig. 11.12A gives the SFD, BMD of the longitudinal beams FB1, shown in Fig. 11.12B. With $b = 60$ cm, $D = 120$ cm, $d = 115$ cm, $k = 0.0189$ M, $A_s = 69\ p_t$. The design of FB1 is given in Fig. 11.12A.

Shear at the critical section $= 2.225 \times 22.5 = 50.0$ t

$$T_v = \frac{1.5 \times 50 \times 1000}{60 \times 115 \times 10} \quad = 1.09 \text{ N/mm}^2$$

For $\quad p_t = 0.7, \qquad T_c = 0.53 \text{ N/mm}^2$



M	+129 -10	+60 +73 -40	+60 -60	-106	(t m)
K = .0189M	2.4 19	1.1 1.4 0.8	1.1 1.1	2.0	(N/mm²)
As = 69Pt	55.9 14.1(MIN) 6.2	23.2 30.5 16.4	23.2 23.2	47.3	(cm²)
As	2 7(56.29) 2(16.08) 2	3 4 2 2	2 3(24.12) 3 2	2 6 (48.25)	ALL ϕ 32 BARS
2 LEGS ϕ 10 STPS	150 % 150 %	150 %	150 %	150 %	

Fig. 11.12A. Analysis of strip footing in the longitudinal direction.

$V_{us}/d = (0.109 - .053) \times 60 = 3.36$ KN/cm

2 legged stirrups ϕ 10/150 $= 3.78$ KN/cm

Side face steel $= \dfrac{1}{100} \times 60 \times 120 = 7.2$ cm^2

6 $\bar{\phi}$ 12 bars 6.78 cm^2 nearly OK.

The details of calculations of at various locations are given as under (Fig. 11.12A).

$M_1 = -22.5 \times 0.93^2/2 = -10$ tm

$M_{1\text{-}2} = -22.5 \times 4.43^2/2 + 100 \times 3.5 = -221 + 350 = +129$ tm

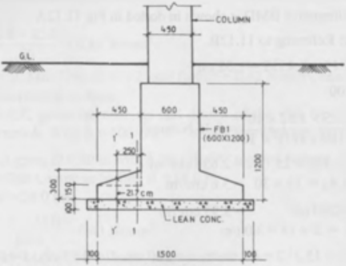


Fig. 11.12B. Cross-section of strip footing.

$M_2 = -22.5 \times 6.93^2/2 + 100 \times 6 = -540 + 600 = +60$ tm

$M_{2\text{-}3} = -22.5 \times \dfrac{8.0^2}{2} + 80 \times 1.07 + 100 \times 7.07 = -7.20 + 86 + 707 = 73$ tm

$M_4 = -22.5 \times \dfrac{3.07^2}{2} = -106$ tm

$M_{4\text{-}3} = -22.5 \times \dfrac{6.67^2}{2} + 150 \times 3.6 = -501 + 540 = +39$ tm

$M_3 = -22.5 \times 9.07^2/2 + 150 \times 6 = -925 + 900 = -25$ tm

$M_{3\text{-}2} = -22.5 \times 12^2/2 + 120 \times 2.93 + 150 \times 8.93$
$= -1620 + 352 + 1340 \quad = +72$ tm

The moments are calculated from left to near centre (where shear force is zero) and then again from right to the near centre of the point considered earlier. This way, we get reasonable values of moments. In certain offices, the practice is to find moments from left to the right end and again from right to the left end. This way, we get unreasonable results, in that, at free ends, the calcualted moments work out the highest in value. This is absurd, but still the practice continues!

Alternate BMD

$$M = \pm \frac{PL}{10}$$

$$(2): M = \pm \frac{80}{10} \times \frac{(6.0 + 4.0)}{2} = \pm 40 \text{ tm}$$

$$(3): M = \pm\frac{120}{10} \times 5 = \pm 60 \text{ tm}$$

The alternative BMD is shown in dotted in Fig 11.12A.

Slab design: Referring to 11.12B,

$$M = 15 \times 0.45^2/2 = 1.52 \text{ tm}$$

With $b = 100$

$$k = \frac{1.5 \times 1.52 \times 10^5}{100 \times (25)^2 \times 10} = 0.36 \text{ N/mm}^2$$

$A_{bt} = .105 \times 25 \quad = 2.626 \text{ cm}^2\text{/m}$
Min $A_{st} = .12 \times 30 \quad = 3.6 \text{ cm}^2\text{/m}$

Φ 10/200 c/c $\quad = 3.93 \text{ cm}^2\text{/m}$
$s_{1'-1'} = .2 \times 15 = 3.0$ t/m

$M_{1'-1'} = 15.2^2/2 = 0.3$ tm/m, $b = 100$ cm, $D = 21.7$ cm, $d = 16.7$ cm,
$\tan\beta = 15/45 = 0.3$

$$T_v = \frac{1.5 \times 3.0 \times 1000}{100 \times 16.7 \times 10} - \frac{1.5 \times 0.3 \times 10^5}{100 \times (16.7)^2 \times 10} \times 0.3$$

$$= 0.27 - 0.05 = 0.22 \text{ N/mm}^2.$$

$T_c = 0.35 \text{ N/mm}^2$ (minimum).

So, $D = 30$ cm $\rightarrow$ 15 cm, is safe in shear.

11.2.3. Example of a Slab Type Raft of Irregular Plan

In a certain basement, the area available was irregular in plan and a slab type raft was required so that excavated depth could be limited. Fig. 11.13A gives plan of the raft with column locations and loads. It gives,

$$\Sigma P = 1200 \text{ t}, p_a = 15 \text{ t/m}^2; \text{ Area required} = \frac{1200}{15} = 80 \text{ m}^2$$

$$\text{Provided area} = \frac{8+10}{2} \times 5 \times 2 = 90\text{m}^2.$$

$$p = \frac{1200}{90} = 13.33 \text{ tm}^2 < 15 \text{ tm}^2, \text{OK.}$$

C.G of loads is coincident with the C.G. of the area provided. Fig. 11.13B gives the SFD and BMD and design of the slab type raft in the lingitudinal direction. With $b = 100$ cm, $D = 90$ cm, $d = 85$ cm, $k = 0.0208$ M, $A_s = 85\ p_t$, the design gives for $D = 90$ cm, Φ 16/120 C/C top and bottom steel.

Max.shear at the critical section = 2.0 × 120 = 240 t

Width B available at the section = 10 – .4 = 9.6 m, shear B = 240/9.6 = 25.0 t/m.

$$T_v = \frac{1.5 \times 25.0}{85} = 0.44 \text{ N/mm}^2.$$

For p_t = .2, Min T_c = .35 × 1.2 (age factor) = 0.42 N/mm², nearly OK.

D = 90 cm is safe in shear.

Fig. 11.13C gives the analysis and design of the slab type raft in the transverse direction. With b = 100 cm, d = 85 – 1.6 = 83.4 cm, k = 0.0216 M, A_s = 83.4 p_t, it gives D = 90 m with ϕ 16/120 C/C top and bottom.

Shear = 240 t, same as before, B = 10.0 m.

Shear/B = 24.0 t/m.

$$T_v = \frac{1.5 \times 24.0}{83.4} = 0.43 \text{ N/mm}^2$$

T_c = .35 × 1.2 = 0.42 N/mm², nearly safe.

Slab type rafts are quite common, but generally, these are made rectangular in plan and the value of width (B) is then constant. But in this example the value of B varies and that makes this example interesting.

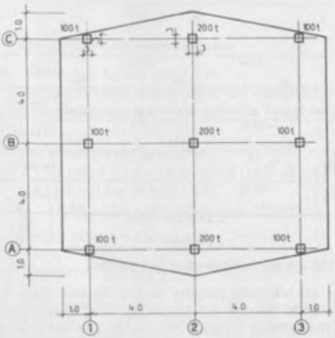


Fig. 11.13A. Plan of slab type raft of irregular plan.

11.2.4 Example of an Annular Raft

In a flour mill, an annular raft foundation was provided with the safe bearing capacity $p_a = 15$ t/m^2

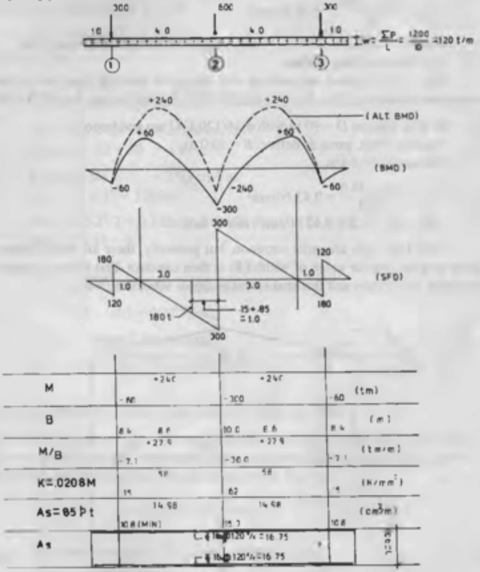


Fig. 11.13B. Design in longitudinal (↔) direction

The plot has adjoining property on two opposite sides. Fig.11. 14A gives the plan of the annular raft foundation with columns loads and Fig. 11 14A also gives sections 1-1 and 2-2 giving the slab and beam dimensions. All columns are 40 cm × 40 cm.

a. *Proportioning the foundation.* The load pattern is symmetrical about grid lines $\overline{C}$ and (8) so, cross wise, foundation is made symmetrical about $\overline{C}$. i.e.

5.85 + 1.6 = 7.45 m,
6.00 + 1.45 = 7.45 m.

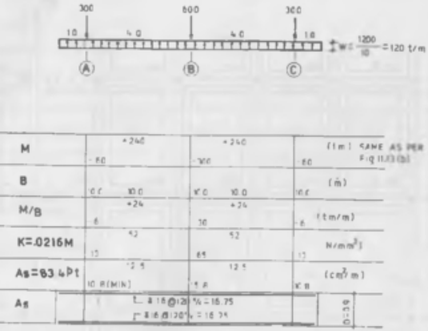


Fig.11.13C. Design in transverse direction

Let $b_1 = 0.8$ m, $b_2 = 1.2$ cm, $b_3 = 1.5$ m.

This is the master stroke assumption of this example. C.G. of loads and the C.G of the foundation area as a whole coincide. This is seen by inspection (Fig. 11.14A).

A = area of annular raft foundation

$= 2 \times 1.2 \times 18.4 + 1.5 \times 18.4 + 2 \times .8\,(14.9 - 2 \times 1.2 - 1.5) +$
$3 \times 1.5\,(14.9 - 2 \times 1.2 - 1.5)$

$= 44.2 + 27.6 + 17.6 + 49.5 = 138.9\ \text{m}^2.$

$\Sigma P = 72 \times 4 + 140 \times 6 + 170 \times 3 + 99 \times 2 = 1836$ t.

$$p = \text{soil pressure} = \frac{\Sigma P}{A} = \frac{1836}{138.9} = 13.2\ \text{tm}^2 < 15.0, \text{OK}.$$

b. *Design in the longitudinal direction* (↔). The entire raft is considered as one unit and the moments and the shears at any section are to be resisted by the number of beams available on lines A, C̄ and D, in proportion to their h-values.

Fig 11.14B gives SFD and BMD of the entire raft in the longitudinal direction. The alternate BMD is shown in dotted line. Table 11.2 gives the calculations for moments and shears at various sections in the scheme given by Reynolds.[70]

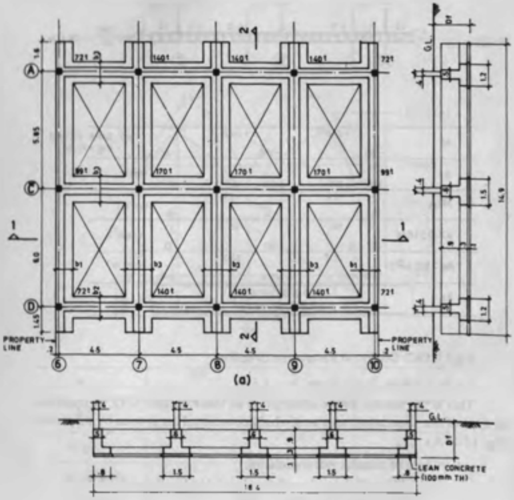


Fig. 11.14A. Plan and sections of an annular raft foundation. (a) Plan of annular raft (all columns 0.4 × 0.4); (b) Section 1-1; (c) Section 2-2.

Alternate BM at 7,8 $M = \pm \frac{450}{10} \times 4.5 = \pm 203$ tm.

The final moments and shears will be distributed to beams at A, $\bar{C}$, D in proportion to their beams widths, i.e. moment or shear αw or αpb or αb (where p is constant).

beam	base width (b)	$k = b/\Sigma b$
A	1.2 m	0.31
$\bar{C}$	1..5 m	0.38
D	1.2 m	0.31
Σb =	3.9 m	1.00

Table 11.2. Calculations of moments and shears in the longitudinal direction.

	Moment(t_m)			Shear (t)		
Section	Calculation	+ve	−ve	Calculation	+ve	−ve
6	$-100\times2^2/2$		−2	-100×0.2		−20
6-7	$-100\times2.45^2/2$	+547	−300			
	$+243\times2.25$	+247				
7	$-100\times4.7^2/2$		−1105	-100×4.7		−470
	$+243\times4.5$	+1090			+243	
			−15			−227
7-8	$-100\times6.95^2/2$		−2420			
	$+243\times6.75$	+1640				
	$+450\times2.25$	+1010				
		+230				
8	$-100\times9.2^2/2$		−4250	-100×9.2		−920
	$+243\times9.0$	+2190			+243	
	$+450\times4.5$	+2025			+450	
			−35			−227

Fig. 11.14C gives the SFD and BMD and design of beam $\bar{C}$ (60 cm × 120 cm). The same is given by Fig. 11.14D for beams A, D (50 cm × 120 cm). For beam $\bar{C}$, shear at critical section = (.95/2.3) × 89 = 37 t

$$T_v = \frac{1.5\times37\times1000}{60\times115\times10} = 0.8 \text{ N/mm}^2$$

For $p_t = \dfrac{32.17}{69} = .47$, $T_c = 0.44$ N/mm²

$$\frac{V_{us}}{d} = (.08 - .044)\times60 = 2.16 \text{ KN/cm}$$

ϕ 10/200 C/c STPS give = 2.836 KN/cm

$$\text{Side steel} = \frac{0.1}{100} \times 60 \times 120 = 7.2 \text{ cm}^2$$

$$6\ \bar{\phi}\ 12 \text{ bars} = 6.78 \text{ cm}^2$$

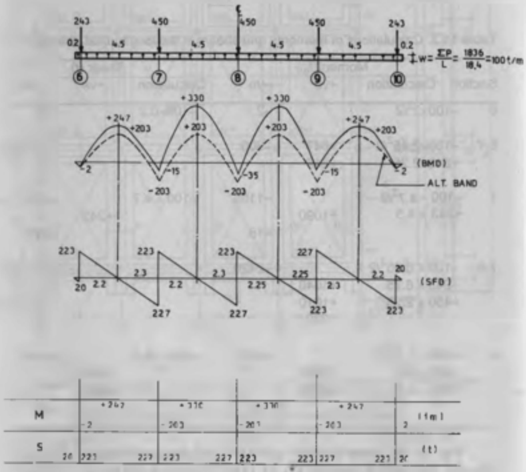


Fig 11.14B. Analysis of annular raft in the longitudinal direction.

For beams A, D also, $\bar{\phi}$ 10/200 steps with 6 ϕ12 bars for side steel are given.

Slab design: Referring to Fig. 11.14E,

$M = 15 \times .45^2/2 = 1.5$ tm, $b = 100$cm, $D = 30$ cm, $d = 25$ cm.

$$k = \frac{1.5 \times 1.5 \times 10^5}{100 \times (25)^2 \times 10} = .36 \text{ N/mm}^2$$

$$A_{st} = .10 \times 25 = 2.5 \text{ cm}^2/\text{m}$$

Min. $A_{st} = .12 \times 30 = 3.6$ cm^2/m, ϕ10/200 c/c = 3.93 cm^2/m.

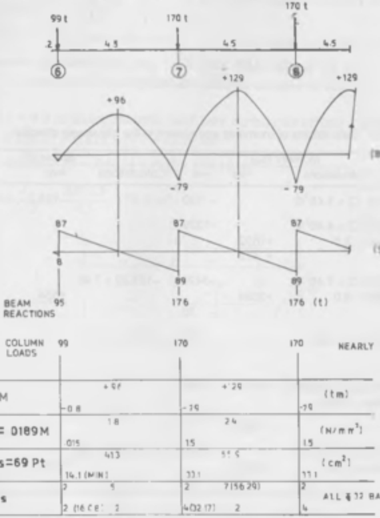


Fig. 11.14C. Design of beam C (600 × 1200).

Design in the transverse direction (↕). Fig 11.14F gives the SFD and BMD of the entire raft in the transverse direction with the alternate bending moment diagram shown in dotted lines. Table 11.3 gives the details of calculations of moments and shears.

$$\text{Alt. BM at } C = \pm \frac{708}{10} \times \frac{(6.0 + 5.85)}{2} = \pm 420 \text{ tm.}$$

Moments and shears are to be resisted by beams on lines,

$$7, 8, 9 : b_3 = 1.5 \text{ m } 3b_3 = 3 \times 1.5 = 4.5 \quad k = \frac{1.5}{6.1} = = .25$$

$$6, 10 : b_1 = 0.8\text{m} = 2b_1 = 2 \times .8 = 1.6 \quad k = \frac{.8}{6.1} = .13$$

$$\Sigma b = 6.1$$

Check $= 3 \times .25 = .75$

$2 \times .13 = .26$

$1.01 > 1.0$ OK.

Table 11.3. Calculations of moments and shears in the transverse direction.

	Moment (tm)			Shear (t)		
Section	Calculations	+ve	−ve	Calculations	+ve	+ve
D	$-123.22 \times 1.45^2/2$		− 130		-123.2×1.45	−179
D-C	$-123.22 \times 4.45^2/2$		−1220			
	$+564 \times 3.0$	+1692				
		+ 472				
C	$-123.22 \times 7.45^2/2$		−3420	-123.22×7.45		−918
	$+564 \times 6.0$	+3384			+564	
			− 36			−354
A	$123.22 \times 1.6^2/2$		− 158	123.22×1.6		−197
A-C	$-123.22 \times 4.525^2/2$	+1650	−1262			
	$+564 \times 2.925$					
		+ 388				
C	$-123.22 \times 7.45^2/2$		−3420	-123.22×7.45		−918
	$+564 \times 5.85$	+3300			+564	
			− 120			−354

The SFD, BMD and design of beams on lines 7,8,9 (60 cm × 120 cm) are given in Fig.11.14G and the same for beams on lines 6,10, (50cm × 120 cm.) are given in Fig. 11.14H. Shear stps. ϕ 10/200 with side steel 6 ϕ 12 bars are provided in all the beams. The bottom slab is D = 30 cm with ϕ10/200 both ways at the bottom provided.

11.2.5 Example of a Solid Octagonal Raft Foundation

An octagonal shaft supporting an overhead water tank was given a raft of octagonal shape in plan. The soil data assumed were:

$p = 13$ t/m² $\qquad D_f = 2.50$ m below G.L.

p_a (with earthquake) $= 13 \times 1.25 = 16.25$ t/m².

(i) *Tank Full Case*:

VL: $P = 541$ t $\quad M = 92.5$ tm.

Area required for direct load only $= 541/13 = 41.6$ m² $= 0.828\ L^2$, $L = 7.0$ m, where, L = diameter of the inscribed circle in an octagon of side a ($a = 0.414\ L$).

Let $L = 9.0$ m taking moment (and later earthquake effect) into account.

$A = 0.828\ L^2 = 0.828 \times (9)^2 = 67.07$ m²

$I = 0.05474\ L^4 = .05474 \times (9)^4 = 359.15$ m⁴

$$Z = \frac{I}{L/2} = \frac{359.15}{4.5} = 79.8 \text{ m}^3$$

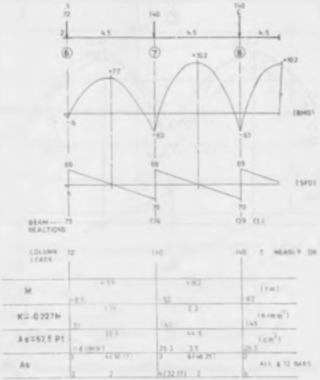


Fig. 11.14D. Design of beams A, D (500 × 1200).

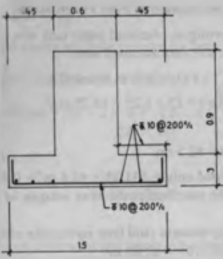


Fig. 11.14E. Design of footing slab.

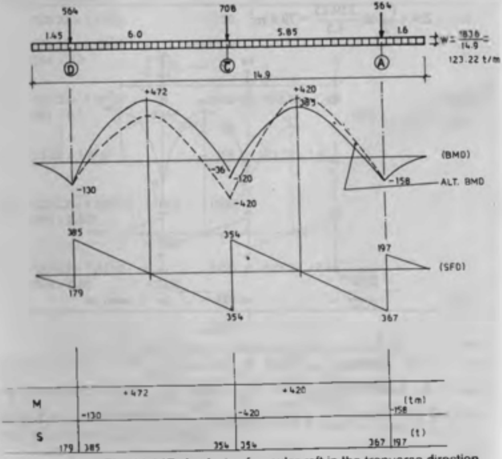


Fig. 11.14F. Analysis of annular raft in the tranverse direction.

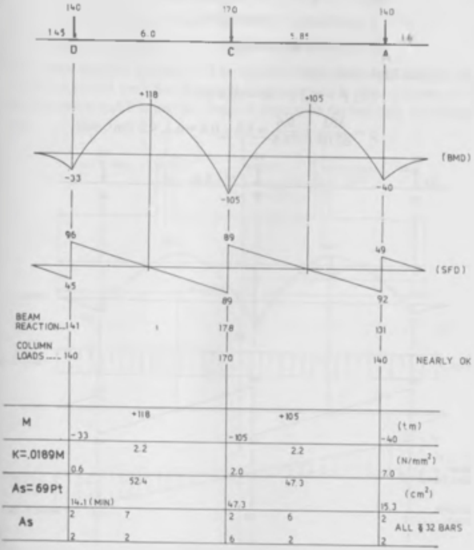


Fig. 11.14G. Design of beams on lines 7,8,9 (600 × 1200).

$$p = \frac{541}{67.07} \pm \frac{92.5}{79.8} = 8.1 \pm 1.2 = 9.3,\ 6.9\ \text{t/m}^2.$$

$p_{max} = 9.3\ \text{t/m}^2 < 13\text{t/m}^2$, OK.

$p_{min} = 6.9\ \text{t/m}^2 > 0$, OK.

VL+ EQ: $P = 541$ t $M = 612$ tm

$$p = \frac{541}{67.07} \pm \frac{612}{79.8} = 8.1 \pm 7.7\ \text{t/m}^2$$

$p_{max} = 15.8 \text{ t/m}^2 < 16.25 \text{ t/m}^2$, OK.

$p_{min} = 0.4 \text{ t/m}^2 > 0 \text{ t/m}^2$, OK.

$L = 9.0$ m is just adequate.

(ii) *Tank empty case*

VL: $P = 353$ t $\quad M = 60.4$ tm

$$p = \frac{353}{67.07} \pm \frac{6.4}{79.8} = 5.3 \pm 0.8 = 6.1,\ 4.5 \text{ t/m}^2\text{, safe}$$

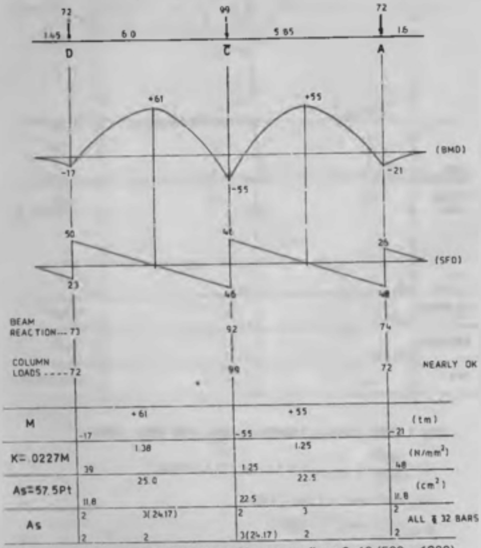


Fig. 11.14H. Design of beams on lines 6, 10 (500 × 1200).

VL + EQ: $P = 353$ t $\quad M = 429.4$ tm

$$p = \frac{353}{67.07} \pm \frac{429.4}{79.8}$$

$= 5.3 \pm 5.4 = 10.7, -0.1$ t/m²

$p_{max} = 10.7$ t/m² < 16.25 tm² OK

$p_{min} = -0.1$ t/m²

This negative pressure will be countered by 2.5m height of earth on top = 2.5 × 1.8 = 4.5 t/m². So, this small tension on soil is of no account. $L = 9.0$ m is adequate and it is just the required dimension for ensuring no tension on soil.

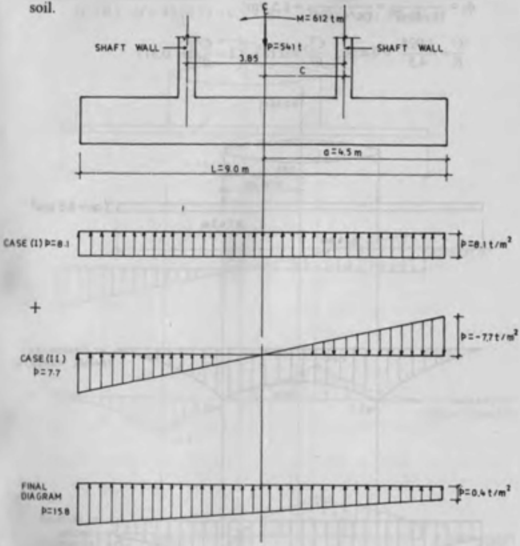


Fig. 11.15A. Soil pressure diagrams for an octagonal footing under VL + EQ.

(iii) *Analysis.* Tank full case with VL + EQ governs the design as it gives maximum value of soil pressure. Fig. 11.15A gives for the case VL + EQ, with $P = 541$ t and M 612 tm, the soil pressure diagrams on the footing. The octoganal plan is taken as circular plan for using methods of analysis based on circular plates.

Case (i) *Uniform soil pressure due to* $P = 541$ t. (Referrence Arya, Indian concrete Journal Dec. 1971)

Referring to Fig. 11.15B, we have,

$$q_0 = \frac{P}{(\pi/4)\,D^2} = \frac{541}{(\pi/4) \times 9^2} = 8.5 \text{ t/m}^2$$

$$\frac{C}{R} = \frac{1.925}{4.5} = 0.428\,,\ \frac{C^2}{R^2} = 0.183,\ \left(1 - \frac{C^2}{R^2}\right) = 0.817$$

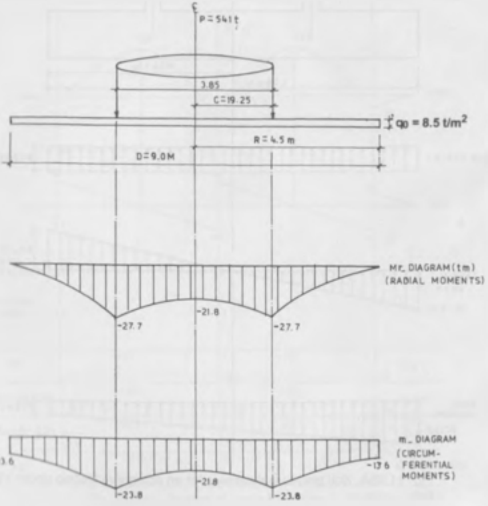


Fig. 11.15B. Analysis of circular slab under uniform pressure due to P = 541 t.

$$\ln\left(\frac{R}{C}\right) 0.849,\ q_0 \times \frac{R^2}{4} = 8.5 \times \frac{(4.5)^2}{4} = 43,\ \mu = 0$$

Radial Moments (M_r)

$$r \le c: M = \frac{q_0 R^2}{4}\left[\frac{(3+\mu)}{4}\left(1 - \frac{r^2}{R^2}\right) - \frac{(1-\mu)}{2}\left(1 - \frac{c^2}{R^2}\right)^2 - (1+\mu)\ln\left(\frac{R}{C}\right)\right]$$

At centre

$$(r = 0) : M = 43\,(0.75 - 0.5 \times .817 - .849) = -21.8 \text{ tm/m.}$$

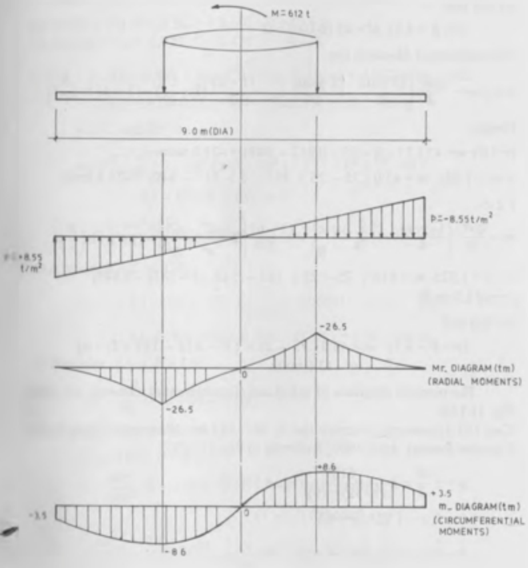


Fig. 11.15C. Analysis of circular slab under asymmetric pressure due to $M = 612$ tm.

Shaft point

$(r = c = 1.925)$: $M = 43.0\ [0.75 \times .817 - .5 \times 0.817 - 0.849] = -27.7$ tm/m

$$r \geq c.\ M = \frac{q_0 R^2}{4}\left[\frac{(3+\mu)}{4}(1 - \frac{r^2}{R^2}) - \frac{(1-\mu)}{2}(\frac{C^2}{r^2} - \frac{C^2}{R^2}) - (1+\mu)\ln(\frac{R}{r})\right]$$

Shaft point

$$(r = c = 1.925): M = 43\ [\ 0.75 \times 0.817 - .5 \times .817 - .849]$$
$$= -27.7 \text{ tm/m.}$$

At free end

$$(r = R = 4.5): M = 43\ (0\text{-}0\text{-}0) = 0$$

Circumferential Moments (m)

$$r \leq c: m = \frac{q_0 R^2}{4}\left[\frac{(3+\mu)}{4} - \frac{(1+3\mu)}{4}\frac{r^2}{R^2} - \frac{(1-\mu)}{2}(1 - \frac{C^2}{R^2}) - (1+\mu)\ln(\frac{R}{C})\right]$$

Centre

$(r = 0)$: $m = 43\ (.75 - 0 - 05 \times 0.817 - .849) = -21.8$ tm/m .

$r = c = 1.925$: $m = 43.0\ (.75 - .25 \times .183 - .5 \times .817 - .849) = -23.8$ tm/m

$r \geq c$:

$$m = \frac{q_0 R^2}{4}\left[\frac{(3+\mu)}{4} - \frac{(1+3\mu)}{4}\frac{r^2}{R^2} - \frac{(1-\mu)}{2}\left|2 - \frac{C^2}{r^2} - \frac{C^2}{R^2}\right| - (1+\mu)\ln\left(\frac{R}{r}\right)\right]$$

$r = c = 1.925$ $m = 43.0\ [\ .75 - .25 \times .183 - .5\ (2 - 1 - .183) - 0.849]$
$= -23.8$ tm/m

As free end

$$(r = R = 4.5):\ m = 43.0\ [.75 - .25 \times 1.0 - .5\ (2 - .183 \times 2) - 0]$$
$$= -13.6 \text{ tm/m}$$

The moment diagrams of radial and circumferential moments are given Fig. 11.15B.

Case (ii) *Asymmetric pressure due to* M = 612 tm. [Reference Arya, Indian Concrete Journal, April 1966] Referring to Fig. 11. 15C,

$$p = \pm\frac{M}{Z} = \pm\frac{612}{(\pi/32) \times (9)^3} = \pm 8.55 \text{t/m}^2.$$

$$b = 0,\ c = 1.925,\ a = 4.5$$

$$B = b/a = 0,\ C = c/a = .428, \quad R = \frac{r}{a} = \frac{r}{4.5}$$

$$\ln C = -0.8492,\ \frac{p\,a^2}{96} = 8.55 \times (4.5)^2/96 = 1.80$$

For $\theta = 0$, $\cos\theta = 1$

For $\theta = \pi$, $\cos\theta = -1$

Radial moments (M_r)

$R \le C$ with $B = 0$

$$M_r = \frac{p\,a^2}{96R^3}\,[-2R^2(5R^2 - 5R^4) + \frac{3}{C^2}(3 + C^2)\,(1 - C^2)\,R^4]\cos\theta$$

$$= \frac{p\,a^2}{96}\,[-10R + 10R^3 + \frac{3}{C^2}(3 + C^2)(1 - C^2)R]\,(\pm 1)$$

At centre, $r = 0$, $R = 0$: $M_r = 1.8\,(-0 + 0 + 0) = 0$

At support, $r = c = 1.925$, $C = 0.428$, $R = 0.428$

$$M_r = \pm 1.8[-10 \times 0.428 + 10 \times (.428)^3 + \frac{3}{.1832}(3 + .1832)\,(1 - .1832)$$
$$\times .428] = \pm 1.8(-4.28 + .784 + 18.2231) = \pm 26.5 \text{ tm/m}$$

$R \ge C$ with $B = 0$

$$M_r = \frac{p \times a^2}{9R^3}\,[0 - 2R^2(5R^2 - 5R^4) + 3(1 - R^4)(C^2 - 0) +$$
$$6(1 - R^2)(R^2 + 0)]\cos\theta$$

$$= \frac{p \times a^2}{96}\,[-10R + 10R^3 + \frac{3}{R^3}(1 - R^4)\,C^2 + \frac{6}{R}(1 - R^2)]\,(\pm 1)$$

At support, $R = C = .428$,

$$M_r = \pm 1.8\,[-4.28 + .784 + \frac{3}{(.428)^3}\,(.9664) \times .1832 + \frac{6}{.428}(1 - .1832)]$$

$$= \pm 1.8\,(-4.28 + .784 + 6.774 + 11.45) = \pm 26.5 \text{ tm/m.}$$

At free edge, $r = a$, $R = 1.0$,

$$M_r = \frac{pa^2}{96}\,(-10 + 10 + 0 + 0) = 0$$

Circumferential moments (m)

$R \le C$ (with $B = 0$),

$$m = \frac{\rho a^2}{96R^3}\,[0 + 2R^2(-\frac{5}{3}R^2 + R^4 + 0) + \frac{1}{C^2}(3 + C^2)$$
$$(1 - C^2)\,R^4]\cos\theta$$

$$= \frac{pa^2}{96}\left[-\frac{10R}{3} + 2R^3 + \frac{1}{C^2}(3 + C^2)(1 - C^2)\,R\right](\pm 1)$$

At centre $r = 0$, $R = 0$, $m = \pm 1.8\,(-0 + 0 + 0) = 0$

At support $r = C$: $\quad m = \pm 1.8\,[-\frac{10}{3} \times .428 + 2\ (.428)^3 + \frac{1}{0.1832} \times$

$$3.1832 \times .8168 \times .428](\pm 1)$$

$$= \pm 1.8(-1.427 + .157 + .6074) = \pm 8.6 \text{tm/m}$$

$(R = C = .428)$

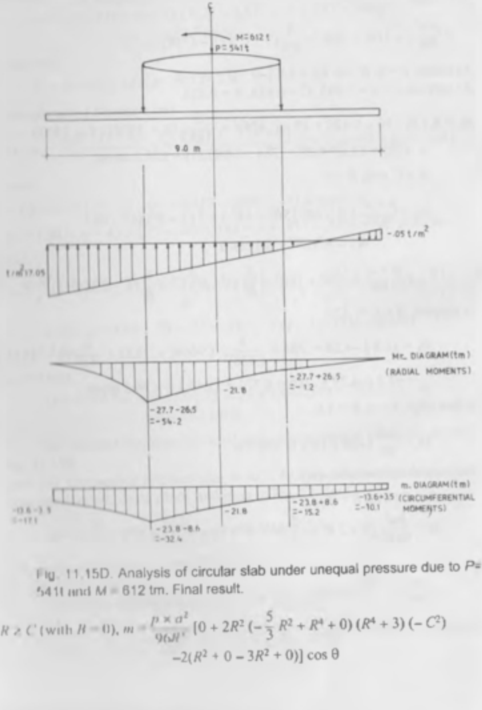


Fig. 11.15D. Analysis of circular slab under unequal pressure due to P = 541t and M = 612 tm. Final result.

$R \geq C$ (with $B = 0$), $m = \frac{p \times a^2}{96D}\,[0 + 2R^2\,(-\frac{5}{3}\,R^2 + R^4 + 0)\,(R^4 + 3)\,(-C^2)$

$$-2(R^2 + 0 - 3R^2 + 0)]\cos\theta$$

$$= \frac{p \times a^2}{96} \left[-\frac{10}{3}R + 2R^3 - \frac{C^2}{R^3}(R^4 + 3) - 2R + 6/R\right] (\pm 1)$$

At support $r = c$, $m = \pm 1.8[-1.427 + .157 - \frac{1}{.428}(3.0336) - .856 + 14.02]$

$(C = R = 0.428) = \pm 8.6$ tm/m

At free edge $r = a$, $R = 1.0$, $C = 0.428$,

$$m = \pm 1.8\left[-\frac{10}{3} + 2 - \frac{.1832}{1.0}(1 + 3) - 2 + 6\right] = \pm 3.5 \text{ tm/m.}$$

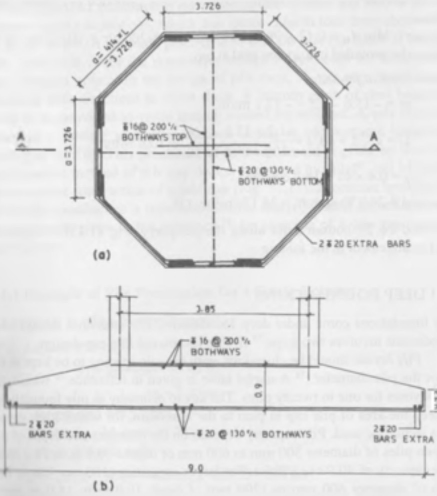


Fig. 11.15E. Detailing of octagonal footing. (a) Plan; (b) Section A-A.

Fig. 11.15C gives diagrams for the radial and circumferential moments for the case (ii).

The final result is summation of cases (i) and (ii) and the relevant moment diagrams are given in Fig 11.15D. It may be noted that the final moment diagrams will alternate when the direction of the applied $M = 612$ tm changes direction.

(iv) *Design*: $D = 90$ cm, $d = 85$ cm, M15 concrete, Fe 415 steel.

Bottom steel: Max $M_r = -27.7 - 26.5 = -54.2$ tm/m

$$K = \frac{1.2 \times 54.2 \times 10^5}{100 \times (85)^2 \times 10} = 0.9 \text{ N/mm}^2$$

$$A_s = .27 \times 85 = 22.95 \text{ cm}^2\text{/m}$$

ϕ 20/130 c/c = 24.17 cm²/m bothways to be provided in a square grid at bottom.

Top steel: Min $A_{st} = 0.12 \times 90 = 10.8$ cm²/m, 16/200 c/c = 10.05 cm²/m both ways to be provided in a square grid at top.

Bottom steel at free edge

$$m = -13.6 - 3.5 = -17.1 \text{ tm/m.}$$

$$D = 45,\ d = 40,\ k = \frac{1.2 \times 17.1 \times 10^5}{100 \times (40)^2 \times 10} = 1.28 \text{ N/mm}^2$$

$$A_{st} = 0.4 \times 40 = 16.0 \text{ cm}^2\text{ /m}$$

Provided ϕ 20/130 bottom = 24.17 cm²/m, OK

Provided 2ϕ 20 bottom extra along the periphery. Fig 11.15Ea, b gives the steel arrangement in the footing.

11.3 DEEP FOUNDATIONS

Pile foundations come under deep foundations. The structural design of pile foundations involves two steps:[73] (i) Pile layout; (ii) Pile cap design.

Pile layout should be close-knit, with the pile spacing to be kept at three times the pile diameter. [74] A useful table is given in reference,[73] which gives pile layouts for one to twenty piles. The key to economy in pile foundations is to keep the area of pile cap in plan to the minimum, for which high capacity piles should be used. Pile capacity depends on the diameter and depth of piles. Driven piles of diameter 300 mm to 600 mm of depths 10.0 m to 20.0 m may give capacity of 40.0 t to 100.0 t. For larger capacities (100 t to 300 t), bored piles of diameter 600 mm to 1200 mm of depth 10.0 m to 15.0 m may be used. Driven piles cause a lot of noise and vibrations during the driving process, while bored piles use auger-like equipment which is virtually vibration-free. Under-reamed piles are short-length piles of low capacities of 20.0 t to 40.0 t. These are distinguished by their tension-resisting capacities, for which these are mainly used for buildings on black cotton soil, which is an expansive clayey type of soil. The spacing of under-reamed piles is specified to be twice the bulb diameter, which is equivalent to five times the pile diameter. This leads to a large pile cap resulting in an expensive design. For this reason, the relevant code[75] allows the spacing of under-reamed piles to be 1½ times the bulb diameter which is equivalent to 3.75 times the piles to be 1½ times the pile capacity should be reduced by 10%. By this device, pile cap area works out to be less than before and it leads to economy.

Pile capacity is to be fixed by soil specialists in the soil investigation report of the site. It is to be later confirmed at the site by the initial and the routine tests as prescribed by the code.[75] Pile cap depth should be kept on the high side to effect economy in the consumption of steel and also to provide adequate rigidity to *pile cap*, which distributes column load from above to the pile group below. A common practice is to kept the pile cap depth at 0.6 m or more. The Code gives the criteria for design of pile caps for moment and shear. Shear is critical in the design of pile caps, so, we may provide shear stirrups in both directions to resist shear. A bottom mesh of steel bars is required to be provided to resist tension caused by moment. A pile cap is regarded as a footing with pile reactions acting as concentrated loads from below (Fig. 11.16). This procedure of pile cap design is given by the Code. An alternative method of pile cap design suggested by Hilal[69] and Mallick,[76] who use space truss action of a pile cap (Fig. 11.17). A concise book for design of pile foundations is recommended for study.[77] Useful numerical examples have been given in the reference.[73] An example of a pile cap design is given below.

11.3.1 Example of Pile Foundation for a Single Column

This example is taken from reference 73.

a) *Pile Layout*: Pile capacity = 70 t

Pile diameter = 0.45 m

$S = 3 \times .45 = 1.35$ m

Concrete mix M15, steel Fe415

Column size 45 cm × 67.5 cm.

Case (i) *Vertical loads only*

$P = 201$ t M45 = 4.8 tm, M67.5 = 6.4 tm

A 4 – pile group (2 × 2 grid) is proposed using Table 1 of reference 73.

$$\text{Pile load} = \frac{201}{4} \pm \frac{4.8}{2 \times 1.35} \pm \frac{6.4}{2 \times 1.35}$$

$= 50.3 \pm 1.8 \pm 2.4 = 54.5$ t (max)

Self-weight of pile cap per pile $= 2.1 \times 2.1 \times \frac{1.2 \times 2.5}{4} = 3.3$ t

From grade beams per pile (say) $= 5.5$ t

$\underline{63.3}$ t < 70.0 t, OK

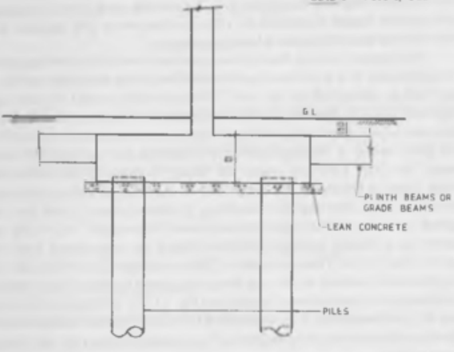


Fig. 11.16. Pile cap as footing.

Case (ii) VL + EQ(45)

$P = 201$ t $\quad M_{45} = 4.8 + 32.4 = 372$ tm, $\quad M_{67.5} = 6.4$ tm

$H_{45} = 12.4$ t.

Load per pile $= 50.3 \pm 13.8 \pm 2.4 = 66.5$ t (max)

Pile cap + grade beams $= 3.3 + 5.5 = 8.8$

$\underline{75.3}$ t $< 70 \times 1.25 = 87.5$ t, OK

Earthquake shear per pile $= \frac{12.4}{4} = 3.1 < \frac{5}{100} \times 70 \times 1.25 = 4.4$ t, OK

Case (iii) VL+ EQ(67.5)

$P = 201$ t $\quad M_{45} = 4.8$ t, $\quad M_{67.5} = 6.4 + 40.5 = 46.9$ tm

$H_{67.5} = 11.4$ t.

Load per pile $= 50.3 \pm 1.8 \pm 17.4 = 69.5$ t

Pile cap + grade beams $= 8.3 + 5.5 = 8.8$ t

$\overline{78.3}$ t < 87.5 t. OK

EQ shear per pile = $\frac{11.4}{4}$ = 2.9 t < 4.4 t, OK

Thus, a 4-pile group is adequate.

b. *Pile cap without a pedestal* (Method I). Fig. 11.18a gives a plan of the 4-pile foundation and Fig. 11.18b gives its section.

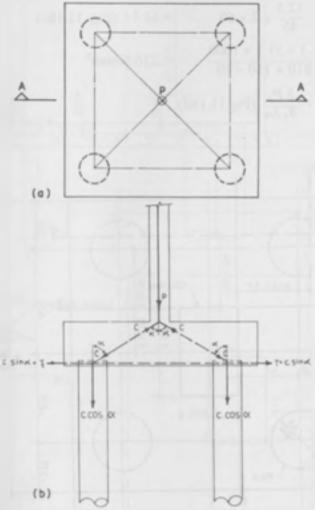


Fig 11.17. Space truss theory for pile cap design. (a) Plan of 4-piles group; (b) Section A-A.

Case (i) VL only: $P = 63.3 - 3.3 = 60.0$ t.

$M_{1-1} = 2 \times 60\ (0.675 - .225) = 54$ tm

With $b = 210$ cm, $D = 120$ cm, $d = 110$ cm

$$k = \frac{1.5 \times 54 \times 10^5}{210 \times (110)^2 \times 10} = 0.32 \text{ N/mm}^2$$

$A_{st} = .09 \times 110 \qquad = 9.9 \text{ cm}^2/\text{m}$

Min $A_{st} = .12 \times 120 \qquad = 14.4 \text{ cm}^2/\text{m}$

Provided by ϕ16/120 C/c $\quad = 16.75 \text{ cm}^2/\text{m}$ bottom both ways.

$$S_{1'-1'} = \frac{12.5}{45} \times 2 \times 60 \qquad = 33.3 \text{ t} \quad \text{(Fig. 11.18c).}$$

$$\tau_v = \frac{1.5 \times 33.3 \times 1000}{210 \times 110 \times 10} \qquad = .216 \text{ N/mm}^2$$

$$\delta = 1 + \frac{3\,P_u}{A_g f_{ck}} \quad \text{(Fig.11.18d)}$$

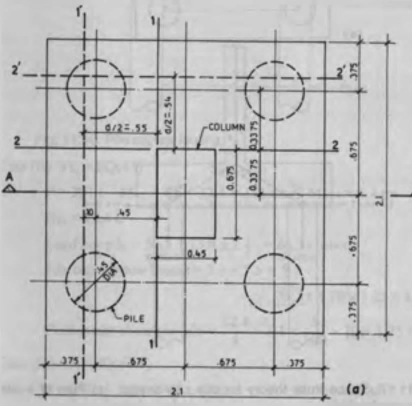


Fig.11.18. Contd.

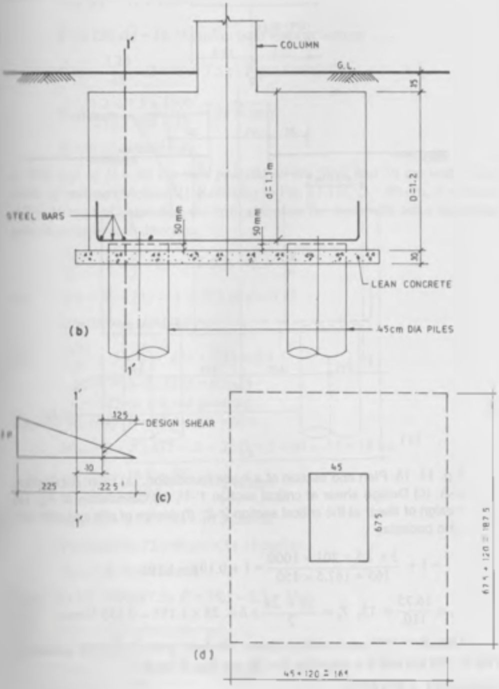


Fig.11.18 Contd.

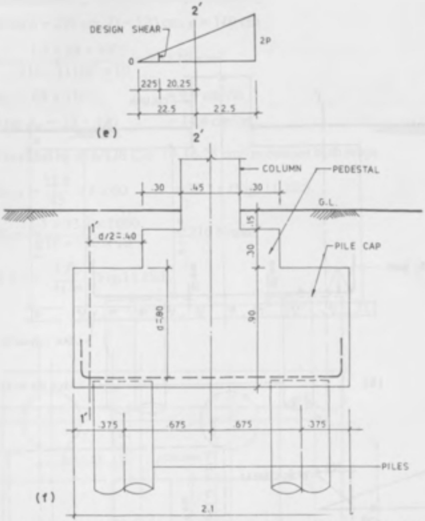


Fig. 11. 18. Plan and section of a 4-pile foundation. (a) Plan; (b) Section A-A; (c) Design shear at critical section 1'-1'; (d) Calculation of Ag; (e) Design of shear at the critical section 2'-2'; (f) design of pile cap with column pedestal.

$$= 1 + \frac{3 \times 1.5 \times 201 \times 1000}{165 \times 187.5 \times 150} = 1 + 0.195 = 1.195$$

$$p_t = \frac{16.75}{110} = .15, \ T_c = \frac{.32 + .24}{2} \times \delta = .28 \times 1.195 = 0.335 \text{ N/mm}^2$$

Thus $D = 120$ cm is safe in shear. We have done the above excercise with $D = 90$ cm and it is seen that $D = 90$ cm fails in shear.

Case (iii) VL + EQ(67.5)

$P = 78.3 - 3.3 = 75$ t. Referring to Fig. 11.18a,

$M_{2\text{-}2} = 2 \times 75\,(0.675 - .33\,75) = 50.625$ tm, $d = 110 - 2 = 108$ cm

$$k = \frac{1.2 \times 50.625 \times 10^5}{210 \times (108)^2 \times 10} = 0.248 \text{ N/mm}^2$$

Min $A_{st} = .12 \times 120 = 14.4 \text{ cm}^2/\text{m}$

ϕ 16/120 c/c = 16.75 cm^2/m both ways at bottom

$$S_{2'\text{-}2'} = \frac{2.25}{45} \times 2 \times 75 = 7.5 \text{ t (Fig. 11.18e)}$$

$$T_v = \frac{1.2 \times 7.5 \times 1000}{210 \times 108 \times 10} = .04 \text{ N.mm}^2$$

$T_c = 0.35 \text{ N/mm}^2$,safe.

c. *Pile cap of* $D = 90$ cm *with pedestal* 30 cm *thick and* 30 cm *wide on all sides of column* (Method II). Referring to Fig. 11.18f, $D = 90$ cm, $d = 80$ cm, $d/2 = 40$ cm and coinciding the critical section for shear with outer face of the pile, it gives in each direction,

$$\frac{45}{2} + .3 + \frac{d}{2} = 0.675 + .225 = 0.9$$

Or $\quad d/2 = .9 - .225 - .3 = .375$ or $d = 0.75$

$D = 0.90$ m, $d = 0.80$ m

Or $$\frac{.675}{2} + .3 + \frac{d}{2} = .675 + .225 = 0.9$$

$d/2 = \; = .9\text{–}3\text{–}.3375 = 0.2625$

$d = .525$ m, it is not govering.

Case (i) VL only: $P = 63.3 - 3.3 = 60$ t

$M_{1\text{-}1} = 2 \times P\,(.675 - .3 - .225) = 2 \times 60 \times .15 = 18$ tm

$$k = \frac{1.5 \times 18.0 \times 10^5}{210 \times (80)^2 \times 10} = 0.2 \text{ N/mm}^2$$

Min $A_{st} = 0.12 \times 90 = 10.8 \text{ cm}^2/\text{m}$

Provided by 12/100 c/c = 11.31 cm^2/m

$S_{1'\text{-}1'} = 0$. Shear is safe.

Case (iii) VL + EQ(67.5). $P = 78.3 - 3.3 = 75$ t

$$M_{1\text{-}1} = \frac{75}{60} \times 18 = 22.5 \text{ tm}$$

$$k = \frac{1.2 \times 22.5 \times 10^5}{210 \times (80)^2 \times 10} = 0.2 \text{ N/mm}^2$$

$D = 90$, ϕ12/100 C/c both ways at bottom provided as the minimum steel.

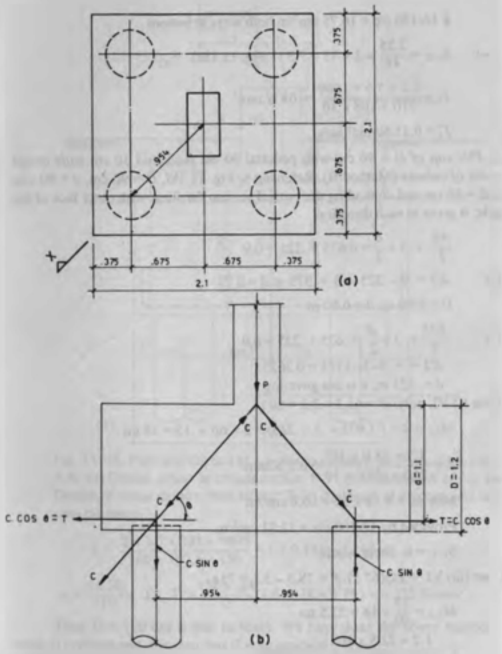


Fig. 11.19. Design of pile cap by truss analogy. (a) Plan of pile cap; (b) Section X-X.

d Pile cap design by the method of space truss (Method III). Referring to Fig. 11.19a, b and with $D = 1.2$ m, $d = 1.1$ m,

$$\tan\theta = \frac{1.1}{0.954} = 1.153,\ \theta = 49.066°,\ \cos\theta = 0.6552,\ \sin\theta = .7555,$$

$$4C\sin\theta = P$$

$$C = \frac{P}{4\sin\theta} = \frac{201}{4\times 7555} = 66.5\ \text{t}$$

$$T = C\cos\theta = 66.5 \times .6552 = 43.6\ \text{t}$$

$$2T = 2 \times 43.6 = 87.2\ \text{t}$$

$$A_{st} = \frac{1.5 \times 87.2 \times 1000}{0.87 \times 4150} = 36.23\ \text{cm}^2$$

$$A_{st}/B = \frac{36.25}{2.1} = 17.26\ \text{cm}^2/\text{m}$$

16/120 = 16.75, nearly OK.

We have given three methods of design of pile caps. Methods I and III lead to nearly the same result with $D = 1.2$ m and with ϕ16/120 both ways at bottom, while Method II gives $D = 0.9$ with ϕ12/100 c/c bothways at bottom, with a pedestal of 30 cm thickness all around the column. Method II leads to economy.

CHAPTER 12

Design of Reinforced Contrete Elements

12.1 INTRODUCTION

Design of reinforced concrete elements like slabs, beams, columns, footings, retaining walls, etc. is based on the limit state method (section 5 of IS: 456-1978[6]). Previously, working stress method (Section 6 of the Code) was used in practice but it has now become obsolete. In the present approach, all critical sections of a reinforced concrete member are designed for the forces acting at these sections, these forces being given by the structural analysis of members. In a multistorey building, there are many members and in each member, there may be more than one critical sections to be checked, so it follows that the design of sections is a time-consuming and repetitive process, which can only be done by using design aids – tables and charts – which are based on the prevalent code, i.e. IS:456:1978.[6] Useful design aids are available in the following sources:

i. Manual by Varyani and Radhaji.[17]
ii. SP: 16 Design Aids.[58]
iii. Handbook by Ghanekar and Jain.[78]
iv. Handbook by Iyengar and Viswanatha.[79]
v. Handbook by Karve.[80]
vi. Handbook by Sinha (under print).[81]

For achieving economy in design, structural analysis should be carefully and accurately done, so that the forces on the critical sections are realistic. Then design of critical sections is carefully done to get reasonable concrete sizes and optimum steel consumption in members. This step may result in a small economy of 3% to 5% of cost of building. More substantial economy in structural design can be achieved by choosing a suitable structural system (chapter 2) for a given building.

Salient feactures for design of reinforced concrete members are discussed below separately for important elements like slabs, beams, columns, footings, plinth beams etc. Retaining walls and water tanks have been discussed in the Manual.[17] Shear walls and shear cores are designed in chapter 8, while stairs are given in chapter 4.

12.2 SLAB DESIGN

Slab panels are to be designed for the limit states of bending moment and deflection. The thickness of slab is governed by deflection, while the steel areas at mid-span and support sections depend on the bending moments. For an overall economy in the structural design of building, it is necessary to keep slab thickness as small as practicable. Thick slabs add to the dead load of building, by which earthquake forces also get increased. Further, loads on beams, columns and footings all get increased, on account of increase in slab thickness. So, overall economy demands a small slab thickness. As *l/d*-ratios for control of deflection depend on the tension steel area, which in turn, depends on moment and effective depth of slab, slab design in accordance with the Code, is an iterative process. Direct slab design for single and two-way slab panels is given by N. Pandian and S. Rajkumari,[82] in their discussion of a paper of the author.[83] Further, Pandian[84] has also given a direct method of slab design, on similar principles, independently. In a discussion of reference 84, the author has also given charts[85] (Fig. 12.1) for direct design of slab panels using concrete mix M15 and steel type Fe 415.

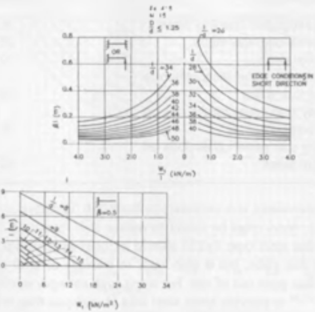


Fig. 12.1. Minimum effective depth for solid slabs for moment and deflection controls, in accordance with IS:456-1978. Here $W = W_1 + W_2$; W_2 = self wt. of slab; W_1 = slab loading less self wt. of slab in kN/m^2; l = span in short direction in m; for two-way slabs, replace l with l_x; β = coefficient of moment ($M = \beta W l^2$) at the midspan section in the short direction (exception: for cantilever slabs $\beta = 0.5$); slab panels are supposed to support brick partitions.

12.2.1 New Ideas Within or Beyond the Code

i) The Code has stated, in its, clause 23.1, that 'deflection provisions (clause 22.2) for beams apply to slabs also'. Now, this is true for single-way slabs but it is definitely not valid for two-way slabs. This aspect needlessly gives more thickness for two-way slabs.

ii) The deflection provisions of the code are mainly required for prevention of cracking in the brittle partition walls and finishes resting directly on slabs and beams. Normally, walls are made to rest on beams, which have adequate depth for deflection. Slab panels not supporting brick walls, like roof and stairs slabs, etc. may be made as thin as given by the old code IS: 456-1964.[3] Table 12.1 gives the slab thickness as per the old code. It is seen that in this way,[86] the first allowable deflection limit of $l/250$ is easily satisfied. So, it is recommended that all roof slab panels, stairs slabs and floor slab panels not supporting brickwalls should be given thickness as per the old code[3] (Table 12.1). This satisfies the spirit of the code and it results in economy.

Table 12.1. Span-to-depth ratios for control of deflection as per old code.

S.No	Slab panel	l/D or l_x/D
1.	Simply supported one way slabs	30
2.	Continuous one-way slabs	35
3.	Cantilever slabs	12
4.	Two-way slab panel discontinuous on all four sides	35
5	Two-way slab panel continuous on two adjacent sides	40
6.	Two-way slab panel continuous on all four sides	40

iii) For economy, it is necessary to have slab thickness, as small as practicable. Hence, ways must be found to reduce slab thickness. It is suggested elsewhere,[83] that steel type Fe250 should be used as reinforcement in slabs. This leads to thin slabs, but it also leads to wastage of steel and on this account, Fe250 has gone out of use. For using high strength steels like Fe415, it is suggested[7,87,88] to provide more steel area at midspan than what is required for moment, so that, steel stress will then work out less than 0.58 f_y (= $0.87f_y/1.5$). By this artifice, modification factor given by Fig. 3 of the Code gets increased and then a reduced slab thickness will be adequate for deflection requirement. In this way, individual slab panels workout costly, but reduced slab thickness will lead to an overall economy of building. Fig. 12.2 gives an amplification of Fig. 3 of the Code and it is taken from Ghanekar and Jain.[78]

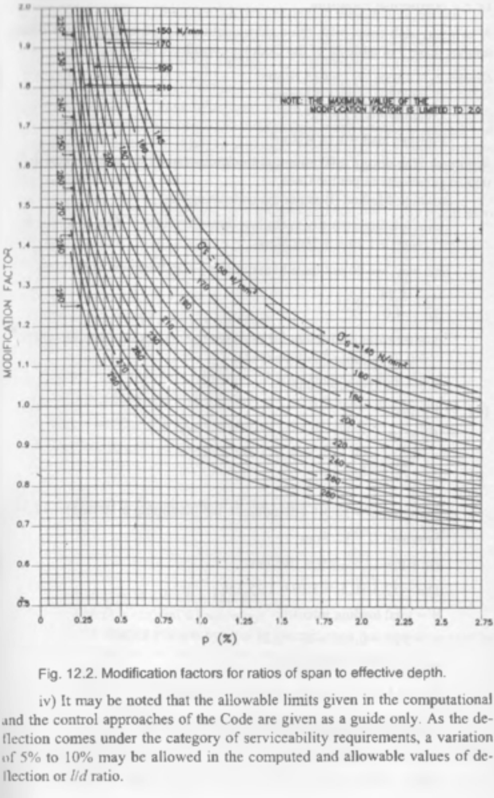


Fig. 12.2. Modification factors for ratios of span to effective depth.

iv) It may be noted that the allowable limits given in the computational and the control approaches of the Code are given as a guide only. As the deflection comes under the category of serviceability requirements, a variation of 5% to 10% may be allowed in the computed and allowable values of deflection or *l/d* ratio.

12.2.2 Numerical Example

A two-way slab panel with three sides continuous and one long side discontinuous (case 3 of table 22 of the Code) is considered here.

Given: $l_x = 4.0$ m $f_y = 415$ N/mm^2
$l_y = 6.0$ m $f_{ck} = 1.5$ N/mm^2
Liveload = 4.0 KN/m^2

Finishes load = 1.24 KN/m^2

Required: Slab design off minimum thickness, satisfying moment and deflection requirements as per the Code.

Solution: Method (i) as per the code using Fig. 12. 1

w_1 = loading intensity excluding the self weight of slab
$= 40 + 1.25 = 5.25$ KN/m^2

This way, we do not have to assume any slab thickness at the start.

$$\frac{w_1}{l_x} = \frac{5.25}{4.0} = 1.31$$

β = moment coefficient for midspan moment in the short direction
$= 0.051$ (Table 22 of the Code)

$\beta l_x = 0.051 \times 4.0 = 0.204$

Fig. 12.1 gives,

$l/d = 39$

$$d = \frac{400}{39} = 10.3 \text{ cm}$$

Required $D = 10.3 + 1.5$ (cover) $+ 0.5$ (assuming ϕ 10 bars) $= 12.3$ cm

Provided $D = 12.5$ cm

Now $d = 12.5 - 2.0 = 10.5$ cm

$D/d = 12.5/10.5 = 1.19 < 1.25$, OK

W = total loading intensity

$= 5.25 + 0.125 \times 25 = 5.25 + 3.125 = 8.375$ kN/m^2

$Wl_x^2 = 8.375 \times (4.0)^2 = 134$

$D = 12.5$ cm, $d_x = 12.5 - (1.5 + .4) = 10.6$ cm

(assuming ϕ 8 bars), $d_y = 10.6 - 0.8 = 9.8$ cm

Table 22 of the code gives:

$-M_x = -.067 \times 134 = -9.0$ KNm, $-k_x = 1.2$, $-As_{tx} = 3.93$ cm^2/m

$+M_x = +.051 \times 134 = +6.8$ KNm, $+k_x = 0.9$, $+As_{tx} = 2.26$ cm²/m

$-M_y = -.037 \times 134 = -5.0$ KNm, $-k_y = .78$, $-As_{ty} = 2.32$ cm²/m

$+M_y = +.028 \times 134 = +3.8$ KNm. $+k_y = 0.59$, $+As_{ty} = 1.72$ cm²/m

This is a direct design, without any iteration. The Code can be directly used also but then we have to assume a slab thickness at the first instance. This will be an iterative process and it takes three to five trials to reach a convergence, involving a lot of labour and time. Use of Fig. 12.1 saves time and effort of a designer. This method should be used for all slab panels, supporting brick partition walls.

Method (ii) *As per the Code*[6] *and its Explanatory Handbook.*[7] By using more steel at the midspan in the short direction we may get a thinner slab.

Let, $d = 11.0$ cm (less than 12.5 cm found before).

$d_x = 11.0 - 1.9 = 9.1$ cm

$d_y = 9.1 - 0.8 = 8.3$ cm

$W = 5.25 + 0.11 \times 25 = 8.0$ KN/m²

$Wl_x^2 = 8.0 \times (4.0)^2 = 128$

$-M_x = -.067 \times 128 = -8.6$ KNm $-K_x = 1.56$, $-As_{tx} = 4.58$ cm²/m

$+M_x = +.051 \times 128 = +6.5$ KNm $+K_x = 1.18$, $+As_{tx} = 3.31$ cm²/m

$-M_y = -.037 \times 128 = -4.7$ KNm $-K_y = 1.02$, $-As_{ty} = 2.56$ cm²/m

$+M_y = +.028 \times 128 = +3.8$ KNm $+K_y = 0.76$, $+As_{ty} = 1.97$ cm²/m

Check on deflection. By the method of the Code, $p_t = 3.31/9.1 = .364\%$

$$l/d = 26 \times 1.36 = 35.36$$

$$d = \frac{400}{35.36} = 11.3 \text{ cm}$$

Required $D = 11.3 + 1.9 = 13.2$ cm

Provided $D = 11.0$ cm is inadequate.

Now, we may provide more steel at the mid-span, say the same steel as required at the support section, this being easy in detailing also.

$$p_t = \frac{4.58}{9.1} = 0.503\%$$

$$f_s = 0.58 f_y \times \frac{364}{503} = 0.58 \times 415 \times 0.72 = 174 \text{ N/mm}^2$$

Using Fig. 12.2, we get,

$l/d = 26 \times 1.66 = 43.16$

$$d = \frac{400}{43.16} = 9.3 \text{ cm}$$

Required $D = 9.3 + 1.9 = 11.2$ cm

Provided $D = 11.0$ cm, nearly OK (1.8% difference is small).

It is recommended to put more steel at the midspan section in the longer direction also, for the sake of compatibility of deflections in both the directions. For ease in detailing, support steel is provided at the midspan also in either direction. Slab panels, detailed in this way, will be adequate for deflection.

Method (iii) *As per the old code.*[3] If it is known that no brick wall is to rest on this slab panel (the load considered has no wall load component), we may use slab thickness as per the old code[3] (Table 12.1).

Table 12.1 requires, $l_x/D = 40$

$$D = \frac{400}{40} = 10 \text{ cm}$$

$$W = 5.25 + 0.1 \times 25 = 7.75 \text{ KN/m}^2$$

$$Wl_x^2 = 7.75 \times (4.0)^2 = 124$$

$$D = 10 \text{ cm}, d_x = 10 - 1.9 = 8.1 \text{ cm}$$

$$d_y = 8.1 - 0.8 = 7.3 \text{ cm}$$

$$-M_x = -.067 \times 124 = -8.3 \text{ KNm}, -K_x = 1.9, -As_{tx} = 5.29 \text{ cm}^2/\text{m}$$

$$+M_x = +.051 \times 124 = +6.3 \text{ KNm}, +K_x = 1.4, +As_{tx} = 3.58 \text{ cm}^2/\text{m}$$

$$-M_y = -.037 \times 124 = -4.6 \text{ KNm}, -K_y = 1.3, -As_{ty} = 2.96 \text{ cm}^2/\text{m}$$

$$+M_y = +.028 \times 124 = +3.5 \text{KNm}, +K_y = 1.0, +As_{ty} = 2.21 \text{ cm}^2/\text{m}$$

The first limit of the Code of $l/250$ will be satisfied by following the procedure given in reference 86. This approach satisfies the spirit of the Code and it leads to thin slab panels.

The above three methods, all based on the Code, give slab thicknesses of 12.5 cm, 11.0 cm and 10.0 cm, the thinnest slab will lead to an over-all economy in building design. It may be noted that all these methods of slab design for deflection are based on the Code, either directly or indirectly.

12.3 BEAM DESIGN

Limit states of moment, shear and deflection govern the design of beams. Concrete mix M-15 (1:2:4) is generally used in beams, as richer mixes do not result in saving of tension steel, these are, of course helpful in reducing compression steel only at the doubly reinforced sections. Over all, it is seen that M-15 concrete results in economy in beam design. This aspect applies to slab design also, which has singly reinforced sections at the critical sections. Shear stirrups will be provided to take care of the excess shear, beyond the shear capacity of the concrete section of beams. For saving steel quantity beam depth is generally taken at 1/10th to 1/12th (1/15th minimum) of span (1/5th to 1/7th for cantilever beams). These beam depths, generally, work out to be more than adequate for deflection. In beam design, we do not go in for less depth as, then steel consumption will increase sharply, leading to ineconomy. Further, the dead load of beams is only a small part of the total dead load of building. Width of a beam is helpful in shear and also in bar placement in one layer. It is kept small in the vicinity of 230 mm to 300 mm. It is desirable to have beam depth as much as allowed by the architect.

Critical section for moment is the column face, while the same for shear is the section effective depth (d) away from the column face in beam design. Critical section for checking deflection is the midspan section (or free edge of a cantilever beam) of beams. Beams are to be designed for the effects of vertical loads with a load factor of 1.5 and also for the combined effects of vertical and horizontal loads with a load factor of 1.2. In other words, the combined effects multiplied by 0.8 (= 1.2/1.5) or the vertical load effects whichever greater, will be considered for design with a load factor of 1.5.

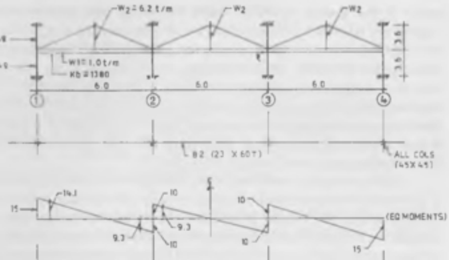
For moment design, K-value method of SP-16[58] is convenient to use. For shear stirrups and deflection check also, SP-16 design aids are easy to use. In SP-16, the maximum value of K is given as 6.0 while K-values upto 13.0 are given by Ghanekar and Jain.[78] Manual[17] also will be helpful, when K-values work out high. Some engineers get baffled when $K > 6.0$, the maximum value given in SP-16[58] and they think of choosing bigger size for the beam. It is not at all necessary to change the size of beam on this account only. Other references as mentioned above will also come to their aid in such cases.

Bent up bars in beams as shear reinforcement are getting obsolete due to the more labour involved. Two or four legged shear stirrups are adequate as shear reinforcement. However, when shear is accompanied by torsion , only two legged stirrups will have to be used, the four legged stirrups are not fully effective for torsional effects. In such cases two legged double stirrups (two stirrups placed side by side are called double stirrups) should be used.

12.3.1 Numerical Example

This example is taken from Chapter 6. The beam B2 is to be designed for VL and EQ loadings. Effect of T-action on the moment of inertia of beam is considered and face correction is also applied. This is the strict design of beam, taking all the fine points into account. The stepwise design of B2 (23 × 60) is given in Fig. 12.3. The additional details are given below:

$$\text{F.E.M.} = 1.0 \times \frac{6.0^2}{12} + \frac{5}{96} \times 6.2\,(6.0)^2 = 3.0 + 11.6 = 14.6 \text{ tm}$$



D.F.	.42		.3	.3		.3	.3		0.42	
fem	-14.6		+14.6	-14.6		+14.6	-14.6		+14.6	
DIST	+6.1		0	0		0	0		-6.1	
C.O.	0		+3.0	0		0	-3.0		0	
DIST.			-.9	-.9		+.9	+.9			
C.O.	-.4	+23.1	0	+.4	+23.1	-.4	0	+23.1	+.4	
DIST	+.2	-12.6	-.1	-.1	-15.2	+.1	+.1	-12.6	-.2	
FINAL MTS. (VL)	-8.7	+10.5	+16.6	-15.2	+7.9	+15.2	-16.6	+10.5	+8.7	
FACE CORRECTION	1.8			1.8			1.8		1.8	
Va/3 , Va/6		.9			.9			.9		
DESIGN MTS (VL)	-6.9			-14.8			-14.8		-6.9	
		+9.6			+7.0					
EQ MOMENTS AT FACE	±14.1			±9.3			±9.3	+9.6	±14.1	
DESIGN MTS	-16.8			-19.3			-19.3		-16.8	
=.8(VL+EQ)	+5.8	+9.6			+7.0			+9.6	+5.8	
K=.208M	3.5			4.0			4.0		3.5	
	1.2	2.0			1.5			2.0	1.2	
As=12.88Pt	14.9			16.9			16.9		14.9	
	5.7	8.8		7.7	6.2		7.7	8.8	5.7	
As	5	2		6	2		6	2	5	ALL ɸ 20 BARS
	2	3		3	2		3	3	2	
SHEAR (V)	12.3		12.3	12.3		12.3	12.3		12.3	
MT. EFFECT	1.3		1.3	–		–	1.3		1.3	
FINAL SHEAR (VL)	11.0		13.6	12.3		12.3	13.6		11.0	
EQ. SHEAR	4.2		4.2	3.3		3.3	4.2		4.2	
DESIGN SHEAR = .8(VL+EQ)	12.2		14.2	12.5		12.5	14.2		12.2	
STPS.	ɸ 8 @ 150 %			ɸ 8 @ 150 %			ɸ 8 @ 150 %			2 LEGS

Fig. 12.3. Design of beam B2 (230 × 600).

$$\text{Simple } M_t = 1.0 \times \frac{6.0^2}{8} + \frac{1}{12} \times 6.20\,(6.0)^2 = 4.5 + 18.6 = 23.1 \text{ tm}$$

$$T_v = .116 \times 14.2 = 1.65 \text{ N/mm}^2$$

For $p_t = 1.46$, $T_c = 0.68$ N/mm^2

$$\frac{V_{us}}{d} = (.165 - .068) \times 23 = 2.23 \text{ KN/cm}$$

Provided by ϕ 8/150 stirrups of capacity = 2.42 KN/cm.

Check on deflection. The mid-span section of the end span 1-2 is the critical section for deflection check.

$$A_{st} = 3\ \Phi\ 20 \text{ bars} = 9.43 \text{ cm}^2$$

$$p_t = \frac{9.43 \times 100}{23 \times 56} = 0.73, \text{ concrete M15,}$$

Design Aids[58] (chart 22) give

$l/d = 21$

$$d = \frac{600}{21} = 28.6 \text{ cm}$$

$D = 28.6 + 4.0 = 32.6$ cm, required for adequacy in deflection.

Provided $D = 60$ cm is more than adequate for deflection.

12.4 COLUMN DESIGN

Limit states of axial compression and bending moments about two axes, in general, govern the design of columns. Shear in columns is small and shear stress works out to be safe. Stirrups in columns are provided mainly for holding column bars in place and making them strong against buckling as these bars come under direct compression. Moments in columns change sign in each storey, so that, we generally provide symmetrical bar arrangement in a column section and the steel area is kept constant throughout a given storey. Columns, therefore, consume a good quantity of steel. Further, minimum eccentricity moments and additional moments due to slenderness, if any, are also included in the column design.

For preparing architectural working drawings, fairly accurate column sizes are to be given by the Structural Engineer at the start of design itself. Once these column sizes are given, architect prepares working drawings, showing window divisions etc. and then it will be difficult to change column sizes later on. So, it is important to be reasonably precise on the estimation of column sizes.

12.4.1 Preliminary Column Sizes[89]

Sizing of vertical elements like columns and shear walls has significane both on the utilization of building area and on the economy of building cost. Archıtectes in their attempts to derive maximum functional advantage of a given area, prefer to have as small columns as possible. Further, many architects prefer thin columns for attractive elevations of buildings. Economy in building cost needs large-sized columns with the minimum steel reinforcement. Columns consume a large quantity of steel, as there is , in general, no variation in column steel in a given storey. With the present cost of concrete at Rs 1200/- per m^3 and cost of steel at Rs.12,000/- per tonne, it is prudent to save steel and consume more concrete instead. By using rich concrete mixes and taking advantage of age factors in the lower storeys of buildings, size of columns can, to some extent, be reduced. But thin columns have to be designed for additional slenderness moments, involving more steel consumption, which can be easily avoided by adopting large-sized columns. So the requirements in respect of column sizes are contradictory from the view points of architecture and structure.

The same considerations hold good for sizing of shear walls, except that richer concrete mixes are not required and slenderness effects can be avoided by adopting stiff, geometrical shapes of shear walls in plan, which often enclose lifts, stairs, toilets, stores etc. For efficient utilization of building area, the optimum carpet area is about 80% of the avilable floor area, leaving about 20% of the area to be enclosed by shear walls or shear cores.

The preliminary column size can be fixed for axial load with steel reinforcement being restricted to, say, 1.0% which may be regarded as the minimum. By including moments due to the minimum eccentricity provisions of the code, steel percentage is expected to increase to a value, say, 2.0%. Further, the effect of horizontal loads will tend to increase the steel percentage to a maximum value of 4.0%, which is accepted as reasonable in practice. This is the basis of starting with 1.0% steel for axial load only. This approach is a via media between the conflicting requirements of architecture and structure.

With the total loading intensity w KN/m^2 and the tributary area A (m^2) of a column of N storeys, the axial load on the column is given by

$$P = w.A.N \tag{12.1}$$

It is seen in practice that w varies from 15 KN/m^2 to 20 KN/m^2 for normal buildings. The factored column load (P_u), in accordance with the Code, is given by,

$$P_u = 1.5w.A.N \tag{12.2}$$

The axial load capacity of the column section is given by the Code,

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_s \tag{12. 3}$$

which is easily transformed into,

$$P_u = f_{ck}A_g\,[0.4 \;+ \frac{r}{f_{ck}}\,(0.67\,f_y - 0.40\,f_{ck})] \tag{12.4}$$

where, A_c = net concrete area of column section

A_g = gross concrete area of column section

A_s = area of steel in column section

$$r = \frac{A_s}{A_g}$$

Equations (12.2) and (12.4) give,

$$\frac{A_g}{A} = \frac{1.5\,wN}{f_{ck}\,[0.4 + \frac{r}{f_{ck}}\,(0.67 f_y - 0.40\,f_{ck})]} \tag{12.5}$$

with w to be given in N/mm², as also f_{ck} and f_y in N/mm². With a grid of 6.0 m × 6.0 m, A = 36.0 m², w = 15 KN/m² = .015 N/mm², f_{ck} = 25 N/mm², f_y = 415 N/mm² and r = 0.01, Eq. (12.5) gives,

$$\frac{A_g}{A} = \frac{1.5 \times 0.015\,N}{25\,[0.4 + \frac{.01}{25}\,(0.67 \times 415 - 0.40 \times 25)]}$$

$$= \frac{0.02225\,N}{25(0.4 + .10722)} = \frac{N}{569.91}$$

with N = 15, A_g/A = .0263199, A_g = .95 m² = D^2, D = .97 m sq column

N = 8, A_g/A = .0140373, A_g = .505 m² = D^2, D = .71 m sq column

N = 4, A_g/A = .0070186, A_g = .253 m² = D^2, D = .50 m sq column

N = 2, A_g/A = .0035093, A_g = .126 m² = D^2, D = .35 m sq column

These column sizes can be rounded off to 0.9 m × 0.9 m for N =15, 0.6 m × 0.6 m for N = 8, 0.45'm × 0.45 m for N = 4 and 0.3 m × 0.3 m for N = 2. These sizes are quite reasonable and are often adopted in practice. It is seen that columns occupy about 2.66% the floor area for a building of 15 storeys, with a grid of 6.0 m × 6.0m.

The same approach can be adopted for an isolated shear wall, but the steel percentage can be taken at 0.4% for axial load only, this steel percentage being the minimum as given in the code. For shearwalled buildings, a full approximate analysis has to be made with a given configuration of shear walls as given in chapter 8. If the stresses in walls are not reasonable, a few more walls are added and the trial is repeated till a satisfactory arrangement of shear walls is achieved. The area of shearwalls in a tall building may work out to be 2% to 3% of the floor area. For a shear cored building of 20 storeys, the area enclosed by the shear core may come out to be about 20% of the floor area.

12.4.2 Final Column Design

The code gives two methods for design of reinforced concrete members subject to combined axial load and biaxial bending. The first method is based on the conditions of equilibrium, which are to be applied to a given section, on the basis of assumptions of the limit state of collapse in combined axial load and uniaxial bending given by the Code, with a suitably chosen inclined neutral axis. This method is extremely laborious, but computer is being used to help prepare charts[17,78,81] for ready use.

The second method of the Code makes use of an exponential relation, giving a unity check, using the already available interaction curves of members subject to combined axial load and uniaxial bending. This method involves a trial and error process and it is suitable only for design and not for checking of sections, as no idea of location of neutral axis is given by this method. Further, this method is reasonably accurate only for members under biaxial bending combined with direct compression, the error being large (though on the conservative side) when the axial load is zero or tensile. The major advantage of this method lies in the fact that different arrangements of steel in a given concrete section can be easily considered, as charts for design of sections under uniaxial bending combined with axial load for various steel arrangements are either available (see references 17, 58, 78, 79, 80) or can be easily developed.

In applying this method, both steel area and its arrangement in the column section are to be assumed at the outset. The iterative process can be short-ended if the initial value of steel area is properly selected. Everard[90] has suggested for this purpose to use steel area required for the section under axial load combined with a modified uniaxial moment M'_{ux}, which is given by

$$M'_{ux} = M_{ux} + M_{uy}\frac{D}{b} \qquad (12.6)$$

where M_{ux} = column moment about x-x axix
M_{uy} = column moment about y-y axix
D = depth of rectangular section, normal to x-x axis
b = width of rectangular section, normal to y-y axis

When M_{uy} is more prominent than M_{ux}, the modified moment about y-y axis can be assumed as,

$$M'_{uy} = M_{uy} + M_{ux}\frac{b}{D} \qquad (12.7)$$

This approach leads to a safe but conservative design.

BS 8110: part 1: 1985[91] Section 3, gives a direct method of design for a column section under axial load and biaxial bending and it is an improvement over Everard's [90] method. With reference to Fig. 12.4, which is the same as

Fig. 3.22 of BS 8110, the two moments M_x, M_y acting on a section can be reduced to a single moment about a given axis, using the following formulae:

(a) For $\frac{M_x}{h'} \geq \frac{M_y}{b'}$,

$$M'_x = M_x + \beta \frac{h'}{b'} M_y \tag{12.8}$$

(b) For $\frac{M_x}{h'} < \frac{M_y}{b'}$,

$$M'_y = M_y + \beta \frac{b'}{h'} M_x \tag{12.9}$$

where h, h', b and b' are as shown in Fig. 12.4 and β is given in Table 12.2, which is the same as Table 3.24 of BS:8110 and it shows that β depends on $P_u/f_{ck}bh$. This method, surprisingly, gives correct results and it avoids iteration altogether. It is a good breakthrough and saves time and effort of the designer.

Table 12.2. Values of coefficient β (BS:8110).

P_u/bhf_{ck}	0	.1	.2	.3	.4	.5	≥ .6
β	1.0	.88	.77	.65	.53	.42	.30

Sinha[81] is also doing an excellent work in the area of column design. Charts for design of columns under biaxial bending, given in the Manual[17] and by Ghanekar and Jain[78] involve interpolation in respecxt of $P_u/f_{ck}bD$. But Sinha and Narendra Kumar[92] give a direct method of design of rectangular column section under biaxial bending, wherein this type of interpolation is altogether avoided. This is a good step forward in column design.

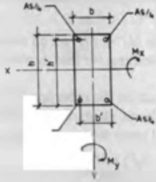


Fig. 12.4. Column section under biaxial bending (by BS8110).

12.4.3 Columns of Unusual Cross-Sections

Most of the available design aids apply to rectangular column sections. For circular or ring type sections, biaxial moments can be easily reduced to a resultant moment[17] equal to,

$$M_u = \sqrt{M^2 u_x + M^2 u_y} \tag{12.10}$$

The section is then designed for the given axial load P_u and the above moment M_u. Charts for solid circular sections are given in many places,[17,58,78,79,80] but charts for circular ring-shaped columns are given in the Manual.[17] Charts for design of hollow rectangular sections are given by Ranjan and Sinha.[93] Likewise, we expect to get many useful charts for column design in the reference 81, which is under printing.

In T- or L-column sections (Fig. 12. 5), P, M_x, M_y are'to act. Moments in columns have changing directions, so that the flange can be either in compression or in tension. Neglecting concrete in tension, only rectangular section becomes effective. so, we adopt the following procedure for design of L- or T- column sections.

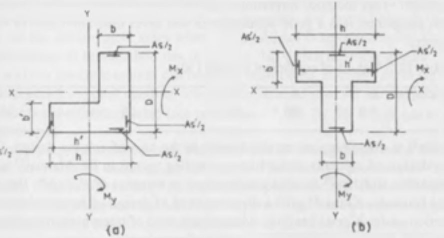


Fig. 12. 5 Design of flanged column. (a) L-shaped section; (b) T-shaped section.

(i) Let P and M_X (where $M_X >> M_Y$) act on one leg $b \times D$. By using column charts for uniaxial bending, we find A_s which is divided at $A_s/2$ opposite faces.

(ii) Let $P = 0$ and M_Y act on the other leg $h' \times h$. The value of A_s' can be found by using column charts for uniaxial bending. A_s' can also be found by using steel beam theory, which gives,

$$A_{sc} = A_{st} = \frac{A'_s}{2} = \frac{(LF) \times M_y}{h' \times 0.87 f_y} \tag{12.11}$$

Both approaches lead nearly to the same result.

(iii) Steel arrangement (Fig. 12.6) in the L- or T-column section shall be put in conformity with the assumptions made in the charts used in (i) and (ii) above. Nominal extra bars may have to be provided in the long legs in order to keep the bar spacing within 30 cm. Column links are also to be provided to hold all columns bars in position.

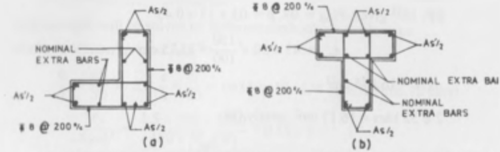


Fig. 12.6. Steel arrangement in columns. (a) L-shaped section; (b) T-shaped section.

12.4.4 Numerical Examples

(i) *Example of an L-shaped column section* (Fig. 12.7)

$P = 110$ t M15, Fe 415

$M_x = 154$ tm $f_{ck} = 150$ kg/m^2

$M_y = 57$ tm

a. Apply $P = 110$ t, $M_x = 154$ tm on rectangular section with $b = 50$ cm, $D =$ 150 cm.

$$\frac{P_u}{f_{ck}bD} = \frac{1.5 \times 110 \times 1000}{150 \times 50 \times (150)} = 0.147 \approx 0.15$$

$$\frac{M_{ux}}{f_{ck}bD^2} = \frac{1.5 \times 154 \times 10^5}{150 \times 50 \times (150)^2} = 0.137 \approx 0.14$$

With $d'/D = 5/150 = .033 \approx 0.05$, $A_s/2$ on E.F., SP 16[58] gives,

$$\frac{p}{f_{ck}} = 0.055,\ \text{P} = .055 \times 15 = 0.825$$

$$A_s = 0.825 \times 50 \times \frac{150}{100} = 61.88 \text{ cm}^2$$

$$8 \phi\ 32 \text{ bars} = 64.34 \text{ cm}^2$$

i.e. 4 ϕ 32 bars on each face provided (Fig. 12.7)

b. Apply $P = 0$, M_y =57 tm on a rectangular

Section with $b = 50$ cm, $d = 150$ cm

$$\frac{P_u}{f_{ck}bD} = 0$$

$$\frac{M_{uy}}{f_{ck}bD^2} = \frac{1.5 \times 57 \times 10^5}{150 \times 50\,(150)^2} = 0.05$$

SP: 16^{58} gives, $P/f_{ck} = .03, p = .03 \times 15 = 0.45$

$$A_s' = .45 \times 50 \times \frac{150}{100} = 33.75 \text{ cm}$$

$A_s'/2 = 16.875 \text{ cm}^2$

$2\ \phi\ 32$ bars = 16.17 cm^2, nearly OK.

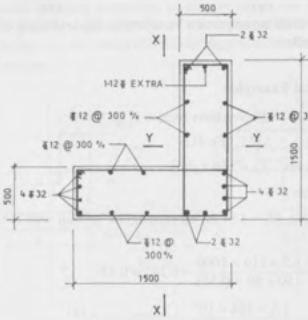


Fig. 12.7. Design of L-shaped column section.

Alternatively, we may use steel beam theory, which gives by Eq. (12.11),

$$A_{sc} = A_{st} = \frac{1.5 \times 57.0 \times 10^5}{138.8 \times .87 \times 4150} = 17.06 \text{ cm}^2$$

which is close to 16.875 cm^2 found above.

Extra steel ϕ 12/300 c/c provided on each face of either leg with suitable links at ϕ 8/200 c/c. Fig. 12.7 shows the steel arrngement in the given column section.

(ii) *Example of a Rectangular Column Section.* Fig. 12.8 gives the column section with the following data:

$P = 100$ t, M15, Fe 415
$M_x = 10.0$ tm $\quad b = 30$ cm
$M_y = 5.0$ tm $\quad D = 60$ cm

This example will be solved by different methods available to us.

Method I (*Ghanekar and Jain*[78])

$$\frac{h}{b} = \frac{60}{30} = 2.0,\ \frac{c}{h} = \frac{5}{60} = .0833 \approx .10,\ A_s/4 \text{ on E.F. (min 12 bars)}$$

$$v = \frac{P_u}{f_{ck}bD} = \frac{1.5 \times 100 \times 1000}{150 \times 30 \times 60} = 0.55 \approx 0.6$$

$$\mu_x = \frac{M_{ux}}{f_{ck}bD^2} = \frac{1.5 \times 10.0 \times 10^5}{150 \times 30 \times (60)^2} = 0.09$$

$$\mu_y = \frac{M_{uy}}{f_{ck}b^2D} = \frac{1.5 \times 5.0 \times 10^5}{150 \times (30)^2 \times 60} = 0.09$$

$$w = 0.75 = \frac{A_s f_y}{b\,D\,f_{ck}}$$

$$A_s = 0.75 \times 30 \times \frac{60 \times 15}{415} = 48.8 \text{ cm}^2$$

4 ϕ 20 + 8 ϕ 25 bars give an area = 12.56 + 39.28 = 51.84 cm with p = 2.9%. The steel arrangement is shown in Fig. 12.8.

Fig. 12.8. Design of rectangular column section (Ghanekar and Jain[78])

Method II (*Using Evarard's suggestion*)

$P = 100$ t
$M_x' = M_x + M_y\,D/b$
$= 10 + 5 \times \frac{60}{30} = 10 + 10 = 20.0$ tm

$$\frac{P_u}{f_{ck}bD} = \frac{1.5 \times 100 \times 1000}{150 \times 30 \times 60} = 0.55$$

$$\frac{M'_{ux}}{f_{ck}D^2} = \frac{1.5 \times 20.0 \times 10^5}{150 \times 300\,(60)^2} = 0.185$$

With $d'/D = 5/60 = .0833 \approx .10$, $A_s/4$ on E.F., SP. 16[58] gives,

$$\frac{p}{f_{ck}} = 0.2, p = .2 \times 15 = 3.0$$

$$A_s = 3.0 \times 30 \times \frac{60}{100} = 54.0 \text{ cm}^2$$

12 $\bar{\phi}$ 25 bars give = 58.9 cm^2

The steel arrangement is shown in Fig. 12. 9.

This is a preliminary design, which must be checked by the second method of the Code.

Referring to Fig.12.9, we have

X-X: $A_s = 58.9$ cm^2 $p = 3.27$, $A_s/4$ on E.F., $p/f_{ck} = .218$.

$$\frac{P_u}{f_{ck}bD} = 0.55, d'/D = .10, \text{SP–16}^{58} \text{ gives,}$$

$$M_{ux}1 = .205 \times 150 \times 30 \times (60)^2/10^5 = 33.21 \text{ tm}$$

$$M_{ux} = 1.5 \times 10 = 15.0 \text{ tm}$$

Y-Y: $A_s/4$ on E.F., $A_s = 58.9$ cm^2, $p = 3.27$, $p/f_{ck} = .218$

$$\frac{P_u}{f_{ck}bD} = .55, b'/b = 5/30 = .167 \approx 0.20, \text{SP:16}^{58} \text{ gives,}$$

$$M_{uy}1 = 0.155 \times 150 \times (30)^2 \times 60/10^5 = 12.56 \text{ tm}$$

$$M_{uy} = 1.5 \times 5.0 = 7.5 \text{ tm}$$

$$\alpha_n: \alpha_n = \frac{2}{3} + \frac{5}{3}\,\frac{P_u}{P_{uz}}$$

$$P_u = 1.5 \times 100 = 150 \text{ t}$$

$$P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}$$

$$= 0.45 \times .15\,(30 \times 60 - 58.9) + 0.75 \times 4.15 \times 58.9$$

$$= 117.5 + 183.3 = 300.8 \text{ t}$$

$$\alpha_n = 0.67 + \frac{5}{3} \times \frac{150}{300.8} = 1.5$$

$$\text{Check: } \left(\frac{M_{ux}}{M_{ux}1}\right)^{\alpha n} + \left(\frac{M_{uy}}{M_{uy}1}\right)^{\alpha n}$$

$$= \left(\frac{15}{33.21}\right)^{1.5} + \left(\frac{7.5}{12.56}\right)^{1.5} = 0.304 + 0.461 = 0.765 < 1.0$$

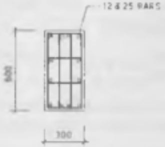


Fig. 12.9. Design of rectangular column section (by Everard).

This is oversafe. Everard's suggestion generally leads to a conservative design ($p = 3.27\%$).

Method III (*Using method of* BS:8110). With $P = 100$ t, $M_x = 10$ tm, $M_y = 5$ tm and referring to Fig. 12.10, we have

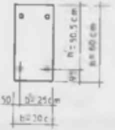


Fig. 12.10. Values of b' and h' as per BS 8110.

$h' = 50.5$ cm, $b' = 25$ cm, $h = 60$ cm, $b = 30$ cm

$$\frac{M_x}{h'} = \frac{10.0}{0.505} = 19.8,$$

$$\frac{M_y}{b'} = \frac{5.0}{.25} = 20.0,$$

As, $\frac{M_y}{b'} > \frac{M_x}{h'}$, Eq. (12.9) gives,

$$M_y' = M_y + \beta \frac{b'}{h'} M_x$$

For $\frac{P_u}{f_{ck}bD} = 0.55$, Table 12.2 gives,

$\beta = 0.36$

$$M'_y = 5.0+0.36 \times \frac{25}{50.5} \times 10.0$$

$$= 5.0 + 1.8 = 6.8 \text{ tm}$$

For $\frac{P_u}{f_{ck}bD} = 0.55$

$$\frac{M'_{uy}}{f_{ck}b^2D} = \frac{1.5 \times 6.8 \times 10^5}{150 \times (30)^2 \times 60} = 0.126 \approx .13$$

SP: 16[58] gives, for $A_s/2$ on E.F, $b'/b = 5/30 = .167 \approx 0.20$

$$p/f_{ck} = .16, p = .16 \times 15 = 2.4$$

$$A_s = 2.4 \times 30 \times \frac{60}{100} = 43.2 \text{ cm}^2$$

8 $\bar{\phi}$ 28 bars = 49.26 to be arranged as shown in Fig. 12.11. 2 $\bar{\phi}$ 12 extra bars may be placed on the centre of long sides. Overall steel area = 51.52. with $p = 2.9\%$.

The steel percentage is the same as given by Method I, but the steel arrangements are different.

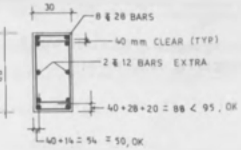


Fig. 12.11. Design of rectangular column section as per BS 8110.

Check by Unity Method

X-X: 8 $\bar{\phi}$ 28 bars, $A_s = 49.26 \text{ cm}^2$, $p = 2.737$, $p/f_{ck} = .18$,

$$\frac{P_u}{f_{ck}bD} = 0.55, d'/D = 9.5/60 = 158 \approx 0.15. A_s/2 \text{ on E.F}$$

$$M_{ux}1 = 0.17 \times 150 \times 30 \times (60)^2/10^5 = 27.54 \text{ tm}$$

$$M_{ux} = 1.5 \times 10 = 15 \text{ tm}$$

Y-Y: 8 $\bar{\phi}$ 28 bars, $A_s = 49.26 \text{ cm}^2$, $p = 2.737$, $p/f_{ck} = .18$,

$$\frac{P_u}{f_{ck}bD} = .55, b'/b = 5/30 = .167 \approx .20,$$

$$M_{uy}1 = 0.15 \times 150 \times (30)^2 \times 60/10^5 = 12.15 \text{ tm}$$

$$M_{uy} = 1.5 \times 5 = 7.5 \text{ tm}$$

α_n: $\quad P_u = 1.5 \times 100 = 150 \text{ t}$

$$P_{uz} = 0.45 \times .15(1800 - 49.26) + .75 \times 4.15 \times 49.26$$

$$= 118 + 153 = 271 \text{ t}$$

$$\alpha_n = 0.67 + \frac{5}{3} \times \frac{150}{271} = .67 + 0.92 = 1.59$$

Check $\left(\frac{15}{27.54}\right)^{1.57} + \left(\frac{7.5}{12.15}\right)^{1.57} = 0.38 + 0.46 = 0.84 < 1.0$, safe

(ii) *Example of a thin rectangular column.*

Data $P = 100$ t $\quad b = 30$ cm $\quad f_y = 4150$ kg/cm²

$M_x = 15.0$ tm $\quad D = 90$ cm

$M_y = 5.0$ tm $\quad f_{ck} = 150$ kg/cm²

We solve this problem by different methods.

Method 1 (*Sinha and N. Kumar*): Using 10-bar arrangement (Fig.12.12)

$A_g = 30 \times 90 = 2700$ cm²

$Z_x = 30 \times (90)^2/6 = 40,500$ cm³

$Z_y = 90 \times (30)^2/6 = 13500$ cm³

$$\frac{P_u}{f_{ck}A_g} = \frac{1.5 \times 100 \times 1000}{150 \times 2700} = 0.37$$

$$\frac{M_{ux}}{f_{ck}Z_x} = \frac{1.5 \times 15.0 \times 10^5}{150 \times 40,500} = 0.37$$

$$\frac{M_{uy}}{f_{ck}Z_y} = \frac{1.5 \times 5.0 \times 10^5}{150 \times 13,500} = 0.37$$

$$D/b = 90/30 = 3.0,\ d'/D = \frac{5.6}{90} = .06 \approx .05$$

$$\frac{100\, A_s f_y}{f_{ck}\, A_g} = 30$$

$$A_s = \frac{30 \times 15 \times 2700}{100 \times 415} = 29.28 \text{ cm}^2$$

$10\ \phi\ 20 = 31.41$ cm², $p = 1.16\%$

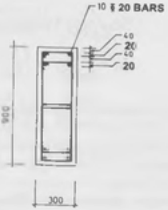


Fig. 12.12. Design of a thin rectangular column section (10-bar arrangement).

Check by Unity Method

Use 8 ϕ 20, $A_s = 25.13$ cm², $p = 0.93$, $p/fck = .062$

$P_u/f_{ck}bD = 0.37$, $d'/D = 9.5/90 = .10$

$M_{ux1} = 0.10 \times 150 \times 30 \times (90)^2/10^5 = 36.5$ tm

$M_{ux} = 1.5 \times 15 = 22.5$ tm

Y-Y: 10 $\bar{\phi}$ 20, $A_s = 31.41$ cm^2, $p = 1.16$, $p/f_{ck} = .08$.

$$\frac{P_u}{f_{ck}bD} = .37,\ b'/b = \frac{5.25}{30} = .175 \approx .20$$

$$M_{uy}1 = .10 \times 150 \times (30)^2 \times 90/10^5 = 12.2 \text{ tm}$$

$$M_{uy} = 1.5 \times 5 = 7.5 \text{ tm}$$

α_n: $$P_u = 1.5 \times 100 = 150 \text{ t}$$

$$P_{uz} = .45 \times .15\,(2700 - 31.41) + .75 \times 4.15 \times 31.41 = 180 + 90 = 278\text{t}$$

$$\alpha_n = .67 + \frac{5}{3} \times \frac{150}{278} = .67 + .90 = 1.57$$

check: $$\left(\frac{22.5}{36.5}\right)^{1.57} + \left(\frac{7.5}{12.2}\right)^{1.57} = .47 + .47 = 0.94 < 1.0.$$ The design is safe.

Method II (*Ghanekar and Jain*[78])

$$\frac{h}{b} = \frac{90}{30} = 3.0,\quad \frac{c}{h} = \frac{5.6}{90} = .062 \approx .10,\ A_s/4 \text{ on E.F}$$

$$P = 100 \text{ t},\ M_x = 15.0 \text{ tm},\ M_y = 5.0 \text{ tm}$$

$$v = \frac{P_u}{f_{ck}\,bD} = \frac{1.5 \times 100 \times 10000}{150 \times 30 \times 90} = 0.37 \approx 0.4$$

$$\mu_x = \frac{M_{ux}}{f_{ck}bD^2} = \frac{1.5 \times 15.0 \times 105}{150 \times 30 \times (90)^2} = 0.06$$

$$\mu_y = \frac{M_{uy}}{f_{ck}b^2D} = \frac{1.5 \times 5.0 \times 105}{150 \times (30)^2 \times 90} = 0.06$$

$$w = 0.4 = \frac{A_s \times f_y}{b\,D\,f_{ck}}$$

$$A_s = 0.4 \times 30 \times 90 \times \frac{15}{415} = 39.04 \text{ cm}^2$$

12 $\bar{\phi}$ 20 bars = 37.96 cm^2, $p = 1.4\%$z

Fig. 12.13. Design of a thin rectangular column section ($A_s/4$ on each face).

The steel arrangement is given in Fig. 12.13, the controlling factor being that 4 $\bar{\phi}$ 20 bars can be accommodated in a width of 30 cm.

Check by Unity Method

XX: 12 ϕ 20 bars, $A_s = 37.69$ cm^2, $p = 1.396$, $p/f_{ck} = .093$,

$$\frac{P_u}{f_{ck} bD} = .37,\ d'/D = 5/90 = .055 \approx .05,\ A_s/4 \text{ on E.F.}$$

$$M_{ux1} = 0.13 \times 150 \times 30 \times (90)^2 / 10^5 = 47.4 \text{ tm}$$

$$M_{ux} = 1.5 \times 15 = 22.5 \text{ tm}.$$

YY: 12 ϕ 20 bars, $A_s = 37.69$ cm^2, $p = 1.396$, $p/f_{ck} = .093$

$$\frac{P_u}{f_{ck} bD} = ..37,\ b'/b = 5/30 = .167 \approx .20,\ A_s/4 \text{ on E.F.}$$

$$M_{uy1} = .095 \times 150 \times (30)^2 \times 90 / 10^5 = 11.5 \text{ tm}$$

$$M_{uy} = 1.5 \times 5 = 7.5 \text{ tm}$$

$$P_u = 1.5 \times 100 = 150 \text{ t}$$

$$P_{uz} = 0.45 \times .15\,(2700 - 37.69) + .75 \times 4.15 \times 37.69$$

$$= 180 + 117 = 297 \text{ t}$$

$$\alpha_n = .67 + \frac{5}{3} \times \frac{150}{297} = 67 + .84 = 1.51$$

Check: $\left(\frac{22.5}{47.4}\right)^{1.51} + \left(\frac{7.5}{11.5}\right)^{1.51} = 0.325 + 0.524 = 0.849 < 1.0$, safe.

12.5 Footing Design

All footings are designed on the basis of an inverted floor with column loads acting on top and a uniform soil pressure acting from below (Fig. 12.14a). The centre of gravity of the footing area and the point of resultant of all column loads shall lie on the same point, this case will give uniform soil pressure. In case, the resultant of column loads is eccentric with respect to the centre of gravity of the footing area, the soil pressure diagram is of a trapeozidal nature (Fig. 12.14b). In Fig 12.14c, when the eccentricity is large, a triangular soil pressure diagram may also be given. With reference to Fig. 12. 14a,

$$p = \frac{P}{AB} \tag{12.12}$$

where, A = short side of footing

B = long side of footing

With reference to Fig. 12. 14b,

$$p = \frac{\Sigma P}{AB}\left(1 \pm \frac{6e}{B}\right)$$

where ΣP = resultant of all column loads

B = length of footing

A = width of footing

e = eccentricity of ΣP w.r.t the centre of footing

When,

$$\frac{e}{B} = \frac{1}{6}, p_1 = \text{maximum soil pressure} = 2\frac{\Sigma P}{AB}$$

$$p_2 = \text{minimum soil pressure} = 0$$

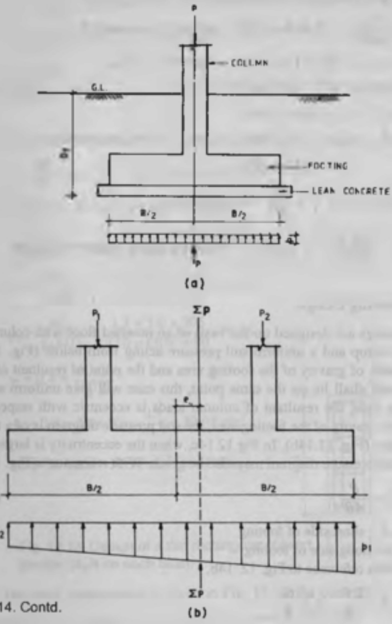


Fig.12.14. Contd.

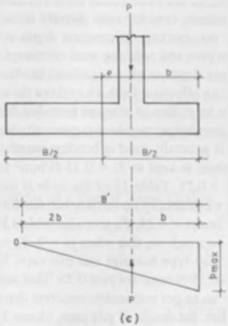


Fig. 12.14. Soil pressure on footing. (a) Uniform rectangular soil pressure; (b) Trapezoidal soil pressure; (c) Triangular soil pressure.

When equation (12. 13) gives a negative value for p_2, it indicates tension, which the soil is not capable of resisting. Then the effective footing area will be AxB' where (Fig.12.14c)

$$B' = 3.b = 3\left(\frac{B}{2} - e\right) \tag{12.14}$$

where B' = effective length of footing

The maximum soil pressure, p_{max}, in such casses, is given by,

$$P = \frac{1}{2} B' p_{max}$$

$$\text{or, } p_{max} = \frac{2P}{B'} = \frac{2P}{3\,(B/2 - e)} \tag{12.15}$$

This gives a triangular soil pressure diagram (Fig.12.14c).

Isolated footings are slab-type footings, with slab thickness taken as uniform or variable linearly or in steps. The design of isolated footings with useful design aids is given in the Manual.[17] Combined and strip footings are slab-cum-beam type footings. Rafts can be either slab-type or beam-slab type. Bouyant foundation is a cellular type raft, while cutouts may be made in plan of rafts, giving what may be called annular rafts. The design of elements like slabs and beams will be done on the basis of an inverted floor. In general concrete depths are kept on the high side in order to save steel. In superstructure design, concrete depths are selected on the low side in order to reduce dead load. But, in foundations, concrete rests directly on soil, which does not cause bending effect, so this increase in concrete depth is beneficial in providing fixity to the column base and reducing steel consumption in foundations.

Large concrete depth is also beneficial in shear design as the critical section for shear is an effective depth away from the column face. This aspect causes reduction in steel area of stirrups provided for shear resistance. Clear concrete cover in foundation members is generally kept at 40 mm, this value is more than what is generally used in bending members of the superstructure. Allowable shear stress is kept at $T_c = 0.35$ N/mm^2 for $p_t \leq 0.25$. When the steel percentage $p_t > 0.25$, Table 13 of the code is used to get value of T_c. In slab-type footings we generally do not use stirrups for reasons of convenience in construction. When $p_t < 0.25$, T_c given by table 13 of the Code gives very low values for T_c, so much so, that when $p_t = 0$, $T_c = 0$. This leads to a large concrete depth for slab-type footings and pile caps. For this reason, it is proposed to have $T_c = 0.35$ N/mm^2 for $p_t \leq 0.25$. This amedment in the Code has been suggested by us to get reasonable concrete depth for slab-type footings and pile caps. Further, for design of pile caps, clause 39.2.2 of the Code gives,

$$\delta = 1 + \frac{3\,P_u}{A_g f_{ck}} \quad (\not> 1.5) \qquad (12\ 16)$$

where, δ = multiplication factor for T_c

T_c = allowable shear stress

P_u = axial compressive force in Newtons

A_g = dispresed area of concrete section at mid-depth of pile cap in mm^2

f_{ck} = characteristic compressive strength of concrete in N/mm^2.

Alternatively, we may provide stirrups in slab type footings and pile caps in order to resist shear.

This way, we may avoid using chairs which, otherwise, will be required to support top mesh of steel bars in slab type footings. Numerical examples of design of footings have already been given in chapter 11.

Plinth beams and retaining walls, being placed below ground, are generally included in foundations. Plinth beams serve two functions:

i) They connect columns in the two principal directions to act as earthquake ties. This causes compression or tension in plinth beams.
ii) They support brick walls resting on top of plinth beams. This causes bending and shear in plinth beams. Brick work in foundation is saved by providing plinth beams.

As plinth beams may rest directly on soil, a part of the load may go directly to the ground and further, the concrete plinth beam and the brick wall resting on it, act together as a deep beam. For these reasons, some authorities suggest very low value for bending moment in a plinth beam of the order of $WL/30$ only. But, in practice, both the ground support and the presence of brick walls may not be certain, so, a moment of $\pm WL/10$ may be taken for design of plinth beams. This moment together with axial tension causd by earthquake with a load factor of 1.2 will govern the design of plinth beams. Column base moments may also be given to plinth beams, so that, footings can then be designed only for vertical column loads. Design of retaining walls has been covered in the Manual.[17]

12.5.1 Numerical Example of a Plinth Beam Design

With $b = 23$ cm, $D = 60$ cm, $L = 6.0$ m

w: selfweight .23 × .6× 1.0 × 2.5 = 0.35 t/m

brick wall above .23 (3.0 + 0.6+ .15) = 1.65 t/m

w = 2.00 t/m

$$M = \pm\frac{wL^2}{10} = \pm 2.0 \times \frac{6.0^2}{10} = \pm 7.2 \text{ tm}$$

$$S = wL/2 = 2.0 \times \frac{6.0}{2} = 6.0 \text{ t}$$

T (due to EQ) = 7.5 t

$$\frac{P_u}{f_{ck}bD} = -\frac{1.2 \times 7.5 \times 1000}{150 \times 23 \times 60} = -0.0435$$

$$\frac{M_u}{f_{ck}bD^2} = \frac{1.2\times7.2\times10^5}{150 \times 23 \times (60)^2} = 0.07$$

With $d'/D = \frac{5}{60} = .083 \approx .10$, $A_s/2$ on EF, SP-16[58] gives.

$$\frac{p}{f_{ck}} = .06,\ p = .06 \times 15 = 0.9$$

$$A_s = .9 \times 23 \times \frac{60}{100} = 12.42 \text{ cm}^2$$

4 ϕ 20 bars give an area of 12.56 cm^2, i.e. 2 ϕ 20 bars top and bottom will be provided.

Check for the case without EQ.

$M = 7.2$ tm, $b = 23$ cm, $d = 55$ cm

$$K = \frac{1.5 \times 7.2 \times 10^5}{23 \times (55)^2 \times 10} = 1.55 \text{ N/mm}^2$$

$$A_{st} = .503 \times \frac{23 \times 55}{100} = 6.36 \text{ cm}^2$$

2 ϕ 20 bars = 6.28 cm², nearly OK

$$T_v = \frac{1.5 \times 6.0 \times 1000}{23 \times 55 \times 10} = 0.71 \text{ N/mm}^2$$

For $p_t = 0.5$, $T_c = 0.46$

$V_{us}/_d = (.071 - .046) \times 23 = 0.575$ KN/cm

Min. $V_{us}/d = 0.348$ b = $.0348 \times 23 = 0.80$ KN/cm

ϕ 8/200 c/c stirrups have capacity 1.815 KN/cm.

CHAPTER 13

Detailing of Reinforced Concrete Members and Structures

13.1 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 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 floors, while the floor plan giving slab design is kept the same as the typical floor. 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

$$L_d = \phi \frac{\sigma_s}{4\, T_{bd}}$$

$$= \phi \frac{0.87 \times 415}{4 \times 1.0 \times 1.6} = 56.4\,\phi \approx 57\phi \qquad (13.1)$$

where, ϕ = diameter of bars

σ_s = stress in bar at the section considered at design load

T_{bd} = design bond stress = 1.0×1.6 for M15 and Fe415 deformed bars

L_d = lap length or development length of bar

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 where labour 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 cage formation of steel bars or the top bars are better supported on 10mm diameter stirrups than on 8mm diameter stirrups which may get buckled during the process of bar placement. In columns too, the practice is to use a few large diameter bars in order to avoid congestion and achieve good concreting in vertical moulds. Footings are large-sized concrete members and large diameter bars will be required in all their components. In general, 8mm diameter bars are not provided in footings, as these are buried in the ground and the concrete remains in constant contact with earth.

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

slabs: 0.12% of gross area

$$\text{beams: } \frac{A_s}{bd} = \frac{0.85}{f_y} = \frac{0.85}{415} = 0.00205 \tag{13.2}$$

or 0.205% of effective area

where, A_s = minimum area of tension reinforcement

b = width of beam

d = effective depth of beam

Side face steel = 0.1% of gross area, where beam depth exceeds 750 (without torsion) or 450 mm (with torsion.)

Minimum shear reinforcement (A_{sv})

$$\frac{A_{sv}}{bS_v} \geq \frac{0.4}{f_y} \tag{13.3}$$

$$\text{or min } \frac{V_{us}}{d} \text{ (KN/cm)} = 0.87 f_y \frac{A_{sv}}{S_v} = 0.4b \times 0.87 = 0.348b \text{ (in mm)}$$

$$= 0.0348b \text{ (in cm)} \tag{13.4}$$

For wide beams ($b > d$), B.P. Hughes[94] gives,

$$\frac{A_{sv}}{dS_v} \geq \frac{0.4}{f_y} \tag{13.5}$$

$$\text{or} \quad \text{min } \frac{V_{us}}{d} \text{ (KN/cm)} = 0.0348 \text{ d (in cm)} \tag{13.6}$$

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 against 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 $\ngtr$ 4.0% of gross area of section.

Max. compression steel $\ngtr$ 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.

13. 2 SLAB DETAILING

In one-way and two-way continuous slab panels, bent up bars are used as shown in Fig. 13.1. 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. 13.1a, 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.

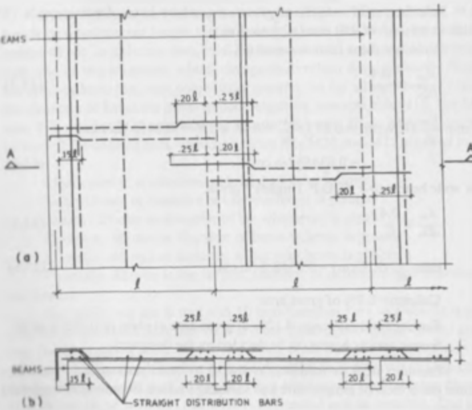


Fig. 13.1. Steel arrangement in one-way and two-way continuous slab panels. (a) Plan of one wayslab panels; (b) Section A-A.

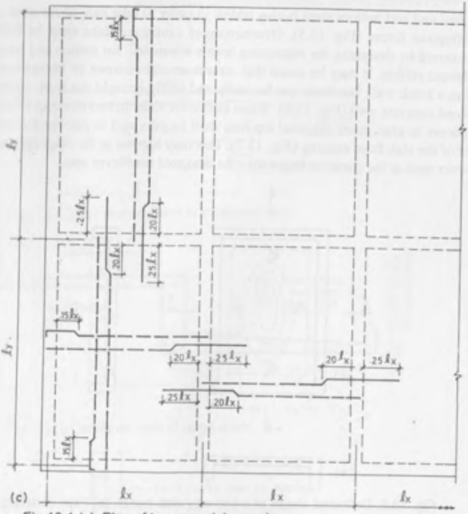


Fig.13.1 (c). Plan of two-way slab panels.

In two-way slab panels,bars are bent at $0.15l_x$ or $0.2l_x$and $0.25l_x$ in both the directions (Fig. 13.1c). The reason for this practice is found in the shape of the deflected shape of a two-way rectangular slab panel (Fig. 13.2). When a small panel is sandwitched by two bigger panels or a small panel is continuous with a big panel, there is a likelyhood 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. 13.3) When a slab panel is small, say, 2.0 m 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 × dia. of bar) in the adjoining slab panel or beam (Fig. 13.4). When a cantilever span is large, it is better to introduce cantilever beams and make 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. 13.5). 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. 13.6). When cantilever slabs in two directions form a corner in plan, extra diagonal top bars shall be provided to prevent the corner of the slab from sagging (Fig. 13.7). This may happen as the diagonal cantilever span at the corner is larger than the designed cantilever span.

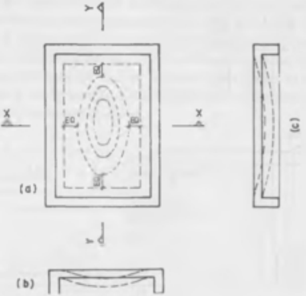


Fig. 13.2. Deflected shape of a two-way slab panel. (a) Plan of defected slab panel; (b) Section X-X; (c) Section Y-Y.

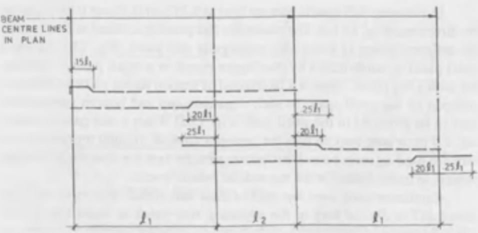


Fig. 13.3. Small panel sandwitched by bigger panels.

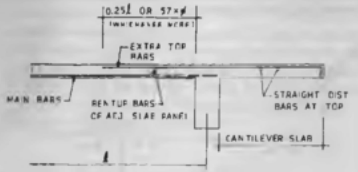


Fig. 13.4 Steel arrangement in cantilever slab.

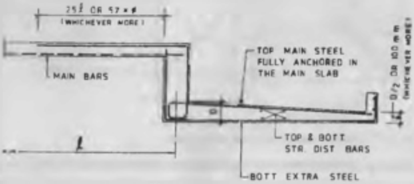


Fig. 13. 5 Cantilever slab of large span.

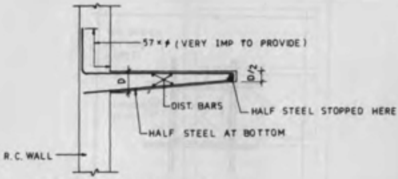


Fig. 13.6 Cantilever slab from a RC wall.

Small openings in slabs, which do not alter their structural behaviour, are to be provided with two extra bars (ϕ 10 or ϕ 12 bars) on all four sides with anchorage length on either side (Fig. 13.8). However, when an opening in slab is large, it may change the structural behaviour of the slab panel. For example, if ther is a huge cut-out in a two-way slab at the centre, it will not behave as a two-way slab. Different strips will be designed as one-way slabs separately (Fig. 13.9), with extra bars around the opening or a large opening is to be surrounded by subsidiary beams (Fig. 13.10), then extra bars in slab around the opening will not be required.

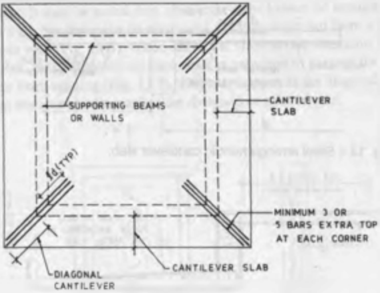


Fig. 13.7. Plan of extra top bars at corners.

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. 13.10A).

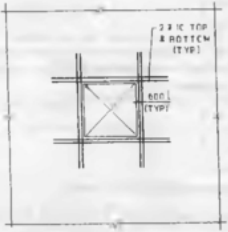


Fig. 13.8. Plan of small cut-out in a slab panel.

13. 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 corners of shear stirrups. When the span (L) is long, say, 6.0 m or more, stirrups are closely spaced at $L/4$ distance near supports and these are more widely spaced in the centre half of span (Fig. 13. 11).

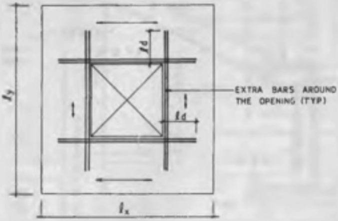


Fig. 13. 9. Plan of a large opening in a two-way slab.

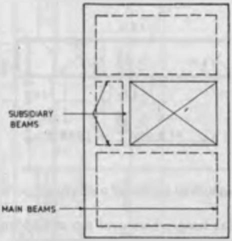


Fig. 13.10. Extra beams around large cut-outs in slab in plan.

In continuous beams, straight bars are used at all critical sections with 2 Nos. bars of the same diameter are provided through, top and bottom. Bent-up bars in beams are not being used in practice, being costly in labour. Fig. 13.12 gives distances for curtailment of bars in continuous beams. In some offices, 2

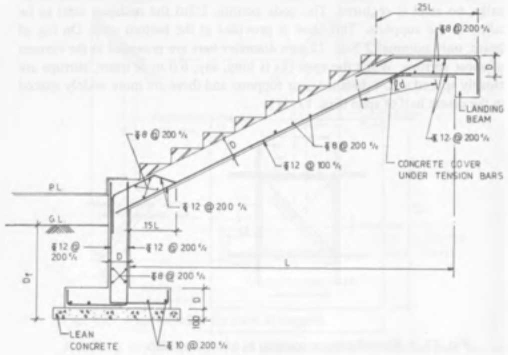


Fig. 13.10A. Detailing of a flight slab.

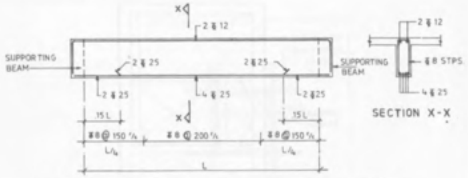


Fig. 13.11. Longitudinal section of a simply supported beam.

ϕ 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 more than 2 ϕ 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.

13.12). Where a short span is adjacent to a long span, there is all likelyhood of getting negative moment at the midspan of the short span, for which top steel bars from support shall be extended on top of the short span length (Fig. 13.13).

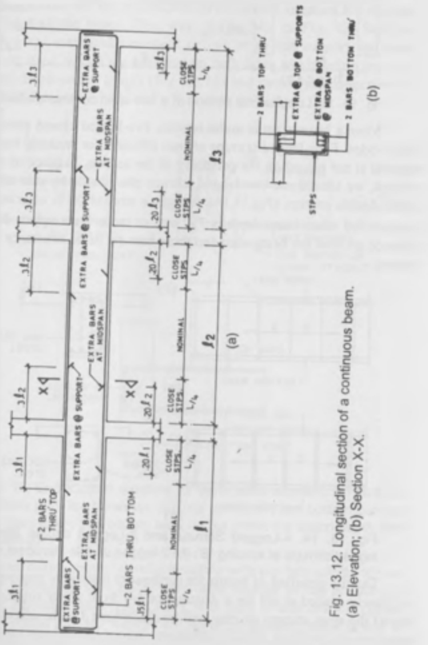


Fig. 13.12. Longitudinal section of a continuous beam.
(a) Elevation; (b) Section X-X.

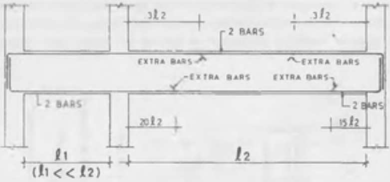


Fig. 13. 13. Longitudinal section of a two-span continuous beam.

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.13.14). 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.

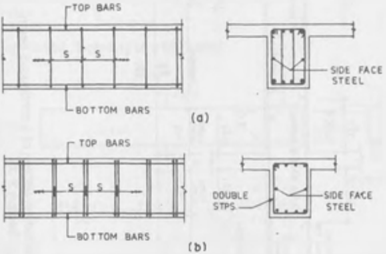


Fig. 13. 14. 4-Legged Stirrups and 2-Legged double stirrups. (a) 4-legged stirrups at spacing 'S'; (b) 2-legged double stirrupsat spacing 'S'.

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. 13.15a,b,c. These three details are in use in differnt design offices. Detail of Fig. 13.15a is satisfactory, but it leads to congestion at the beam-column joint. The detail of Fig. 13.15b 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. 13.15c is mostly used in our office, where it is felt that the longitudinal bars of the cross beam will compensate for the insufficient development lengh of the top and the bottom bars of the beam. This way, the pitfalls of Fig. 13.15a,b are avoided. When the earthquake moment is high at an end support, hair-pin-shaped bars are used, which are effective on both faces and have no problem of inadequate development length (Fig. 13.16) and cause less congestion at the beam-column joint.

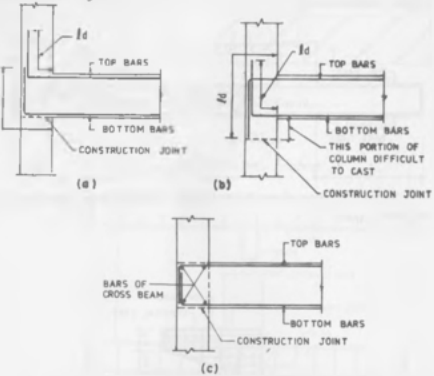


Fig. 13. 15. End support detailing for earthquake moment. (a) Both top and bottom bars are taken up; (b) Top bars down and bottom bars up; (c) Top bars down and bottom bars up but within the beam depth (with insufficient l_d).

Cantilever beams require careful detailing with top bars adequately anchored in the adjoinging span and it may cause tension on top at mid-span section of the adjoining span (Fig. 13.17). 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 regarded as a bracket, which needs both the horizontal and the vertical stirrups (Fig. 13. 18).

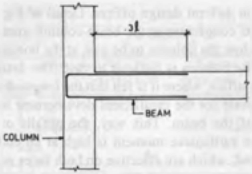


Fig. 13.16. Extra U-bars to be provided at an end support where earthquake moment is high.

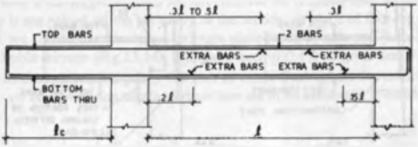


Fig. 13.17. Detailing of cantilever (l_c) with top bars fully anchored in the adjoining span.

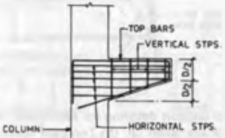


Fig. 13. 18. Detailing of a bracket from a column.

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 embedded 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. 13.19).

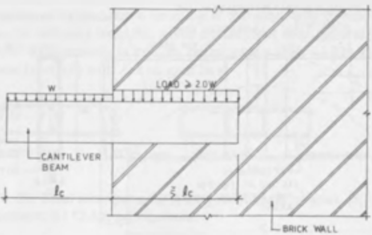


Fig. 13.19. Cantilever beam from a brick wall.

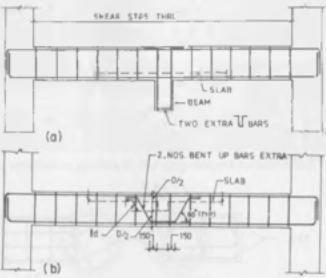


Fig. 13. 20. (a) Extra long bars to support beam at the lower level; (b) Extra bent-up bars under concentrated load of the supporting beam.

When a beam rests at the centre of a beam, shear stirrups as required by design shall be provided at the uniform spacing throughout the span. In reinforced concrete construction, the supporting beam may be deeper or even shallower than the beam supported. They may not be even at the same level, but the floor slab shall connect both the beams. Extra stirrups or U-bars shall be provided to resist the tension equal to the beam reaction (Fig. 13.20a). Bent-up bars may be provided to distribute the concentrated load (Fig. 13. 20b), this being a good German practice.

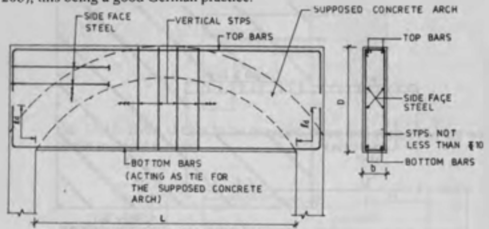


Fig. 13. 21 Elevation of a deep beam.

In deep beams, bottom bars shall be taken through to the supports without 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. 13.21). 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.

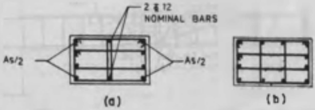


Fig. 13.22. Steel arrangement in a column section. (a) $A_s/2$ on opposite faces; (b) $A_s/4$ on each face (12-bars arrangement shown as an example, 8 to 24 bars can be provided).

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 difficulties at the site.

13.4 COLUMN 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. 13.22), (i) with $A_s/2$ on oppsite faces; (ii) with $A_s/4$ on each face.

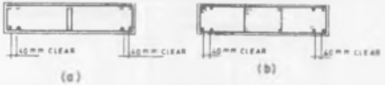


Fig. 13. 23. Steel arrangements in long rectangular column. (a) 10-bar arrangement; (b) 12-bar arrangement.

The first arrangement (Fig. 13. 22a) is suitable for a column under uniaxial bending, while the other one (Fig. 13.22b) is effective for a column under biaxial bending, for which square or squarish column sections are suitable. For long rectangular column, 10- or 12-bar arrangements as given in Fig. 13.23 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 be provided on long faces (Fig. 13. 22a). Minimum diameter of a column bar is specified to be 12 mm. In circular columns, minimum numbers of bars shall be six, to be spaced uniformly on the periphery of the column section (Fig. 13.24). The same restriction applies to the ring-shaped columns too.

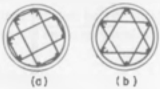


Fig. 13.24. Steel arrangement in circular columns. (a) 8-bar arrangement with 2-nos. rectangular ties and one circular tie; (b) 6-bar arrangement with 2-nos. triangular tiesand one circular tie.

For column ties, the rule is to hold all column bars at the corners of column ties, unless these are spaced at 75 mm or less. In some cases, open links may also be provided, these are suitable for wall-type columns. The diameter of column ties shall be 1/4th the diameter 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. 13.24). For ductility requirements in earthquake-prone areas, 8mm diameter ties at 100mm c/c are to be provided in 1/6th the storey height near floor levels and in the mid-height portion, 8mm diameter bars at 200mm c/c are to be provided (Fig. 13.25).

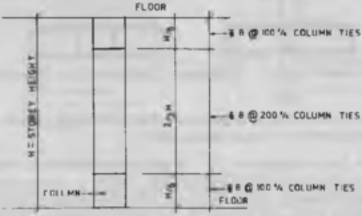


Fig. 13.25. Column ties in earthquake-prone areas.

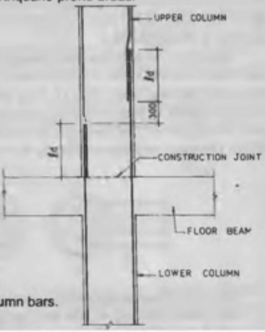


Fig. 13. 26. Staggered laps for column bars.

schedule, along with a typical detail for variation in column ties along a storey height (Fig. 13.25). 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. 13.26). Column size may also be varied along the height of building, this is particularly done for internal columns (Fig. 13.27). External column size is kept the same throughout the height of building, for elevational purposes. If the variation in column steel for any two storeys is small or negligible, column 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.

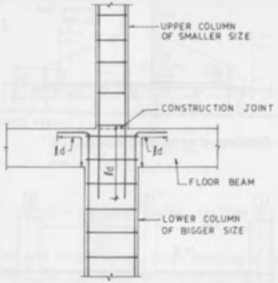


Fig. 13.27. Detail of column bars when lower column is bigger and the upper column is small.

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 column 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.

13. 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, particulatly in the case of isolated footings of small dimensions in plan (Fig. 13.28). Column bars shall be fully anchored into the footings with the development length of column bars into the footing concrete.

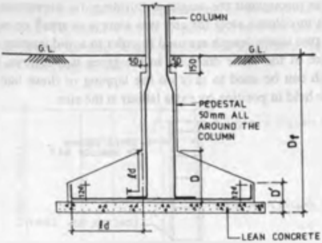


Fig. 13.28. Deatiling of an isolated footing.

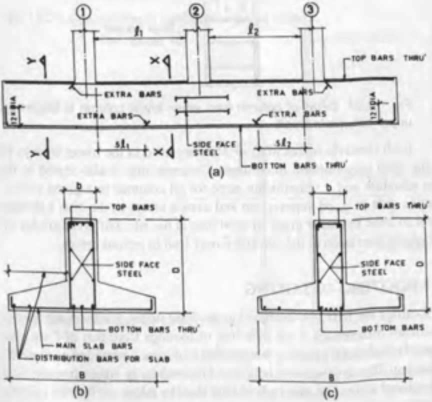


Fig. 13.29. Detailing of strip foundation. (a) Elevation; (b) Section X-X; (c) Section Y-Y.

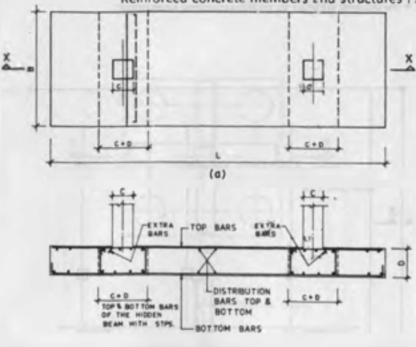


Fig. 13.30. Slab-type combined footing. (a) Plan of footing; (b) Section X-X.

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. 13.29), 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. 13.30). 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 strength (T_c) equal to 0.35 N/mm^2, when $p_t \leq 0.25$ (Fig. 13.31). Pile caps are given stirrups in both directions, with an additional bottom mesh of steel if necessary (Fig. 13.32).

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. 13.33). 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 (Fig.13.34).

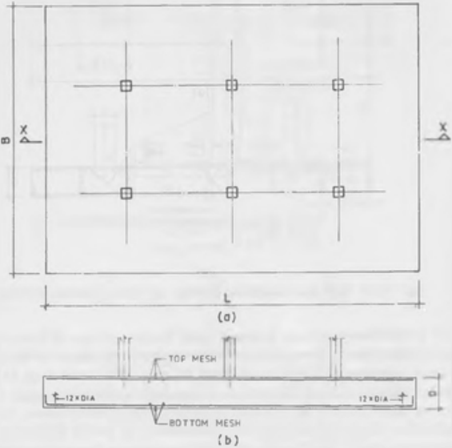


Fig. 13.31. Slab-type raft. (a) Plan of raft footing; (b) Section X-X.

13.6 DEALTING OF SPECIAL STRUCTURES

Shallow domes, as roofs of 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. 13.35). 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. 13.36). 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. 13.37).

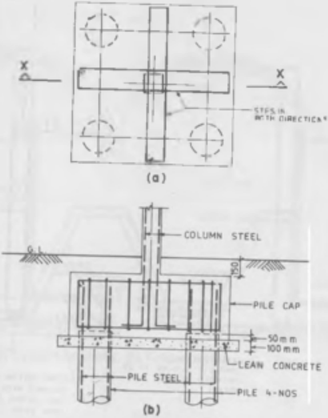


Fig. 13.32. Pile cap detailing. (a) Plan of pile cap and pile group; (b) Section X-X.

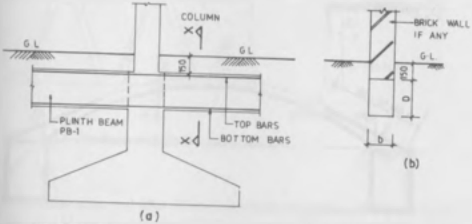


Fig. 13.33 Detailing of plinth beam PB-1. (a) Elevation of plint beam PB-1; (b) Section X-X.

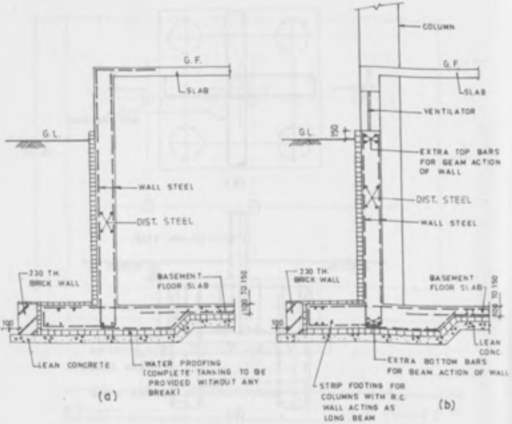


Fig. 13.34 Detailing of basement retaining wall. (a) Basement retainning wall support G.F. slab; (b) Basement retaining wall with columns with ventilator at top.

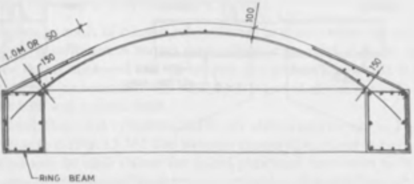


Fig. 13.35 Detailing of domes.

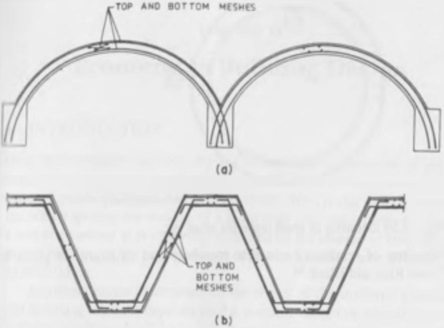


Fig. 13.36. (a) Cylindrical shells; (b) Folded plates.

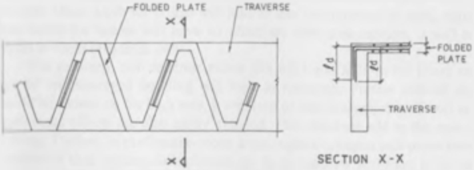


Fig. 13.37. Traverse and folded plate detail.

In circular silos and also in circular shafts supporting overhead water tanks, we should put steel meshes at each face. We have to check for global and local buckling of the shaft walls (Fig. 13.38).

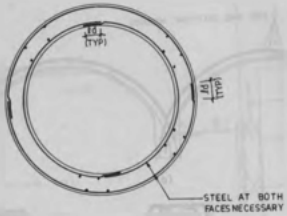


Fig. 13.38 Detailing of shaft walls for silos, etc.

Detailing of reinforced concrete members and structures is given by SP:34[95] and Rice and Black.[96]

CHAPTER 14

Economy in Building Design

14.1 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. 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.

14.2 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 14.1, which varies from 20 kg/m^2 to 80 kg/m^2 of the covered area, depending on the type of the structural system adopted. In Table 14.1,[89] 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.

Expected cement consumption = 0.2 × 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.

Expected steel consumtpion (Table 14.1 S.No.4) = $10{,}000 \times \frac{50}{1000} = 500$ tonnes

Table 14.1. Estimation of steel consumption in buildings.

S No.	Structural system	Method I (kg/m^3 of concrete) slabs	beams	cols	footing	Method II 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
4.	RC framed structure to resist both vertical & hori-						

zontal loads with brick walls to act as filler walls only with plinth beams & shallow column footings.						
(i) N<8 storey	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
7. Flat slab or flat plate structure designed for vertical and horizontal loads with brick filler walls,with shallow footings (upto eight storeys)	200	200	300	200	80	90

Table 14.2. Diameter-wise distribution of steel quantity (tonnes).

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%

Now, for procurement of steel (Fe415, Deformed bars), we need to know the diameter-wise quantities of steel. This is worked out in Table 14.2. 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 14.2 gives the diameter-wise distribution of the total steel 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.

14. 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 compromise on the structural safety of buildings. In general, the break-up of the cost of a building is given as follows:

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%

The average cost of building, at the present time, works out to be Rs.400/ft^2 (Rs.4000/m^2) of the covered area. If the specifications are made on the cheaper side, the cost may go down to Rs. 300/ft^2 of C.A. and if the specifications are rich, it can go upto Rs.600/ft^2 or even more.

CHAPTER 15

Construction and Site Supervision

15.1 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 contruction. 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 slightly. Extra items my crop up in either system of contract. Rate analysis on the basis of market rates is to be given for each extra item and the extra cost of these 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 contractor's work, the architect appoints a clerk of works, who is paid by the client but he remains under the technical guidance of the architect. The clerk of works has to ensure the quality and speed of the work at the site and he is also responsible for co-ordination of the drawings supplied by architect and also by structural, plumbing, electrical and airconditioning engineers. He is also co-responsible for measurements of the works executed at the site and preparation of contractor's 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.

15.2 PERIODIC SITE SUPERVISION BY STRUCTURAL 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 on 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.

a) checking quality of sand, stone aggregates, cement, steel bars, water, bricks, etc.
b) checking soil in the footing trenches
c) 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
d). checking clear covers over steel bars in members. Clear cover is over main bars, but in practice it is wrongly taken over stirrups
e) checking laps of steel bars in members
f) checking provision of chairs and stone or concrete gitties for positioning of top and bottom meshes of steel bars
g) 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.
h) 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
i) checking distances of curtailment of bars in slabs and beams
j) checking anchorage length of top bars in cantilever members and ensur-

ing their safety against overturing, when the shuttering is removed

k) checking the vertical depth of bent-up bars in slabs. It is invariabley short of depth in practice

l) checking overall concrete sizes of members, like slabs, beams, columns, footings, etc.

m) 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 in concrete members

n) noting the remarks of his visit in the site order book for record and for future action to be taken by the contractor

15.3 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 crush 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.[6] This has been done to account for vagaries and variations at the site. Further, the material safety factor for concrete is taken at 1.5, while the same for steel is only 1.15[6] 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 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 year's 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.

A very useful material for study on this subject is given by Camallerie.[97]

CHAPTER 16

Structural Failures or Forensic Engineering

16.1 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 'Forensic 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.

16.2 ANALYSIS AND DESIGN ERRORS

a). In a fertilizer project (in the late fifties), a lime storage underground building was shown in the scope drawings (Fig. 16.1) 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 bars and wall steel were in position. At that stage, the design firm's 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.

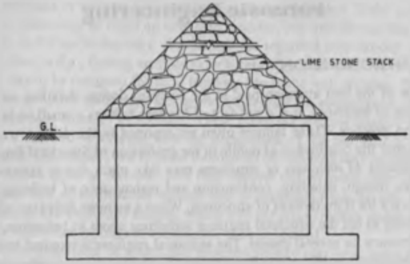


Fig. 16.1 Limestone storage underground structure.

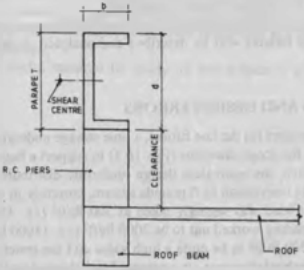


Fig. 16.2 Channel shape of parapet over roof of a cinema building.

b). 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 channel section, in order to save on the weight of the materials and thereby achieve economy (Fig. 16.2). After a few days of the construction, the channel section, with its shear centre outside the section, was seen to be rotating outside and large cracks were formed in the concrete parapet. Had it been a rectangular section ($b \times d$), 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.

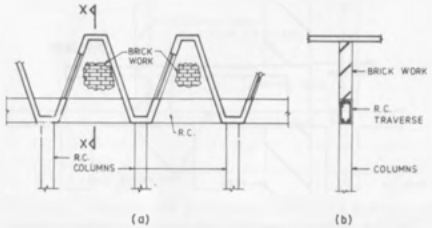


Fig. 16.3. Northlight folded plate failure. (a) Elevation of traverse; (b) Section X-X.

c) A five-plate north-light folded plate over a workshop was built and after sometime, the glass panes in the glazing were reported to be breaking on and off. 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. 16.3). 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 of 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.

d). 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. 16.4). This sort of crack formation cannot be avoided. Rather, a groove can be left at this level to accommodate the separation of 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.

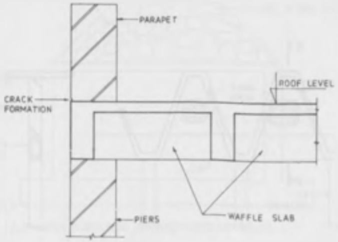


Fig. 16.4. Cracks in supporting brick piers of an auditorium roof.

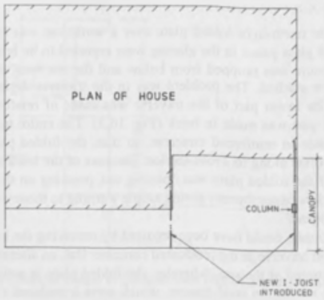


Fig. 16.5. Canopy slab at the roof level of a house.

e) 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. 16.5). 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 situation he was in. We introduced two steel I-Joists as shown in the plan (Fig. 16.5) under the slab, supported on the existing brick walls and these stabilized the cantilever canopy slab. The client had to spend much more than what he had saved by appointing a novice.

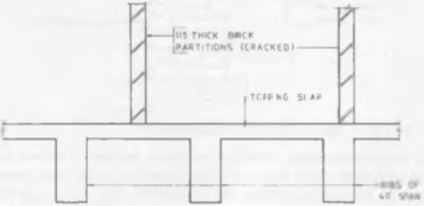


Fig. 16.6. Cracking of brick partitions supported on a long span ribbed slab.

f) 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. 16.6). 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 ribbed 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 sides. Due to this variation in deflections, brick walls get cracked, as these are brittle and cannot absorb these 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.

g) A gabion retaining wall was used in the spillway portion of an earth check dam 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. But to use them as structural 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. 16. 7).

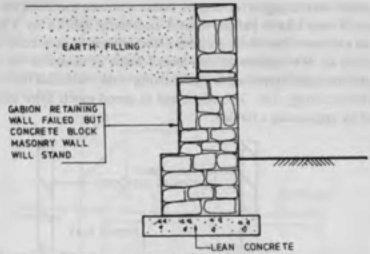


Fig. 16. 7. Failure of a Gabion retaining wall.

h). A workshop building with a flat roof failed suddenly. The roof consisted of red sand stone, slabs, resting on ⊥-steel sections, which were resting on I-joists. The I-joists were resting on RC columns (Fig. 16.8) . 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 ⊥-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.

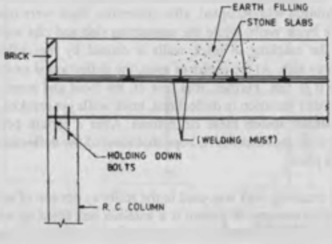


Fig. 16.8. Section of a flat roof on a workshop building.

16. 3 DETAILING ERRORS

(a) In a hostel building, there is a continuous cantilever balcony along the rooms which also turns across at the end of rooms (Fig. 16.9). 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.

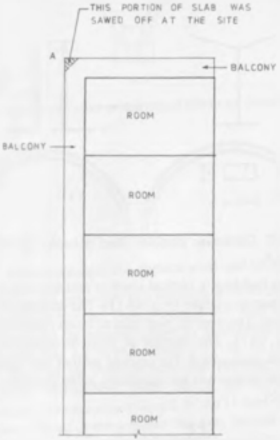


Fig. 16. 9. Sagging of a diagonal corner of a balcony slab.

(b) The columns of a cantilever platform shed were developing horizontal cracks at a fixed height above the ground floor (Fig. 16.10). When the shuttering was removed, these cracks were appearing slowly in all columns at a fixed height. Immediately, the frames were propped at free end of long arms and a study of the design and drawings was made. It was seen that laps in column bars 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 reconcrete 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.

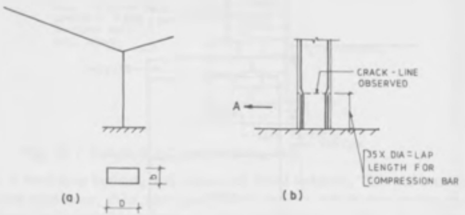


Fig. 16.10. Cantilever platform shed columns. (a) Frame; (b) Column end view A.

(c) In a canteen building, a vertical crack at midspan was observed in a deep beam over the service counter (Fig. 16.11). 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. 16.11. 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]

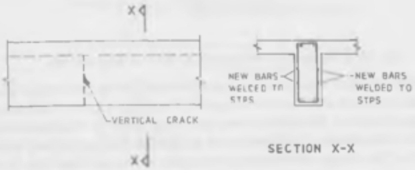


Fig. 16.11. Vertical crack in a deep beam.

(d) In a workshop building, cylindrical shell roof was provided with steel arrangement as shown in Fig. 16.12 and it is taken from Jain and Jaikrishna Vol. II.[5] After completion of the building, extensive thin cracks were observed in the shell roof from the inside. After a thorough study of the analysis and design of the shell roof, it was concluded that transverse steel should have been put on both faces of the shell thickness (Fig. 16.13) all through.

Fig. 16.12. Transverse steel in cylindrical shell roof (incorrect detail).

Fig. 16.13. Transverse steel in cylindrical shell roof (correct detail).

(e) 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. 16.14). The ramp was separated from the adjoining floor by an expansion joint. The span of the cantilever span was 2.35m and section 1-1 (Fig. 16.14) 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 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.

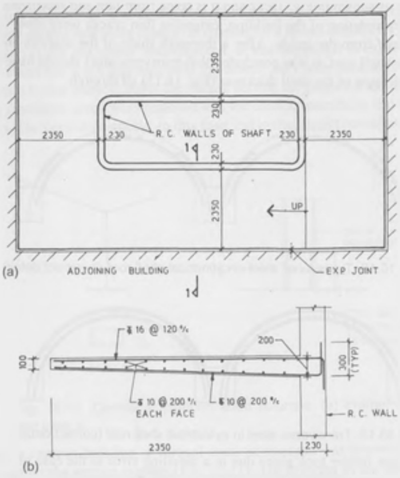


Fig. 16.14. Plan of ramp in a hospital. (a) Plan; (b) Section 1-1.

16. 4 CONSTRUCTION AND MAINTENANCE ERRORS

(a) A petrol pump station structure was a hypar shell structure supported on a single column (Fig. 16.15). This structure was built in accordance with a standard drawing and cracks had developed on roof top as shown in plan of Fig. 16.15. 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 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 incompetence of the contractor and also to the lack of supervision at the site.

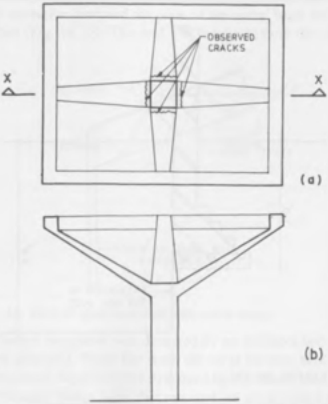


Fig. 16.15. Petrol pump station structure. (a) Top plan of the hypar shell structure; (b) Section X-X.

(b) 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. 16.16). For protecting the 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.

(c) A retaining wall, which was supporting a high ground with a swimming pool, was observed to be tilting outwards (Fig. 16.17). 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.

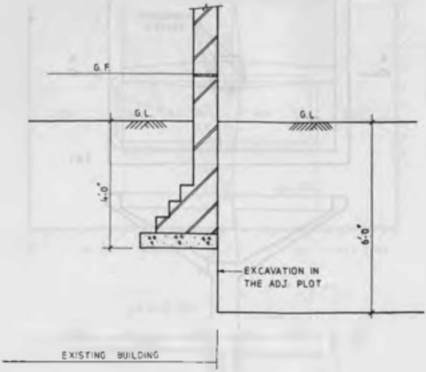


Fig. 16.16. Failure of an wall of an existing bungalow.

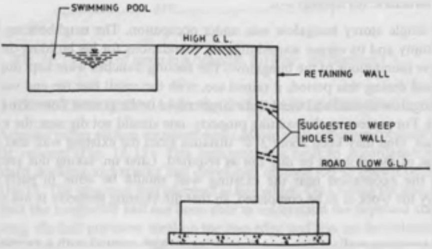


Fig. 16. 17. Retaining wall failure.

(d) 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 proved to be excessive load and the joint of the radial truss with the vertical steel wall failed (Fig. 16. 18). The roof was redone without the cement plaster.

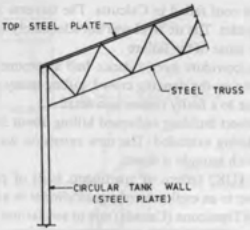


Fig. 16.18. Joint of steel tank wall with radial truss.

(e) 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 all beam locations. The slab was cast and when the shuttering was 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 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.

(f) A reinforced concrete framed factory was being extended upwards. The beams developed cracks as these were not propped from below, when the upper 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 caused by laxity on the part of construction staff.

(g) 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.

16. 5 SOME WELL-KNOWN STRUCTURAL FAILURES

The following structural failures were reported in newspapers or other technical literature.

a) A barrel shell roof failed in Calcutta. The traverse was of the type of a bow-string girder. The tie steel was not adequately anchored at the ends. This was the cause of the failure .
b) 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.
c) 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.
d) Ronan-Point (UK) failure of apartments built of precast concrete elements was due to an explosion of a gas cylinder in a kitchen.
e) Silos tilted in Transcona (Canada) due to soil failure.
f) 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.
g) Three-storeyed children's ward in a Jammu hospital collapsed resulting in casualties. Reason? Inadequate amounts of cement and steel used.
h) 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!
i) Cantilever balconies often fail killing people due to overloading, wrong design or detailing or both.

An excellent book on this subject is by Raikar,[98] in which, a wealth of detail on this subject is available.

16. 6 ADDITIONAL CASE STUDIES OF FAILURES

The following case studies of some failure have been contributed by J.D. Buch:

a) 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. 16.19). 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.

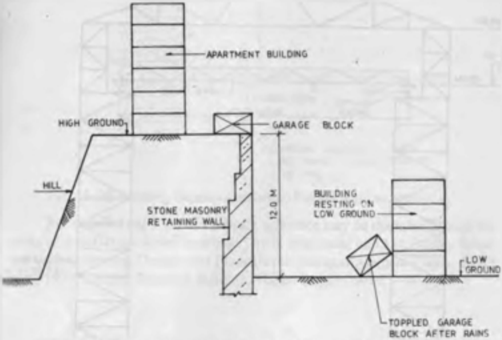


Fig. 16.19. Failure of a garage block in a hill area in Bombay.

b) 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, 15.0 m span collapsed. It was noticed that concrete in the bottom layer of 15.0 m 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 of 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.

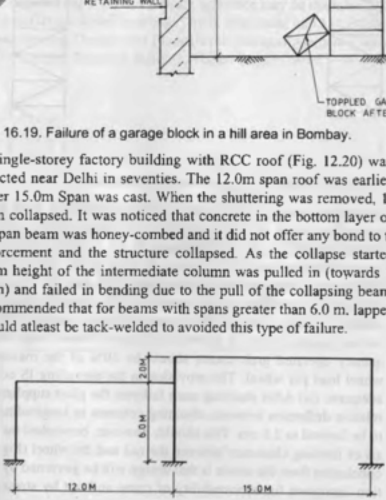


Fig. 16.20. A section through a single-storey factory building.

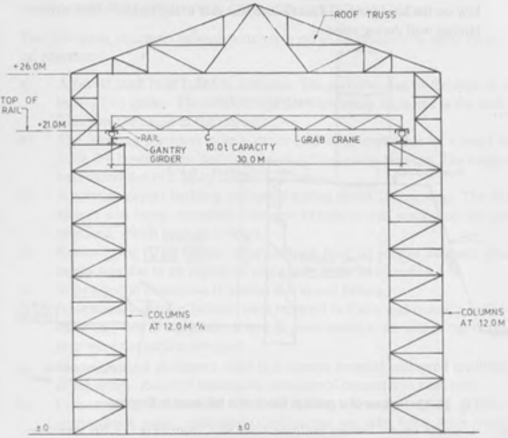


Fig. 16.21. A section through the stock yard building.

c) 30.0m span storage hall (or stock yard) building used to be provided in the earlier plants with a provision of 10.0tonne lifting capacity grab crane (Fig. 16.21). 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 frequency operated grab cranes should be 10% of the maximum static wheel load per wheel. The provision in the prevailing IS code was inadequate. (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. 16.22). A conclusion from the above is that design will be governed by the deflection limitation for serviceability of crane and not by stress considerations.

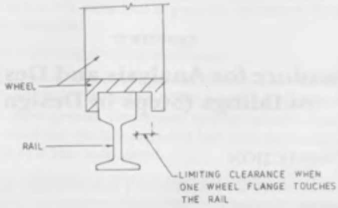


Fig. 16. 22. Limiting clearance between the rail and the wheel

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.

CHAPTER 17

Procedure for Analysis and Design of Buildings (Steps in Design)

17.1 INTRODUCTION

The procedure for analysis and design of a given building will depend on the type of building, its complexity, the number of storeys, etc. First, the architectural drawings of 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 drawings 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.

17.2 DESIGN OFFICE PRACTICE FOR A TALL OFFICE OR APARTMENT BUILDING

The following procedure or steps in design may be followed for a multistoreyed office or apartment building:

Step 1: *General Loadings*. 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 of 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 permitted by IS:875. 5% extra load is generally considered for unforeseen items like elastic shears etc.

Step 3: *Analysis 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 of 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 of the area, the soil data and the flexibility of the building, etc. into account. If the base shear due to earthquake is greater than the base shear due to wind (calculated in step 3a), then the earthquake base shear along the height of the building is calculated in accordance 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 loads 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 moments to the face of the members, as against their values at the centre-line 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.

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 of 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 foundations.* 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 of different types of foundations in order to preference with a view to economy: (i) Individual footings with or without seismic ties; (ii) Combination of individual and combined footings with or without ties; (iii) Strip footings with retaining wall acting as strip beam wherever applicable; (iv) Raft foundations of the types (a) *slab* (b) *beam-slab* and (c) *cellular raft*, i.e. top and bottom slabs with deep beams in both directions with hollow spaces in-between; and (d) *annular raft*, i.e. strips in both directions with cut-outs 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 of 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 designed at this stage, being considered as part of foundations.

At this stage, the draftsman takes up the work of preparing drawings of the foundations and the column schedule of the building. These drawings can then be issued to the site for execution.

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 is checked to get slab thickness as less as practicable. The beams are designed as continuous beams, monolithic with reinforced concrete columns with their far ends assumed fixed (substitute frames). The variation in the live load position is 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 of 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 a designer.

The drawings are prepared floor-wise, starting from the ground floor (i.e. basement roof) and upwards and issued to the site for execution in the same order.

17. 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 it is assumed that no one floor is identical with the other . The following steps in design may then be followed:

Step 1: Design of roof slab and beams for vertical loads.
Steps 2: Design of 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 of the building for horizontal loads

Step 5: Design of 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 of 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.

17. 4 DESIGN OFFICE PRACTICE FOR TALL BUILDINGS WITH SHEAR WALLS

The followings steps in design may be followed:

Step 1: General loadings.
Step 2: Computation of column and shear wall loads by the tributary area method
Step 3: Analysis of 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% of the horizontal shear.
ii) For buildings more than 20 storeys, interaction of shear walls and 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 of 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] For walls with openings, similar procedure is followed, taking the effect of overturning and local bending of piers into account and the reinforcement is calculated to cover tensile or excessive compressive stresses as given by the stress diagram.[16] For the beam effect over openings, extra reinforcement is to be provided which is calculated for the local bending moment and checked for shear.[57]

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 and issued to the site in the same order.

17.5 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 of approach 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 of all frames in both the principal directions. If we consider six degrees of freedom at each joint, then the rotational effect of the earthquake shear, if any, will be considered automatically.[37]
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 of plinth beams.
6. Manual design of footings.
7. Preparation of structural drawings with foundations and upwards by AUTOCAD.

Merits: (i) Maximum work done by Computer. (ii) We get accurate column loads These are total loads without live load reduction.

Demerits: (i) Data preparation takes more time. (ii) computer has to be free for a long time to take the in-put. (iii) Live load reduction in column loads has to 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 triangular 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.
5. Column design for VL and HL.
6. Checking grid beams for HL effect.
7. Design of plinth beams manually.
8. Design of 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 buildings*

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:

(i) Computer use is minimum and easy.
(ii) Data preparation for computer work is quick.
(iii) This method gives results quickly.
(iv) Live load reduction in column loads is considered easily and it is a part 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 buildings* (8 to 12 storeys)

This is given as an alternate method to (b).

Steps in design

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 computer's 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.
(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 draftsman for the drawing work to start.

17.6 USE OF COMPUTERS IN STRUCTURAL DESIGN OF BUILDINGS

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 and much more. When similar members (say, slabs, beams columns, footings) are to be designed, relevant computer programs can be used to get the results correctly and speedily. Further in analysis, computers help us to use more accurate 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 had to be satisfied with them. But now, computer can be used effectively to get accurate results. Likewise, difficult structural problems of analysis and design can be solved by using appropriate computer programs.

17.6.1 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 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 acceptable 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 of 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 of the specialised computer programs available with them.

17.6.2 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 of members by which he can get 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 of structural drawings which will contain the results of the design of the building.

CHAPTER 18

Additional Aspects of Structural Design

18.1 WHAT IS A GOOD STRUCTURAL DESIGN[101]

With a boom in the construction industry, many structures are coming up all over the country. Many structural engineers are active in the area of structural design and it is the aim of all designers to achieve a good structural design. Now the question that needs to be answered is – "what constitutes a good structural design?"

18.1.1 Structural Systems for Safety

Safety is the prime concern of structural design. A building has to be designed for all the loads – vertical as well as horizontal – which are likely to act on a building during its expected life-time. All these loads are given in the relevant IS codes, which are mandatory to be followed in structural design. Structurally speaking, buildings are built to support loads. The primary loads are vertical or gravity loads consisting of dead and live loads. Dead loads consist of self weight of members, walls, finishes, plaster, etc. act on the strcuture all the time. Live loads depend on the type of activity to be carried out at floors of the building. These loads may act from 0% to 100% of their values and these may shift their positions in plan too. Also these loads may act on the entire floor or a part of the floor area. Dead and live loads added together give vertical or gravity loads.

Horizontal loads are caused by the actions of wind, earthquake, temperature variations and shrinkage (and also blast in certain areas). IS codes are available to give an idea of the quantity and the quality of these forces which must be considered in the structural design of buildings. There is a certain vital difference between the natures of the horizontal loads and the vertical loads. While the vertical loads act on a structure for all the time, the horizontal loads with their maximum design values act only for a short while. Due to this difference, the allowable stresses in materials are increased by 33.33% in the Working Stress Method of Design (i.e. $f_a = 4/3 f$) and the partial load factors for safety are decreased by 25% in the Limit State Method of Design (i.e. γ_s =

$\frac{3}{4}\gamma$). Both these approaches virtually mean the same thing and consider the transitory nature of the horizontal loads.

Vertical or gravity loads originate at floor levels and the floor system in horizontal planes should be adequate in strength to transfer these loads to the vertical elements like columns and walls (brick or concrete) which transfer them to the supporting soil by means of footings. Floor systems consist of slab panels (one-way or two-way) and beams in one or both directions. Flat slab or flat plate floor systems are also efficient in supporting vertical loads. For large spans, one-way ribbed slab panels or two-way ribbed slab panels (called grids) are also used, the shuttering item in these systems being costly, needs attention. Floor systems should be so chosen as to be efficient in transferring vertical loads to the supporting columns or walls or both, as the case may be Columns and walls come under compression due to vertical loads. Concrete and brick are strong in compression and suitable sizes of these elements can be worked out, taking into account slenderness effects and minimum eccentricity moments or the frame moments under vertical loads which ever is greater.

Horizontal loads need different structural systems to transfer these loads to the foundations. In small buildings like houses, bungalows, etc. brick walls may be designed to resist horizontal loads by way of box-action of walls in plan. In multi-storeyed buildings, say more than four storeys, reinforced concrete beams and columns, cast monolithically, constitute what are called frames. The frames under horizontal loads undergo shear mode of deflection and these, provided in the two principal directions, are effective in transferring the horizontal loads to the foundations. In tall buildings, the beam and column sizes workout, quite large with congestion of reinforcement at beam-column joints, then it becomes necessary to introduce shear walls which should be as deep as possible and should support as much vertical load as possible, in order to balance the effects of horizontal loads. A shear wall acts as a column under vertical loads. It also acts as a vertical cantilever beam, fixed at its foundation under horizontal loads. A shear wall is, thus, to be designed for vertical compression, horizontal shear and an overturning moment. A shear wall structure has a bending mode of deflection. Shear walls, in general, are very much more stiff than frames in buildings of 10 to 20 storeys, in which case, full horizontal load is resisted by shear walls and the frames are to be designed for vertical loads together with a minimum of 25% of the horizontal load (LF = 1.2), which may not affect the design of frames under vertical loads (LF = 1.5). In taller buildings, shear walls and frames interact together and resist the incumbent horizontal loads together, assuming an intermediate mode of deflection, in-between the shear mode of deflection of frames and the bending mode of deflection of shear walls.

The diaphragm action of floor systems is the basic requirement for distributing horizontal loads to the vertical resisting elements like frames and shear walls. The floor diaphragms are regarded as infinitely stiff in their own planes, due to their large dimensions in plan. This requirement is easily satisfied by even 100 mm thick floor slab, the beams provided, serving as stiffeners to the slab diaphragm.

Horizontal loads like wind and earthquake are erratic in magnitude and direction. It is, therefore, required to design buildings in the two principal directions, separately, under horizontal loads. Further, it is assumed that the worst effects of wind, earthquake and temperature variations do not act together during the lifetime of a structure. This assumption is made in order to keep the cost of the structure within reasonable limits. In shear wall structures too, the arrangements of shear walls in plan has to be so chosen as to be effective in both the principal directions. Shear cores or boxes are elements which are stiff in either principal direction. Long shear wals are stiff in their length-direction only, these being negligible in their thickness-direction. In both the transverse and the longitudinal directions of a shear wall building, adequate arrangements of shear walls have to be privided to resist horizontal loads in each direction.

A symmetrical arrangement of frames and/or shear walls is preferable to an unsymmetrical arrangement. In symmetrical arrangement, floor diaphragms only translate, while in unsymmetrical arrangements, diaphragms translate as well as rotate in plan, causing variation in horizontal loads allocated to frames and shear walls. Unsymmetrical arrangements are costly and are difficult and time-consuming to analyse.

In summary, it can be said that there are basically three structural systems to resist horizontal loads in buildings:

i) Brick walls by way of boxes in plan.
ii) Reinforced concrete frames in the two principal directions.
iii) Reinforced concrete shear walls so arranged in plan as to be effective in both the principal directions.

In practice, it is not easy to decide which system is appropriate in a given building.

Houses, bungalows, flats upto four storeys are built with load bearing brick walls, which resist both the vertical and horizontal loads. In places, where a brick wall may fail reinforced concrete columns are provided whithin the brick to support vertical loads only, the horizontal load being resisted by brick walls by way of box-action in plan. This gives, what may be called a mixed system of brick walls and reinforced concrete columns. In certain situations, reinforced concrete frames are designed for vertical loads only and the horizontal loads are given to brick walls which exist, anyhow, as filler walls.

This gives rise to what may be called a twin-system, one for the vertical loads and the other for the horizontal loads. The twin-system may be adequate for buildings upto four storeys, with adequate number of brick walls. For buildings taller than four storeys and with less number of brick walls, reinforced concrete frames in both the principal directions are to be provided. This system may be adequate for 8 to 12 storeys and it will depend on the intensity of earthquake or wind forces in the area. But in areas of blast activity, reinforced concrete frames are to be provided for even houses. Further, for buildings with large storey heights like electric sub-stations, large-span halls of general gatherings, etc. reinforced concrete frames have to be provided for safety, although these buildings are, in general, of only one storey in height.

With buildings more than 10 to 12 storeys in height, sizes of reinforced concrete frame members (i.e. beams and columns) work out quite large and the beam-column joints get congested with reinforcement, in which case, it is advisable, for reasons of safety, to provide shear walls in buildings. In such cases, horizontal loads are resisted entirely by shear walls, while vertical loads will be resisted by both the shear walls and columns of the available frames, which will be anyhow designed for 25% of the horizontal load also. This system works well for buildings upto 20 storeys in height. For taller buildings, shear cores in the interior with or without a framed core on the periphery or multicored systems have to be used for resisting horizontal loads. These are a few general rules about the efficiency of shear wall systems. But, in practice, shear walls may be provided in short buildings where the column spacing is large or in buildings with flat slab and flat plate floors where the frames thus formed, are adequate in resisting only vertical loads but the same are weak in resisting horizontal loads. Hollow box columns of large dimensions may well be designed as shear walls for resisting both the vertical and horizontal loads.

In shear wall buildings, the foundations are likely to be heavy in the consumption of materials like concrete and steel while the floors will work out light. In framed structures, the opposite may come out to be true, i.e. floors will consume more materials, while the foundations will work out comparatively light.

18.1.2 Considerations for Serviceability

Serviceability is also one of the concerns of structural design. It refers to the durability and appearance of structures. Deflection and cracking of bending members – beams and slabs – come under the serviceability criteria. A bending member with a large deflection will look ugly and unsafe to the view. But it can be easily taken care of by providing a suitable camber to the form work of the member before laying concrete. Further, excessive partial deflection in members, after the filler partitions have been installed, cause cracking in such partitions as these are made of bricks or such brittle materials. In order to control cracking in partitions and finishes, deflection of bending members need to be restricted. But deflection of bending members is a natural phenomenon and it cannot be avioded. Likewise, cracking of concrete in tension zone of bending members is also a natural phenomenon. It has to be restricted by providing small diameter bars at close spacing thereby restricting crack widths. Also a minimum specified amount of reinforcement in slabs and beams is necessary to provide for controlling cracking, and this adds to the durability of buildings.

For durability of concrete structures, the code specifies a certain minimum cement content and water-cement ratio in concrete mixes. The code IS 456 proposes to provide in structures a minimum concrete mix of M20 (1:1 1/2:3), in place of M15 (1:2:4), in vogue presently. This is being done for enhancing durability of structures. In coastal areas where the climate promotes corrosion of steel reinforcement, a suitable coating of steel bars before placement, is being specified in practice. An adequate cover to steel bars in concrete members is also important for achieving a measure of durability and fire resistance.

It is to be noted that serviceability requirements are not as catastrophic for a structure as the safety requirements. These requirements are of a qualitative nature and these may prove crucial only in the long run.

18.1.3 Considerations for Foundation Design

Foundations of a structure have to be designed properly for ensuring safety of the superstructure. Foundations spread the load of the superstructure to the supporting soil below. The safe bearing capacity of the soil and the depth of the foundation below ground level are the data required for the structural design of foundations. These data are supplied by the soil engineers, who also give recommendations in respect of foundation-system suitable to the building.

The depth of foundation depends not only on the nature of soil strata but also on the height of building. For low buildings (less than four storeys), a depth of foundation of 1.0 m to 1.5 m may suffice. For taller buildings (6 to 12 storeys) 2.0 m to 3.0 m foudation depth will be adequate. For still taller buildings, shallow foundations may not be suitable, and pile foundations may have to be provided. The above values of the foundation depth have only intuition as the basis.

The most economical and adequate, foundation system for a building consists of isolated footings with plinth beams in both the principal directions acting as ties. In this system the forces at the base of columns or walls are led to the ground in the shortest possible way. The other systems like combined and strip footings or raft foundations, spread the loads on large areas and then, these are led to the ground, thereby incurring more consumption of materials. Pile foundations are similar to isolated footings with the only change that soil reaction is uniform under isolated footings, while it consists of concentrated pile loads from below acting on the pile cap.

In foundations, the problem of settlement of footings always creates a controversy. In general, footings, cast at one given level with a given soil bearing capacity should settle all uniformly. But as the column loads may vary due to different live load dispositions or values, there may be a chance of an unequal settlement of footings. In practice, live load may be of the order of 10% to 20% of total column load, most of the settlement (i.e. 80%) on account of dead loads, takes place uniformly. The live load variation in columns will cause, therefore, only a negligible amount of differential settlement. IS codes permit 50 mm settlement for isolated footings, 75 mm for raft foundations and 12 mm settlement for pile foundations. In order to restrict differential settlement of footings, all footings shall be tied at the footings base level or at the plinth beam level whichever is convenient. For tall isolated structures like water towers, it is advisable to provide a soild or an annular raft foundation, in order to ensure uniform settlement of these structures.

18.1.4 Considerations for Economy

The aim of structural design is to achieve economy in the consumption of structural materials like concrete and steel, in keeping with the requirements of safety and serviceability. The following measures are taken in practice to achieve economy in design:

a) Floor slab thickness is to be kept minimum, near about 100 mm. to 150 mm. Dead load due to slab constitutes a major part of the dead load of the building. By reducing slab thickness, design of beams, columns and footings will work out economical. Further, earthquake forces acting on building will also work out less, adding to economy.
b) Beam depths should be selected on the high side. This reduces steel consumption in beams. The width of beams should be kept on the low side at about 230 mm to 300 mm. Large beam width adds to the dead load of beams, without reducing tension steel, only helping in the shear design and in the placement of bars in one layer.
c) Column sizes should be selected on the high side. This helps in the reduction of steel reinforcement and also it controls slenderness effects. For efficient earthquake resistance, square columns are preferable to rectangular columns. In apartment buildings, one side of columns is kept the same as the wall thickness (230 mm), this adds to the cost of columns. Further use of richer mixes of concrete and age factors for

concrete strengths in lower storeys lead to economy.

d) Spacing of columns depends on the function of a building. Although, it is known that close spacing of columns leads to economy, but then the function or the purpose of the building may be defeated. The present trend is to sacrifice economy for the sake of functional utility of buildings.

e) In foundations, the concrete sizes of beams and slabs are taken on the high side. These results in less consumption of steel and also it helps provide fixity at the base of columns and/or shear walls which is the normal assumption in anlysis. Further, the dead load of the foundation resting on the soil directly, produces no additional forces in members, as it happnes in the design of super-structure.

f) Uniform sizes of beams and columns are preferable to achieve economy in the cost of shuttering. Beam layout at floor and roof levels shall be kept identical for ease in shuttering.

g) Laps in column bars consume a lot of steel. Measures should be taken to provide laps at every two storeys instead of the present practice of laps at every one storey.

h) In multi-storeyed buildings, self-weight of partition walls is one of the causes which make buildings heavy. 115 mm thick brick partition walls are quite heavy. These should be replaced by gypsum board partitions or light weight concrete block walls. This will cause a substantial reduction in the dead load of partitions, resulting in economy.

18.1.5 Considerations for a Good Structural Design

A good structural design must ensure above all safety. There has to be no compromise on the safety of buildings. Serviceability criteria shall also be satisfied but these are not as crucial as the safety criteria. A suitable structural system for resisting vertical loads and the same for resisting horizontal loads are the requirements of safety. The frame spacing in keeping with the function of the building has to be chosen. A suitable choice of a structural system for resisting both the vertical and horizontal load is the basic requirement of safety. Economy is the test of good structural design. Given two designs, which are satisfactory in respect of safety and serviceability, the more economical of the two, will be adopted for construction which indicates the efficiency of structural design. Economy of design may be measured by the rate of steel consumption in buildings which is given as the steel quantity consumed divided by the covered area of building (in kg/m^2). The values of steel consumption index given in Table 18.1 may be taken as a guide for various structural systems adopted in buildings.

The steel consumption sharply increases for hospitals or similar buildings for inclusion of one or more storeys in further provisions. Also steel consumption is more in flat slab or flat plate structures. The above values of steel consumption index are valid only for Delhi region, elsewhere, these values may change depending on the intensity of earthquake or wind loads prevailing in the area. Further, a good structural design should have the virtue of constructibility, i.e. it should be capable of being executed easily at the site.

Table 18.1. Values of steel consumption index.

Sl.No	Type of buildings	Steel consumption Index (kg/m^2)
1.	Houses with reinforced concrete slabs on load bearing bricks walls	20
2.	Large houses with mixed system of RC columns and load bearing brick walls	30
3.	4 storeyed apartment buildings with twin system of RC frame for vertical loads and filler bricks walls for horizontal loads	40
4.	RC framed buildings with 8 storeys or less	60
5.	RC framed buildings with 12 storeys or less	70
6.	Shear walled buildings with 20 storeys or less	80

18.1.6 Aberrations in Structural Design

In practice, many aberations in structural design have been observed. Some designers may design only for vertical loads, without taking any care of horizontal loads. For horizontal loads, some designers consider only one frame in each principal direction and superimpose these results on all other frames, without considering the building as a whole, which is the way a building behaves under horizontal loads. Foundation systems may also be faulty in respect of footings being laid at different levels and with no adequate footing ties provided or pile and shallow footing systems may get mixed up. In a building, it was observed that frames were provided in the transverse direction only, there being no beams in the longitudinal direction, thereby making the building weak for horizontal loads in the longitudinal direction. Although, codes are the same for all designers, the values of loads, etc. taken, may vary Live load on stairs in public buildings is 500 kg/m^2, but in a certain design it was taken at 300 kg/m^2. Like wise, interpretation of codes may vary from designer to designer. Wind pressure on a building was worked out in a case to be 440 kg per m^2 on taking the worst values of coefficients k_1, k_2, k_3, etc. On rethinking, this value was brought down to 220 kg/m^2 which was reasonable in the case under consideration. Further, some designers may be strong in analysis, i.e. finding forces in members by manual or computer methods, but they may be weak in design, i.e. finding concrete sizes and steel reinforcement of members. Some other designers may be good at detailing techniques. For perfection in design, one has to be good at analysis, design and detailing of structures all at the same time. Pursuit of prefection in structural design is a relentless struggle, as it is so, in all other spheres of life.

18.1.7 Failure of Structures

It is a sad fact of life that structures sometimes fail in spite of the best efforts of structural engineers. This must not happen but reports of structural failures do appear in the press on and off. It shatters the confidence of public in the profession of Structural Engineering. What may be the causes of failure of structures Some of these may be listed below:

a) Lack of experience of structural designers, non-qualified persons like site supervisors or even clients working as structural engineers.
b) Greed of clients by adding storeys without prior provision in the design.
c) Expert designers making serious errors as the best drivers may get involved in accidents, non-application of mind by structural engineers.
d) Construction defects and lack of supervision.
e) Failure due to soil giving way.

In order to prevent failure of structures, the recent trend is to get the structural design checked by another structural engineer, who may be called a checking engineer. This practice is to be welcomed and it is followed by law in some European countries. Each engineer, being a prisoner of his own background, has his own perception and his own understanding of structures. As two doctors do not agree, two structural engineers too may not agree. This often happens in practice.

The structural engineer faces competition in the field from experienced mistries (or work supervisors) or even clients who have already got built a couple of buildings. Further, he faces criticism from checking engineers, who though his colleagues in the same profession but may have different backgrounds like teaching, construction, estimating, etc. It is easy to convince oneself but it is very difficult to convince others, for which a lot more scholarship is required.

18.2 HOW TO REDUCE STEEL CONSUMPTION IN REINFORCED CONCRETE BUILDINGS[102]

It is vital for economy to reduce steel consumption in reinforced concrete buildings. Steel bars are required to be put in reinforced concrete members at the locations where tension occurs and also in members like columns to resist compression along with concrete which is strong in compression but weak in tension. The provision of steel in members is the requirement of safety. For serviceability requirement also, a minimum amount of steel in members shall be provided at close spacing in order to control defelction and cracking of members. For the way we design buildings, the steel consumption ratios are given in Table 18.2 which depend on the height of buildings and also on the type of structural system adopted in buildings. The values given in Table 18.2 are valid for buildings in Delhi region, where the seismic coefficient is 0.05 and design wind pressure is 150 kg/m^2. Elsewhere the values given in Table 18.2 will vary depending on the intensity of earthquake and wind forces prevalent in the area.

Table 18.2. Steel consumption ratios for reinforced concrete buildings in Delhi region.

S.No.	Type of buildings	Steel consumption ratio (kg/m^2)	psf
1.	Houses with reinforced concrete slabs on load bearing brick walls	10	2
2.	Large houses with mixed system of reinforced concrete columns and load bearing brick walls (2½ storeys)	20	4
3.	Reinforced concrete framed buildings with		
	$N = 4$	40	8
	$N = 6$	50	10
	$N = 8$	60	12
	$N = 12$	70	14
4.	Shear walled buildings with $N \leq 20$	80	16

N = Number of storeys

$$\text{Steel consumption ratio} = \frac{\text{Total steel quantity in building in kg}}{\text{Total covered area of building in m}^2}$$

Of the total steel quantity in a building, about 35% of its goes in slabs, 20% in beams, 25% in columns amd the rest 20% in foundations. Of the total concrete consumed in a building about 50% is consumed in slabs, 15% in beams, 10% in columns and the rest 25% in foundations. Steel consumption in members like slabs, beams, columns and footings based on kg of steel/m^3 of concrete used is given in Table 18.3. It is seen that beams and columns are heavily reinforced members, while slabs and footings are comparatively lightly reinforced members. Concrete slab thickness is kept on the low side in order to reduce dead load of buildings, while concrete members in foundation are kept on the high side in order to reduce steel consuption and also in order to make the footings safe in shear. The increase in the dead load of footings has no backlash as footing dead load is directly countered by the supporting soil, producing no additional forces in footing members.

Table 18.3. Steel consumption ratios in RC members

S No.	RC member	Steel consumption (kg/m^3 of concrete)
1.	Slabs	50 to 100
2.	Beams	150 to 200
3.	Columns	200 to 300
4.	Isolated footings	50
5.	Combined/strip footings	100
6.	Rafts	150

In the following sections, we locate the reasons for increase in steel consumption in buildings and give suggestions in respect of measures to be taken in the analysis and the design of reinforced concrete members to achieve the object of reducing steel consumption in buildings.

18.2.1 Analysis of Structures

Computer programs currently in use in design offices like STAAD III, etc. for the analysis of frames are based on stiffness matrix method considering skeletal members having finite dimension of beams and columns. In these programs, no effect of floor or roof slab is considered. The moment of inertia of a beam with floor slab is considerably greater than that of the corresponding rectangular section. By omitting the influence of slab on the moment of inertia of beams, the analysis gives larger column moments which are not realistic and are not in keeping with the behaviour of reinforced concrete buildings. Column design is very sensitive in respect of column moments. So, if we can reduce moments in columns, we can achieve good reduction in steel quantity in columns which are very heavily reinforced members. In earlier manual methods, moment of inertia (MI) of a T-beam was taken at twice the MI of the corresponding rectangular beam, while for an L-beam, this factor was taken at 1.5. Further, Table IX of IS: 456-1964 (clause 11.3.1) was used to get column moments. This table is very useful and it is based on slope deflection method and it leads to reasonably correct column moments. It should be borne in mind that mere use of computers does not lead to correct analysis. The programs in use must be corrected to include the effect of slab on the moment of inertia of beam members.

18.2.2 Evaluation of Column Loads

Column loads in buildings are evaluated either by tributary area method (confirmed by Fintel, p.318) or by summation of beam reactions at floor levels. The tributary area method leads to reasonably correct column loads, while the method of beam reactions gives correct column loads if the slab loading on beams is considered on triangular or trapezoidal distribution as given in the code IS:456-2000 (clause 24.5). But many designers use expressions for slab loading like

$$\frac{wl_x}{3} \text{ or } \frac{wl_x}{6}\left[3-\left(\frac{l_x}{l_y}\right)^2\right]$$

which are based on equivalence of midspan moment of a simply supproted span. By this method, the beam moments work out nearly correct, while the beam shears work out on the high side, thereby giving column loads on the high side. This affects column design giving more steel in columns and also resulting in large footings. In this situation, it is suggested to reduce by 15% all column loads calculated on the above basis. This is applicable to two way slab systems only.

In earlier manual methods, equivalent shear loadings were used for beam shears and column loads. But in the present-day practice, computer programs are being extensively used in design offices. So, it is recommended to use triangular or trapezoidal distribution of slab loading or if moment equivalence slab loading is used, then a reduction of 15% should be applied on all column loads. This will give us nearly correct column loads.

18.2.3 Beam Design at Support Sections

A versatile computer program for analysis and design of a continuous floor beam with columns regarded as fixed at the top and the bottom floor levels is being extensivley used in design offices. It does not consider the effect of slab on the moment of inertia of beams. This leads to more moments in columns than what happens in reality. Further, in this program, support steel is calculated for support moments given at the centre-line of columns. The correct position is to calculate support steel for the beam moment at column face. This makes quite a difference in the value of design support moments which will lead to reduction in beam steel over supports. The computer program should be corrected in the above two aspects. Alternatively, face correction to support beam moments may be applied either by manual calculations or by using the relation given by a CAI publication (Applications of Moment Distribution, p.97).

Face correction = $V.a/3$

where V = support shear for an equivalent simple supported span

a = width of column support

It is seen that the support moment gets reduced by about 15% to 25% of its theoretical value, leading to a saving of steel over beam supports.

It is to be appreciated that in the computer program of the above reference, design shear force is rightly calculated at the critical section which is an effective depth away from the column face as given by the code. But the programmer omitted to do the same for the face correction for support moments, which is a vital aspect for reducing steel consumption in beams, which are also heavily reinforced members. Further, in this program, slab width and thickness are called for in the data. But this information is not utilized in the calculation of MI of beams, but it is rather utilized only for design of midspan section of beam as a T-section. For design of reinforcement, midspan section of a beam acts as a T-section and its support section acts as a rectangular section. But for analysis, the entire beam length should be regarded as of unifom T-section (vide Jain and Jaikrishna Vol. II, p.2).

18.3 SELECTIVE RE-INTRODUCTION OF MILD STEEL REBARS[103]

In general, reinforced concrete (RC) construction all over the world is over 100 years old. It is generally observed that old structures using mild steel bars with 1:2:4 concrete have performed well with minimal crack, indicating thereby that steel bars have not corroded for such a long time. However, structures which are recently consructed, about 30 years ago, are showing signs of distress and many members including columns, beams, slabs and chajjas have developed cracks indicating reinforcement corrosion. It may be noted that in the past 30 years or so, Fe415 steel deformed bars are being used in this country in conjuction with 1:2:4 concrete (M15). This combination of material has not worked in favour of durability. High strength bars develop high stresses leading to larger cracks in tension zones of concrete members, allowing more ingress of atmospheric gases in the concrete which leads to corrosion of steel bars. Prior to the use of high strength steels, structures have behaved well in respect of durability. Now, it is proposed to use a minimum concrete mix of grade M20 (1:11/2:3) in conjuction with Fe415 deformed bars and it is hoped that the structural members will remain crack-free, leading to better durability.

In this connection, it is interesting to refer to a feature, Iravian Temple

Foundation Pour, Kauai, Hawali, published in the December 1999 issue of The Indian Concrete Journal, which uses plain concrete raft foundation having a depth of 1.2 m for a Hindu Temple in Hawaii, USA. As no steel bars are used, there is no fear of any corrosion of steel and so no cracking of concrete will take place ensuring an expected life of a thousand years.

Further, there is a proposal to increase concrete covers of the RC members depending on the severity of zone (that is the region of location of building). In the present practice, concrete cover depends on the type of member and also on the severity of zone. This is a good practice and we should have different covers for slabs, beams, columns and floorings and these basic cover values may be increased by 5 mm to 25 mm, as the zone severity increases. For example, we cannot have more cover for thin members like slabs. It should also be noted that more cover leads to more crack-width, which is not conducive to durability.

The caluse 7 of IS:456-1978 requires an adequate cement content and low water-cement ratio for durability of concrete structures. Also appendix A of IS:456-1978 provides guidance regarding minimum cement content and permissible limits of chloride and sulphate in concrete. It is understood that this code is being revised with improved provisions of durability.

The member size also plays a part in ensuring durability. Thin members which are exposed to weather do not last long. This aspect has been observed in pratice and the minimum sizes of members given in clause 6.5 of the old code IS:456-1964 should be espected not only for the sake of deflection but also for the sake of durability. The old code requires a thickness of slabs varying from $L/30$ to $L/40$ and depth of beams varying from $L/20$ to $L/25$.

The four factors affecting durability of RC members are steel quality conceret mix type, conceret cover and member size. In the opinion of the author, low strength steels, with rich concrete mixes and reasonable concrete cover and member size will lead to more durable concerte members.

Does it mean that we should go back to low strength steel like Fe250? In that case, steel as a material will be consumed more. Steel is a scarce material and it should be conserved. For this reason, high strength steels like Fe415, Fe500, Fe550 are more in use and this practice is now well established all over the world and we cannot turn the clock back. But, one strongly feels that there is a case for bringing back mild steel bars for the sake of improving duability.

Finally, it may be worthwhile to quote from the comments of a prominent engineer on this write-up "high sterngth steels such as Fe415, Fe500, etc will be of grade M25 and above. M20 also may not be considered as matching concrete for Fe415 steel. Major early malfunctioning of RC structure is due to mis-match combination of ribbed steel and low strength concrete which have been extensively used for the last three decades. The attributed/resultant strain levels in columns and flexural members at select locations are unguardedly permitted to be more than desired. Lateral strains, torsional strains due to ribs, etc. increase the microcracking damage of concrete attracting early corrosion:"

So why not consider reintroducing mild steel rebars selectively?

18.4 FINE POINTS OF STRUCTURAL DESIGN OF BUILDINGS[104]

Some fine points in respect of the structural design of reinforced concrete buildings are listed below for achieving efficiency and economy in design.

18.4.1 Effect of Foundation Excavation

Self-weight of a footing at 10% of the column load is not required to be considered. The weight of a footing is nearly balanced by the weight of the earth excavated equal to the volume of the footing. Hence, the area of footing is given by the column load divided by the given safe bearing capacity of the soil. When a basement is provided in a building, the safe bearing capacity of the soil is increased by the weight of the earth excavated (and not filled back) upto the basement floor level. When two basements are provided in a building, the design soil capacity gets considerably increased leading to economy in footing design.

18.4.2 Depth of Foundation

Foundation depth should be adequate to ensure the entire footing to be buried inside the natural ground. 0.9 m may be the minimum foundation depth, but 1.2 m to 1.5 m are the values generally adopted in practice. For tall isolated structures, foundation depth may be increased to 2.0 m to 3.0 or even more. When there is a filling on a site, foundation is taken further down to rest the footings on the natural ground.

18.4.3 Plinth Beams

Plinth beams are provided to tie columns in both the principal directions and also to support brick walls so that brick wall foundations are thereby saved. Plinth beams may be designed for column base moments so that only the vertical column load goes to the column footing. For earthquake loading also, the plinth beams are designed for column base shear acting as direct tension or compression together with the column base moments. The case of moment combined with tension will govern the design of plinth beams. The top level of plinth beams may be kept either at the plinth level or at 150 mm below the made-up ground level.

18.4.4 Shear Consideration

Footing slab thickness is governed by the punching shear and/or the beam shear. As no stirrups can be provided in footing slab, the minimum allowable shear stress at the ultimate load level should be 0.28 N/mm^2 (for M20) for $p_t \leq 0.15$.

Age factors may also be used where appropriate. For pile caps, generous column pedestals should be provided and the minimum beam shear value can be increased by a factor given by a formula of the code (Ref. clause 40.2.2 of IS 456-2000) not exceeding 1.5.

18.4.5 Concrete Member Size

Sizes of concrete members in foundations should be chosen on the higher side, so that the steel reinforcement can be reduced, resulting in the overall saving in cost. Further, deep members help in reducing or avoiding shear reinforcement in footings.

18.4.6 Column Size

Column sizes should be chosen on the higher side and richer concrete mixes and age factors shall be used in the lower storeys. For durability, the minimum concrete mix in all concrete member should be M20 (1:1.5:2 by volume). For achieving economy in shuttering, column size can be kept the same throughout the height of building (or in steps of a few storeys at the least) varying the reinforcement and the concrete mix as required in the design. Steel bar arrangement in column sections shall be efficient with bars placed away from the centre of section. Slender columns should be avoided, if possible, as these consume more steel than that required for the corresponding short columns.

18.4.7 Beam Depth

Beam depth should be chosen on the higher side, while the beam width on the lower side. The tension steel area in a beam section depends solely on its depth, while the compression steel area will depend on the width also and on the role of slab making the beam to act as a T- or L-section.

18.4.8 Slab Thickness

Slab thickness should be as less as practicable, of the order of 100 mm to 150 mm. Thicker slabs lead to a costlier structure. The thickness of slab panels can be taken as per the old code IS 456–1964. Only the slab panels which directly support brick walls should be checked for deflection as given by the new code

IS 456:2000.

18.4.9 Stairs Slab Thickness

Stairs slab thickness may of the order of 150 mm to 230 mm. Thicker slabs are not architecturally pleasing. Some amount of compression steel may be provided to bring down slab thickness to 200 mm or 230 mm.

18.4.10 Column Loads

When column loads are calculated on the basis of the tributary area method, 5% extra increase in column loads is incorporated to account for unforeseen items and also for elastic beam shear effects. When column loads are calculated by the method of beam reactions, 15% reduction in column loads should be made to get realistic loads.

This is due to the fact that the slab loads in beams are calculated by the formulae

$$\frac{wl_x}{3} \text{ or } \frac{wl_x}{6}\left[3-\left(\frac{l_x}{l_y}\right)^2\right]$$

which otherwise, lead to excessive column loads. In both these methods, live load reduction in multi-storeyed buildings should be made as per the relevant code and the column loads, as modified above, shall be used for column and footing design and also for earthquake analysis.

Column loads can be cross-checked by using 3-D computer (or space frame analysis) under VL (= DL + LL). Further, it is seen that the column loads approximately work out an follows:

Column location	Load intensity over tributary floor area at all supporting levels
Corner	2.5 t/m^2
Exterior	2.0 t/m^2
Interior	1.5 t/m^2

18.4.11 Earthquake Analysis

For earthquake analysis, 3-D computer analysis should be done so that we get realistic and correct column and beam moments. In 3-D or 2-D frame analysis, the effect of slab on beam stiffness should be considered. This will give realistic moments in beams and columns. This aspect is important to consider and it will lead to less moments in columns and realistic moments at centre of beam spans. Alternatively, plane frames are to be considered in either principal direction separately.

18.4.12 Moment in Structural Member

Moments in frame members like beams and columns shall be calculated at face for economy in design. This applies to both analysis under VL and earthquake.

In beam design the support moments as given by the computer analysis under VL, should be reduced to the face by applying manually a face correction equal to $Va/3$, where, V = direct shear in beam, a = column width.

This face correction will reduce support steel in beams.

18.4.13 Factors of Safety

Partial safety load factors as given in the clause 36.4.1 of IS: 456-2000 are erroneous and incomplete. For design of buildings, the following load combinations should be considered.

i) UL = 1.5 (DL + LL)

ii) UL = 1.2 (DL + LL ± EL or WL or TL)

iii) UL = 1.2 (0.9 DL + 0.0LL ± EL or WL or TL)

(applicable for tall isolated structures like chimneys, towers, etc. for checking tension on soil)

Where blast loading (BL) occurring frequency is to be taken into account, the following additional requirement is:

iv) UL = 1.5 (DL + LL + BL)

It is assumed that the worst effects of EL, WL, TL and BL do not occur at the same time. The above notation is explained below:

UL = Ultimate Load

DL = Dead Load

LL = Live Load

EL = Earthquake Load

WL = Wind Load

TL = Temperature Load

BL = Blast Load

Often, expansion joints are provided to dissipate the effect of temperature load (TL).

18.4.14 Pile Foundation

For pile foundations, a few large capacity piles closely spaced should be put under each column. This way, pile cap area will be small and it will lead to economy. The spacing of piles shall be three times the pile diameter which gives 100% efficiency of the pile group.

18.4.15 Under-reamed Piles

For light structures and for buildings resting on black cotton soils, under-reamed piles shall be used. The spacing of such piles shall be kept at 3.75 time the pile diameter with the pile capacity reduced by 10%. This way, we shall get a reasonable size of the pile cap leading to economy.

18.4.16 Need of Computer Solutions

In case of beams crossing each other in plan or in the case of frames with floating columns, computer solutions give correct result and manual methods should not be used.

18.4.17 Hollow Columns

Hollow circular or rectangular columns should be checked for both global and local buckling effects. Often, local buckling governs the design of hollow coluimns.

18.4.18 Use of Rich Concrete for Development Length

When development length of bars at joints of members cannot be adequately provided, the best way to solve this problem is to increase the concrete mix.

18.4.19 Mat or Raft Foundations

Mat or raft foundations are usually designed as slab type total strip footings of width equal to the entire width of footing in plan. When the raft plan is irregular in plan, the loading diagram due to soil pressures from below is proportional to width (b) which is variable in plan. The moments and shear at various sections act over the width of section (b) available at the section under consideration. This way, we get a reasonable and realistic design of raft foundation.

Computer solution of rafts are also available but these lead to small reinforcement in the raft slab for which minimum steel provided shall resist $\pm WL/10$ moments everywhere.

18.4.20 Punching Shear in Slab Type-raft

Although punching shear stress in slabs depend on the column size L_w (31.6.3 IS:456-2000)

$$\tau_{v,a} = k_s.\tau_c$$

$$k_s = (0.5 + \beta_c) \quad (\not> 1.0)$$

$$\beta_c = b/D$$

Square or square columns are beneficial for this aspect. Thin rectangular columns will require thicker slab-type rafts for this reason also (or we should provide square pedestals to these columns for safety in punching shear).

18.4.21 3-D Analysis of Framed Building by STAAD-III

It gives column loads, column moments due to VL and EQ separately. There are two deficiencies in this approach. Corner column loads work out less due to the negative elastic shear beam effects from either principal direction and secondly live load reduction in column loads has not been made. Both these deficiencies are to be made up by manual method on the basis of tributary area method. Column design by STAAD-III considers only centre-line column moments due to VL and EQ and no face corrections are made. This leads to more steel in columns.

18.4.22 Basement Retaining Walls

It is better to assume simply supported edge condition of vertical basement wall at both the ends, instead of the usual propped cantilever system. This way we will get a small and economical footing. This is made possible because of the presence of the basement slab connecting all footings and also the presence of the top GF slab system. For ramp walls, as there is no GF slab, fixed at bottom is required to be assumed with wall free at the top.

18.4.23 Provision of Expansion Joints (EJs) and Shrinkage Strips (SSs)

EJs are good for floors with double columns and double beams, i.e. for basement roofs, floors, etc. SSs are good when floors/walls support earth exposed to atmosphere (refer Fintel, p.133), i.e. when basements roof is exposed to atmosphere. Recommended spacing of SSs is 30.0 m, width of strip = 1.0 m (1 month) to be recast after 1 month, bars to be lapped. Note the bars are not to be made continuous. Location of SS shall be on line of points of contriflexure in members.

18.4.24 Column Design by STAAD-III

STAAD-III gives comparable results for column design with $A_s/4$ in EF but we should put value of l equal to 1.2 times the actual value The good point is that the actual value of k for short column is worked out by the computer program. For thin rectangular columns, the detailing can be altered togive the same effect.

18.4.25 Beam Design by STAAD-III

It is given for the bars i) DL + LL, ii) DL +LL + EQ and the results are correct for all beams except for those main beams which support subsidiary beams at midspans, when the steel at the midspan works out quite less leading to unsafe design. For such beams, we should adapt an alternative approach of using substitute frame program with DL + LL and EQ taken from output of STAAD-III. This way we will get correct steel at midspan, the support steel being the same as before.

18.4.26 Column Detailing by STAAD-III

More steel is given in top storey by STAAD-III. This is due to variation in column axial load. With less axial load and more moments with VL + EQ, steel in column section works out more. In order to reduce steel consumption in top storey, we should not reduce column section and concrete mix in top storey. Further, more steel can be put in top storey, while in lower storeys, less steel is required by design shall be put by suitable detailing. Further, time period in earthquake analysis shall be kept open and it should not be fixed at $T = 1n$. This way, axial load component due to EQ will work out to be of a reasonable value.

18.4.27 Time Period in Earthquake (EQ) Analysis

Statical method of EQ analysis assumes for framed buildings $T = 1.0n$, etc. This gives a small value for T, attracting more earthquake force. Code gives in Note 1, etc. that alternative better methods can be used for evaluation of time period which gives more values for T, leading to less earthquake, resulting in economy. STAAD-III uses Ralrigh-Ritz method for calculating T which is better than the empirical formula like $T = 0.1n$. Thus it gives economical design under earthquake loads. Further dynamical analysis should also be mde to confirm the results given by statical analysis. Both statical and dynamical analyses are as per IS:1893, the code governing earthquake.

18.5 VARIATION IN DESIGNS BY DIFFERENT STRUCTURAL ENGINEERS[105]

Structural design is a science as well as an art. So, design by different structural engineers are expected to be different in terms of planning of structural systems, steel consumption in reinforced concrete (RC) members and in detailing aspects. But, as structural analysis is based on the mathematics of the theory of structures and the structural design on IS456: 2000, the results of a given structural design must not vary greatly. But, in practice, it is observed that vast differences exist in designs of similar buildings by different engineers. This aspect is quite disturbing to all concerned, particularly to a client This is not a happy situation, from a professional standpoint. There are many problems in the planning, analysis and design of buildings, some of which are discussed below.

18.5.1 Problems

Planning: For houses and small buildings (say upto four storeys), load-bearing brick walls with RC floors give an economical and efficient structural system. For taller buildings, RC frames in both principal directions need to be provided, with brick walls acting as filler walls. It is not easy to decide in a given case, whether load-bearing brick walls of frames will be appropriate. This is one of the reasons for difference is design results. Further, frames have to be provided in the two principal directions to resist horizontal loads due to wind or earthquake, which may act in any random direction. A building has been designed with frames in one direction supporting one-way slab system with no beams in the longitudinal direction. This building is vulnerable when wind or earthquake acts in the logitudinal direction. Likewise, for taller buildings, shear walls are required to be provided which will interact with frames in resisting vertical and horizontal loads. However, some designers may consider only shear walls to resist horizontal loads neglecting the interaction of shear walls with frames. Buildings under horizontal loads behave as one unit, so 3-D analysis must be employed for correct results. Some engineers may adopt 2-D analysis in each principal direction or may even consider only one frame under horizontal loads and superimpose this analysis on other frames too, thereby giving scope to considerable error in the results.

Analysis. In small buildings with load-bearing brick walls, it is fair to assume that wind or earthquake forces will be resisted by brick walls by way of box-action in plan. In other words, brick walls act as shear walls in resisting wind or earthquake loads. In small buildings upto four storeys, a twin-system of frames with filler walls can also be considered, wherein vertical load is resisted by frames and horizontal load by filler walls acting as shear walls. This system will lead to a good reduction in steel consumption in buildings. In framed buildings, the effect of floor slab on the moment of inetria of beams should be considered by which moments in columns work out correct and reasonable, leading to an efficient column design. In practice, many engineers do not consider this aspect even in computer analysis, thereby leading to incorrect and expensive column design.

Design: Footings are designed as inverted floor loaded with uniform soil pressure from below. For reducing steel in footings, concrete depth is kept on the high side. Raft foundations may profitably be designed by the method of modulus of sub-grade reaction which gives a variable loading of soil pressure from below.

Column reinforcement is very sensitive to moments acting on a column section about the two principal axes. Some designers may design columns only for axial load, neglecting moments altogethert. This approach gives minimum eccentrictly moments or moments due to continuity with floor beams, whichever is more. Some designers consider only one moment with axial load combined with biaxial bending. Further, slendeness effects in columns must not be neglected. Slender columns consume more steel and these should be checked for both the global and the local bucking effects. Some designers use a method of equivalent axial load. But this is an approximate and crude method and it cannot replace the exact methods available to designers. Moments in coulmn design should be reduced in order to achieve reduction in column reinforcement. In beam design, support moments should also be reduced to get some reduction in steel over beam supports. In computer programs currently in use, centre-line moments are used for design, leading to more steel over support section, The depth of beams is kept on the high side, to reduce steel consumption in beams and its width is kept on the low side, to reduce dead load of beams, but it should be adequate for shear and placement of bars in one or two layers for efficiency.

In slab design, slab thickness should work out less in order to save on the dead load which ensures economy in the design of beams, columns and footings. More steel should be provided at midspan, in order to staisfy defelection requirements of thin slab panels. Engineers, unaware of the above fine points of design, may produce costly or unsafe designs.

18.5.2 Coulmns loads

Column loads can be computed by the following three methods.

(i) 3-D analysis by a computer program
ii) tributary area method
iii) method of beam reactions.

These methods lead to reasonably correct values. These column loads are then used for design of columns and footings. Some designers, under pressure of time, use thumb rules for calculating column loads, for example 1.5t/m^2 of covered area for an interior column, 2.0t/m^2 of covered area for an exterior column and 2.5 t/m^2 of covered are for a corner column. These column loads are then used column and footing design. This is not a correct procedure. It is simular to a mason's design. It is not fair to the client and it is against professional ethics.

18.6 DEFLECTION PROBLEM IN RC MEMBERS[106]

Deflection check is generally relevant for bending members like slabs and beams. Limit state of deflection is a serviceability requirement, and not a safety requirement. When the deflection of a members is more, it does not mean that it is structurally unsafe, it only means that visually it may look ugly and brittle finishes and partitions, if resting directly on it, may crack, but there will be no cracking in the deflected structural member which is, otherwise, adequately designed for moment and shear. So, deflection problem is viewed not as a serious problem from the safety point of view.

For checking the adequacy of bending members in respect of deflection, the code, IS 456: 2000 gives two approaches, one is computation of deflection and the other is *l*/*d* ratio method. The method of computation of deflection is a strict method, but is laborious and time consuming. The *l*/*d* ratio method is an approximate and quick method and it is easy to follow in design practice. The calculated total deflection of a member should not exceed *l*/250. This condition is easy to satisfy in practice by providing adequate camber to the formwork before the member is cast. So, this conditions is not governing in design. The other condition is that the calculated partial deflection of a member due to loads of finishes, partitions, live loads tempeature, shrinking and creep should not exceed *l*/350 or 2 cm whichever is less. This condition is difficult to satisfy in practice and it governs the design. This condition can be satisfied by assuming a suitable camber and also by assuming half the effect of temperature, shrinking and creep, as these loads are slow-acting in nature. For members longer than 7 m in span, 2 cm requirements is also difficult to satisfy. It is suggested that 2 cm requirement should be deleted and only *l*/350. requirement should be insisted upon, this being in line with the ACI code. These two approaches of the code will tally in a given example, if a suitable camber is assumed and the long-term deflection due to temperature, shrinking and creep is halved. This is shown in a worked example given elsewhere.

In practical design of beams *l*/*d* ratio method is used as the beam depths in practice are chosen on the higher side for reducing steel consumption in beams. But, when a particular beam has to be restricted in depth for architectural reasons, the computation of deflection method has to be adopted to justify the reduced depth of the beams. Also, the code specifies that for cantilever beams of spans longer than 10 m, deflection needs to be calculated and l/d ratio method will not be then acceptable. Variation of about 5-10 percent in the calculated and the acceptable values of deflection or l/d ratios is allowed as the deflection is a serviceability requirement.

The deflection requirements of the code have complicated the practical design of slabs in two respects. Firstly, the slab design involves iteration and secondly, the slab panels work out thicker than before when the design was based on the old code. The iteration in slab design can be easily avoided by using charts given elsewhere. But the problem of thick slab panels adds to the cost of buildings and the method suggested in SP: 24 should be used, where more steel is provided at the mid span in order to get slab panels of less thickness.

When a slab panel is not supporting brick partitions, slab thickness can be further reduced using the minimum values given in the old code. The code is silent on this aspect and the young structural engineers fail to appreciate this point. We have learnt these ideas from our seniors in the profession and Fintel has also written in support of thin slab panels for an overall economy in building design. A useful table given elsewhere will be highly beneficial to choose thickness of salb panels, which will be conductive to economy and also avoid interaction in salb design. It needs to be highlighted that adequate care should be taken in selecting slab thickness, especially when the slab is subjected to dynamic loads and vibrations.

On the analogy of thin slab panels, we cannot go in for beams of less depth for two reasons. One reason is that concrete quantity in beams is of the order of 15 percent of the total concrete quantity of the entire building, while the same in slabs is of the order of 65 percent. The second reason is that steel consumption in beams is high and it will go up sharply if beam depth is reduced. In slabs, steel consuption is less and it will, no doubt, go up when the slab thickness is reduced, but the total steel consuption in the building will not be much increased. So, for an overall economy in building design, beam depth should be chosen on the higher side and slab thickness on the lower side, at any rate, not lower than those given in the old code.

l/d ratio method given in clause 23.2 of the code rightly applies to beams only. But in clause 24.1 of the code, it has been extended to slabs also. This extension is correct for one-way slabs only, but its application to two-way slabs and flat slabs/plates is wrong in principle, in that, these slab panels bend in two principal directions, while beams span in one direction only. The deflection in two-way slabs work out quite small and l/d ratio method leads to large slab thickness erroneously. The computation of deflection method can be correctly applied to two-way slabs and flat plates and examples have been worked out elsewhere.

An attempt has been made here to highlight major issues involved in the deflection problem of bending members in order to achieve economical and efficient design of these members for the limit state of deflection.

18.7 SLAB DESIGN OF MINIMUM CONCRETE THICKNESS IN ACCORDANCE WITH IS: 456 – 2000 AND SP: 24 -1983[107]

In accordance with the old code IS: 456-1964, slab design was quick and easy and thickness of slab, panels worked out reasonable. But the new code IS: 456-2000 has made this work quite cumbersome. The slab design now involves iteration, has become time-consuming and it leads to more thickness of slab panels than that given by the slab design based on the old code. With thicker slabs, load on beams, columns and footings has increased leading to a sharp increase in the cost of buildings. Further, this has also adversely affected the economy of buildings in earthquake-prone areas with the increase in the slab dead load (confirmed by Fintel, p.296).

Slabs have to be designed for the limit states of moment and deflection. The thickness of slab is governed by the deflection criteria and the steel area is given by the moment. The deflection criteria have been changed drastically in the new code. In the old code l/D-values for various slab panels were given by which the thickness of slab (D) was directly given satisying the deflection criteria. Then it was easy to calculate the slab loading and the bending moment which was used to get the required steel area and with this, the slab design was complete and over.

But, now the slab thickness has to be first assumed and then the slab loading and the bending moment have to be calculated and the required steel area found. Using this area, l/d-values as per the new code are given and the slab thickness (D) is fixed. This value of slab thickness may be different from that assumed at the outset so that all the above steps have to be redone with a new value of slab thickness.

This process of iteration has to be continued till the difference between the assumed and the final values of slab thickness is negligibly small. It may take about three to five trials to achieve convergence. For each slab panel, this amount of work is required to be done. It is a lot of work involving considerable labour and time.

What was wrong with the slab design based on the old code? The slab panels in older buildings have behaved well and have given, generally speaking, no cause for concern in terms of strength and serviceability. In general, slabs have great reserve strength and they rarely fail. Of course, structural materials have changed now. Previously, mild steel plain bars of slow strength (Fe250) were used as reinforcement. Now, the reinforcement is exclusively of high strength deformed bars (Fe415). There is, however, no change in the concrete mix (1:2:4 or M15). The slab sections are mostly singly reinforced concrete section. Recently, a proposal has come to use M20 concrete in all structural members in place of M15 concrete along with Fe415 bars. Fe500 and Fe550 bars are also available in the market, but these higher strength steels have so far, not caught on, in the general building construction.

Deflection of bending members – slabs and beams – is a natural phenomenon and it is to be controlled for the following two reasons: (i) The total deflection (a_t) should not exceed $l/250$.

This restriction is placed for the reason that the bending member should not look sagging. But this deflection can be easily countered by giving a suitable camber (c) to the form work so that $(a_t - c) \ngtr l/250$. For this reason, ACI code does not consider this aspect important at all, so much so, that there is no mention of this aspect in the ACI code. Further, ACI code commentary states that over-correction of deflection by camber is desirable.

(ii) The partial deflection (a_p) after the erection of partitions should not exceed $l/350$ or 2.0 cm whichever is less i.e. $(a_p - c_{net}) \ngtr l/350$ or 2.0 cm (whichever is less). This restriction is placed to prevent cracking of brittle partitions and finishing resting directly on the bending members.

This aspect is important and there have been a few such instance observed in practice. This aspect refers to the main disadvantage caused by the deflection of bending members. ACI code gives values of $l/360$ or $l/480$ (in place of $l/350$) depending on the material of the partitions resting on the bending members. It should be noted that 2.0 cm value is a great disadvantage for long span beams. For a beam of span, say $l = 20.0$ m,

$$\frac{l}{1000} = 20 \times \frac{100}{1000} = 2.0 \text{ cm}$$

This is a very severe restriction and it is uncalled for. So, it is suggested that $(a_p - c_{net}) \ngtr l/350$, should alone be kept and 2.0 cm. restriction should be dropped in line with ACI code.

There are many slab panels at the roof and the floor levels (including stairs slabs) which do not support brick walls. Such panels can be freed from the restrictions of the deflection criteria and these can be made as thin as suggested by the old code. This proposal is in conformity with the spirit of the new code. For slab panels directly supporting 115 mm thick brick walls, the deflection criteria as given by the code shall be considered in design. For supporting 230 mm. thiick brick walls, the general practice is to provide beams under these walls and the depths of these beams are, generally more than adequate for the deflection criteria.

It is vital for economy to strive for a minimum thickness of slab panles as slabs consume about 50% of the total concrete quantity of a building. For beam design, it will be conducive to economy if the beam depth is chosen on the high side, as the concrete consumption in beams is small, being of the order of 15% only.

The main deficiency of the new code lies in clause 23.1 of the code which states that deflection provision (clause 22.2) for beams shall apply to slab also. Now, this is true only for one-way slabs, but it is definitely not valid for two-way slabs. This aspect needlessly gives more thickness for two-way slabs and it adversely affects the economy.

The net result of the above study is that slab panels – not supporting brick walls can be made as thin as suggested by the old code and only the slab panels which support brick or similar brittle partitions should be checked for the deflection criteria given in the new code.

18.7.1 Design of Slab Panels Supporting Brick Partitions

The new code can be directly used to design such slab panels by trial and error. The direct approach involving no iteration is given by N. Pandian and also by U.H. Varyani which are given in Indian Concrete Journal issues of January 1989 and May, 1989 respectively. The slab panels work out quite thick on the basis of the new code. In order to reduce slab thickness, SP:24 1983, Explanatory Hand Book on the new code gives a method which can be easily used to achieve slab panels of less thickness than that required by the new code. SP: 24-1983 (page 55) states that "if one wished to use a shallow member, the deflection can be kept within the required limit by providing more tension reinforcement than that is required from strength consideration or working stress considerations, thereby reducing the service stress in steel. The multiplier can then be arrived at by suing the formula given above. However, while increasing the amount of tension steel to decrease deflection, the limitation of 37.1(f) should be kept in view, in case of sections proportioned for limit state of collapse".

The formula for the multiplying factor (MF) given by SP: 24-1983 is written as,

$$MF = 1/1[0.225 + 0.00322 f_s - 0.625 \log(bd/100\,A_{st})] \tag{18.1}$$

where, $f_s = 0.58\,f_y$, $(A_{st}\text{ reqd})/(A_{st}\text{ prov})$, (18.2)

Fig. 3 of the new code is based on Eq. (18.1) with $f_s = 0.58\,f_y$ giving three curves for steel types Fe250, Fe415 and Fe500, and it also gives MF $\ngtr$ 2.0. For accuracy in calculations, Eq. (18.1) may be directly used, instead of reading values of MF from Fig. 3 of the Code.

When the steel area is increased for the purpose of reducing deflection, the steel strain must exceed,

$$\varepsilon_{sod} = 0.87\frac{f_y}{E_s} + 0.002$$

$$= \frac{0.87 \times 415}{2 \times 10^5} + 0.002 = 0.0018 + 0.002 = 0.0038$$

for the steel type Fe415 to be ductile. The steel strain in tension steel in a singly reinforced section is given by,

$$\varepsilon_{st} = [(d - x_u)/x_u)] \times 0.0035 \qquad (18.3)$$

where,

$$\frac{x_u}{d} = 0.87 f_y A_{st}/0.36 f_{ck}bd \qquad (18.4)$$

These formula are given in the Appendix E of IS: 456-1978.

For the tension steel to be ductile, ε_{st} must exceed 0.0038 (for Fe415) in all cases. This is the requirement of clause 37.1(f) of the new code. It is seen that steel area may be increased by about 25% to 50% of that required for strength consideration, for the tension steel to remain ductile. Too much increase of steel at the mid-span section for the purpose of reducing defelction is not desirable for reason of economy and also for the reason that steel may not then remain ductile.

By using this method suggested in SP: 24-1983, which is only an elaboration of the method given in IS: 456-2000, slab panels can be designed to have concrete thickness less than that given by the new code. Slab thickness on the basis of the old code, the new code as SP: 24-1983 for various slab panels are given in Table 18.4.

A numerical example is given to illustrate the above method.

18.7.2 Continuous Slab Panel Spanning in Two Directions (Corner Panel)

Given: $l_x = 4.0, f_y = 415$ N/mm²

$l_y = 6.0$ m, $f_{ck} = 15$ N/mm²

Live load = 3.0 kN/m²

Finished load = 1.5 kN/m²

Wooden partitions = 1.0 kN/m²

Required: Slab design of minimum thickness satisfying moment and deflection requirements.

Method 1: Based on IS: 456-1978 I.C.J. May 1989:

w_1 = Loading intensity excluding the self weight of slab

= 3.0 + 1.5 + 1.0 = 5.5 kN/m²

$$\frac{w_1}{l_x} = \frac{5.5}{4.0} = 1.375 \approx 1.4$$

$\beta = 0.056$ (Table 22 of the new code)

$\beta l_x = 0.056 \times 4.0 = 0.224$, Charts give,

$$\frac{l_x}{d} = 38$$

$$d = \frac{400}{38} = 10.5 \text{ cm}$$

$$D = 10.5 + 1.5 + 0.5 = 12.5 \text{ cm}$$

$$w_2 = 0.125 \times 25 = 3.13 \text{ kN/m}^2$$

$$w = w_1 + w_2 = 5.5 + 3.13 = 8.63 \text{ kN/m}^2$$

$$+M_x = +0.056 \times 8.63 \times (4.0)^2 = +7.73 \text{ kNm.}$$

$$d = 125 - 20 = 105 \text{ mm}, b = 1000 \text{ mm}$$

$$+k_x = \frac{1.5 \times 7.73 \times 10^6}{1000 \times (1.05)^2} = 1.05 \text{ N/mm}^2$$

$$+A_{stx} = 0.32 \times 10.5 = 3.36 \text{ cm}^2/\text{m}$$

(ϕ 8 bars may be used, $d = 10.6$ cm)

Eq. (18.1) gives for $f_s = 0.58 \times 415 = 240.7$ N/mm^2,

$$\text{MF} = \frac{1}{0.225 + 0.00322 \times 240.7 - 0.625 \log\left(\frac{10.6}{3.36}\right)}$$

$$= \frac{1}{0.225 + 0.775 - 0.312} = \frac{1}{0.688} = 1.45$$

$$\frac{l_x}{d} = 26 \times 1.45 = 37.7$$

$$d = \frac{400}{37.7} \approx 10.6 \text{ cm}, D = 10.6 + 1.9 = 12.5 \text{ cm}$$

Thus $D = 12.5$ cm, is adequate for strength and deflection both. Now, we should strive for a thinner slab for an over-all economy in structural design of the building.

Method 2: Based on SP: 24-1983, Table 18.4 gives

$$\frac{l_x}{D} = 35$$

$$D = \frac{400}{35} = 11.4 \text{ cm of } 11.5 \text{ cm}$$

Table 18 4. Slab thickness by various methods (M20, Fe415).

S.	Description of slab panel	l/D (or l_x/D) values.		
		IS: 450-1964	IS: 456-2000	SP: 24-1983
1.	Simply supported slabs spanning in one direction	30	23	25
2.	Simply supported slabs spanning in two directions	35	25	30
3.	Continuous slab spanning in one direction	35	28	30
4.	Continuous slab spanning in two directions	40	32	35
5.	Cantilever slabs	12	9	10
Remark	Validity of application	For slab panels not supporting brick partitions	For slab panels directly supporting brick partitions	

$$w = 5.5 + 11.5 \times 25 = 8.375 \text{ kN/m}^2$$

$$+M_x = +0.056 \times 8.375\,(4.0)^2 = 7.5 \text{ kNm}$$

With $b = 1000$ mm, $d = 95$ mm,

$$+k_x = \frac{1.5 \times 10^6}{100 \times (95)^2} = 1.25 \text{ N/mm}^2$$

$$+A_{stx} = 0.389 \times 9.5 = 3.7 \text{ cm}^2/\text{m}$$

($\phi 8$ bars will give $d = 9.6$ cm)

Eq. (18.1) gives,

$$\text{MF} = \frac{1}{0.225 + 0.775 - 0.625 \log\left(\frac{9.6}{3.7}\right)} = 1.35$$

$$\frac{l_x}{d} = 26 \times 1.35 = 35.1$$

$$d = \frac{400}{35.1} = 11.4 \text{ cm}$$

$$D = 11.4 + 1.9 = 13.3 \text{ cm} > 11.5 \text{ cm assumed before.}$$

This shows that $D = 11.5$ cm is not adequate for deflection. Let us put more steel at the mid span, say, equal to that required for the support-moment. This gives a satisfactory detailing arrangement with bent-up bars.

$$-M_x = -0.075 \times 8.375 \times (4.0)^2 = -10.05 \text{ kNm}$$

$$-K_x = \frac{1.5 \times 10.05 \times 10^6}{100 \times (96)^2} = 1.64$$

$$-A_{stx} = 0.534 \times 9.6 = 5.13 \text{ cm}^2/\text{m } (\phi \text{ 8 bars ok})$$

Let us provide at mid span $+A_{stx} = 5.13$ cm^2 giving about 39% more steel than that required for the strength considerations. Eq. (18.2) gives,

$$f_s = 0.58 \times 415 \times \frac{3.7}{5.13} = 174 \text{ N/mm}^2$$

Eq.(18.1) gives,

$$\text{MF} = \frac{1}{0.225 + 0.00322 \times 174 - 0.625 \log\left(\frac{9.6}{5.13}\right)}$$

$$= \frac{1}{0.225 + 0.560 - .170} = \frac{1}{0.615} = 1.63\ (< 2.0)$$

$$\frac{l}{d} = 26 \times 1.63 = 42.38$$

$$d = \frac{400}{42.38} = 9.4 \text{ cm}$$

$$D = 9.4 + 1.9 = 11.3 \text{ cm} < 11.5 \text{ cm}.$$

Thus $D = 11.5$ cm, is adequate for strength and deflection both, with about 39% more steel provided at the mid span than that is required for strength considerations.

Check on the ductility of tension steel: Eq. (18.4) gives,

$$\frac{x_u}{d} = \frac{0.87 \times 415 \times 5.13}{0.36 \times 15 \times 9.6 \times 100} = 0.357,\ x_u = 0.357 \times 9.6 = 3.4 \text{ cm}$$

$$\varepsilon_{st} = \frac{(9.6 - 3.4)}{3.4} \times 0.0035 = 0.0064 > 0.0038 \text{, i.e. steel is ductile, OK}$$

Method 3: Based on IS: 456-1964.

If it is known that no brick wall is to rest on this slab panel (the load considered has no wall load component), we may use thickness as per the old code. Table 18.4 gives,

$$\frac{l_x}{D} = 40,\ D = \frac{400}{40} = 10 \text{ cm}$$

$$w = 5.5 + 0.10 \times 25 = 8.0 \text{ kN/m}^2$$

$$+M_x = +0.056 \times 8.0\,(4.0)^2 = 7.2 \text{ kNm},$$

$d = 80$ mm, $b = 100$ mm

$$+k_x = \frac{1.5 \times 7.2 \times 10^6}{100 \times (80)^2} = 1.69,$$

$$+A_{stx} = 0.554 \times 8.0 = 4.43 \text{ cm}^2/\text{m}$$

$$-M_x = -0.075 \times 8.0 \times (4.0)^2 = -9.6 \text{ kNm}, -k_x = 2.2$$

$$\frac{d'}{d} = \frac{1.9}{8.1} = .23 = .20$$

$$-A_{stx} = -0.76 \times 8.1 = 6.16 \text{ cm}^2/\text{m},$$

$$-A_{scx} = -0.04 \times 8.1 = 0.23 \text{ cm}^2/\text{m},$$

$$+M_y = +.035 \times 8.0 \times (4.0)^2 = +4.48 \text{ kNm},$$

$$d = 81 - 8 = 73 \text{ mm}, +k_y = 1.26$$

$$-M_y = -.047 \times 8.0 \times (4.0)^2 = -6.02 \text{ kNm}, -k_y = 1.61,$$

$$+A_{sty} = 2.86 \text{ cm}^2/\text{m}\ A_{sty} = 4.04 \text{ cm}^2/\text{m}$$

ϕ 8 bent up bars will be used in both the directions.
D = 10 cm is adequate for strength considerations and deflection criteria are not important to be applied as no brick partitions rest on the slab panel.

For an overall economy, minimum slab thickness as given by the old code should be chosen.

18.8 DIRECT DESIGN OF SLAB PANELS IN ACCORDANCE WITH IS: 456-2000[108]

It is well-known that the slab design in accodance with IS: 456-2000 is an iterative process which consumes a lot of time and effort. The iteration involved in the slab design has been eliminated by the use of tables.

In the approach followed in the above references, p_t has been elemi-nated giving a relation between βl, w_1/l and l/d. With βl, w_1/l known, l/d is easily found. Then the final slab thickness D is directly given, from which area of steel reinforcement is calculated and bars are chosen.

Here, an alternative approach is given in which l/d is eleminated giving a relation between βl, w_1/l and p_t. With known, p_t is calculated by a trial and error method. The l/d is calculated with the known value of p_t by which the final slab thickness D is fixed and the steel reinforcements are decided.

In the earlier approach, relations between l/d and p_t for various slab panels were found from the curve given in SP: 16-1980, which involved approximation. But in this paper, the approach is direct using a relation given by the SP: 24-1983, which is basis of Fig. 3 of the Code. In this respect, the present approach may be considered more rigorous.

18.8.1 Derivation of Formulae

Slab design is governed by the limit states of bending moment and deflection. The steel area is given by the bending moment and the thickness is governed by the deflection criteria. For singly reinforced slabs, the tension steel area (A_{st} or p_t) is given by:

$$\frac{M_u}{bd^2} = 0.87 f_y \left(\frac{p_t}{100}\right)\left[1 - 1.005 \cdot \frac{f_y}{f_{ck}}\left(\frac{p_t}{100}\right)\right] \tag{18.5}$$

where,

$$p_t = \frac{A_{st}}{bd} \times 100 \tag{18.6}$$

Clause 22.2.1 of the code gives,

$$\frac{l}{d} = f \times (\text{MF}) \tag{18.7}$$

where, f = 20, 26 and 7 for simply supported, continuous and cantilever slabs respectively, and the multiplying factor MF ($\not> 2.0$) is given by Fig. 3 of the Code, which is based on the following relation,

$$\text{MF} = \frac{1}{0.225 + 0.00322 \times f_s - 0.625 \log\left(\frac{1}{p_t}\right)} \tag{18.8}$$

For the steel type Fe415 and for strength design,

$$f_s = 0.58 f_y = 240.7 \text{ N/mm}^2$$

Eq.(18.8) is modified to:

$$\text{MF} = \frac{1}{1.0 - 0.625 \log\left(\frac{1}{p_t}\right)} \tag{18.8a}$$

Eq (18.7) gives:

$$\frac{l}{d} = \frac{f}{1.0 - 0.625 \log\left(\frac{1}{p_t}\right)} \tag{18.7a}$$

Now, the bending moment (M) at the critical section of a slab panel of span l (in metres) is given by,

$$M = \beta \cdot wl^2 \tag{18.9}$$

where,

$$w = w_1 + w_2 \text{ (kN/m}^2\text{)}$$

w_1 = slab loading less the self-weight of slab (kN/m²)

w_2 = self-weight of slab = $\rho . D$ (kN/m²),

$$\frac{D}{d} = 1.25 \text{ (assumed)}, \ \rho = 25 \text{ kN/m}^3$$

Then,

$$M_u = 1.5\,M = 1.5\beta\,(w_1 + w_2).\,l^2$$

$$\frac{M_u}{bd^2} = 1.5\,\frac{\beta}{b}\,(w_1 + 31.25d)\,(\frac{l}{d})^2$$

Putting $b = 1.0$ m and expressing $\frac{M_u}{bd^2}$ in units of N/mm²,

we get,

$$\frac{M_u}{bd^2} = 0.0015.\,\beta l\,.\left[\frac{w_1}{l} + \frac{31.25}{(\frac{l}{d})}\right].\left(\frac{l}{d}\right)^2 \tag{18.9a}$$

Eq. (18.5) gives for $f_y = 415$ N/mm², $f_{ck} = 15$ N/mm²,

$$\frac{M_u}{bd^2} = 3.6105\,p_t - p_t^2 \tag{18.9b}$$

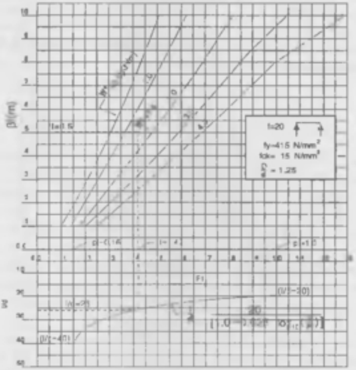


Fig. 18.1.Direct design of slab panels in accordance with IS: 456-1978 (valid for simply supported one-way and two-way slabs).

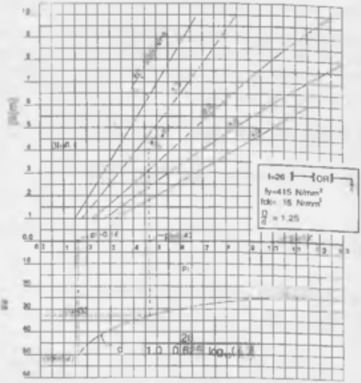


Fig. 18.2. Direct design of slab panels in accordance with IS: 456-1978 (valid for continuous one-way and two-way slabs).

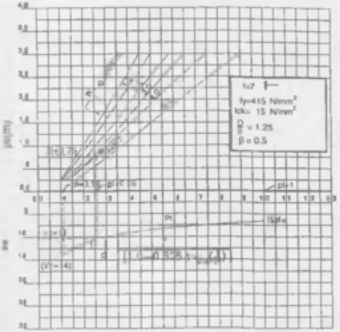


Fig. 18.3. Direct design of slab panels in accordance with IS: 456-1978 (valid for cantilever slabs).

Eqs. (18.7a), (18.9a) and (18.9b) give,

$$3.610\, p_t - p_t^2 = 0.0015.\ \beta l$$

$$\left[\frac{\omega_1}{l} + \frac{31.25}{f} \times \left(1.0 - 0.625 \log\left(\frac{t}{\sqrt{p_t}}\right)\right)\right] \tag{18.10}$$

$$\times \frac{f^2}{\left[1.0 - 0.625 \log\left(\frac{t}{p_t}\right)\right]^2}$$

In this equation, βl, w_1/l and f are known and only p_t is unknown, which can be solved by a trial and error method. Eq. (18.7a) can then be used to get l/d

Figs. 18.1 to 18.3 have been prepared on the basis of Eqs. (18.10) and (18.7a), by which p_t and l/d values can be found for simply-supported, continuous and cantilever slab panels. Two-way slab panels can be designed in the same way by replacing l with l_x and with β being given by Table 22 of the Code.[108] The only assumption made is D/d = 1.25. D/d varies from 1.25 to 1.10 for slab thickness of 10 cm to 20 cm. Fortunately, l/d is quite insensitive to the variation in the values of D/d. So, even one value of D/d can be assumed and the resulting slab thickness can be accepted without any further review. However, D/d = 1.25 gives a slightly conservative value of slab thickness. Figs. 18.1 to 18.3 are easy to use and are self-explanatory. Numerical examples are shown in Figs. 18.1 to 18.3 by way of dotted lines which also indicate the way the charts should be used.

18.9 LESSONS FROM BHUJ EARTHQUAKE[109]

A servere earthquake occurred on the morning of 26 January 2001 with its epicentre near Bhuj (Gujarat) and the entire surrounding area was devastated entailing a huge loss of life and property. The Bhuj area in Gujarat falls in earthquake zone V which is the most severe earthquake zone in the country. Other areas of similar severity fall in Jammu and Kashmir, upper Garhwal in Uttaranchal, Assam and other north-east areas. Earthquake activity is not new in the Bhuj area, but this time the earthquake effect was on a mammoth scale. Further, economic activityand population in the area has increased in recent years so that damage to life and property is more visible and painful than before.

The electronic and print media have brought this incident vividly before the notice of the general public all over India. The public is now rightly worried about the safety of buildings not only in Gujarat but elsewhere too and the role of building contractors, architects and structural engineers is under close scrutiny. All these professionals including the owners of buildings have to learn a few lessons from this earthquake.

Buildings which are properly planned, designed and constructed have a good capacity to withstand earthquakes. The Bureau of Indian Standards (BIS), New Delhi, has published a few codes – IS 1893: 1984, Earthquake resistant design of structure; IS 4326: 1993, Criteria for earthquake resistant construction; IS 13920, Code of practice for detailing of reinforced concrete structure subjected to seismic forces and SP: 22 (S&T)-1982, Explanatory handbook on earthquake engineering. Structural engineers are duty-bound to follow these useful publications and past experience has shown that buildings designed in accordance with these IS codes have withstood earthquake well. The design philosophy of earthquake resistant design is that buildings should not collapse thereby saving lives of residents, but these may crack to a limited extent so that the cost of repairs to damaged buildings is reasonable and bearable.

Earthquake-resistant design of buildings costs money. The owners of buildings should understand this aspect and give up old habits of saving money at all costs. The immediate effect of earthquake-resistant design is the increase in the quantity of steel reinforcement in buildings. About 25 to 30 percent more steel is required to be consumed when a building is designed for earthquake forces. This means that the total cost of building may be increased by 4 to 5 percent on this account. The owners of buildings should bear this in mind and be mentally prepared for this extra cost.

Further, soil investigation of the building site is essential. This is a regular practice nowadays and the cost of this exercise is not much and it pays itself by effecting savings in the foundation design. In earthquake-resistant design, soil quality plays an important role and a suitable factor has to be assumed depending on the quality of soil. Buildings on hard soil, in general, are likely to withstand earthquake forces well, entailing less damage.

Architects need to plan symmetrical buildings with lesser projecting parts as these parts are susceptible to damage during earthquakes. Further, square columns are very efficient in resisting earthquakes and architects should not insist on columns of 230-mm width, concealing them within 230-mm thick brick walls. The column sizes work out too large in earthquake-resistant design and these columns will protrude out of the adjoining brick walls. Beam sizes too are affected by earthquake forces and architects should not insist on lesser depth for beams. Beams of limited depth require more steel consumption in buildings. In areas of severe earthquake zone, symmetry in building plans should be insisted upon. Asymmetrical buildings should be provided with expansion joints in order to divide these buildings into different parts, each of which needs to be symmetrical in plan.

Both owners and architects of buildings are required to cooperate with structural engineers so that buildings are properly designed to withstand earthquakes successfully. Structural engineers should play their professonal role and follow the BIS codes strictly. If a well-designed building bears earthquake forces well, the credit goes to the concerned structural engineer, architect and contractor.

Building contractors should remain true to their function and they should never compromise with the quality of construction materials and practices. An earthquake is an acid test of all professionals involved in the building industry.

Public are well advised to go in for insurance of their buildings against hazards due to earthquake. Many insurance companies offer substantial rebate in premium amounts if buildings are designed for earthquake forces. The recent earthquakes have shown that nearly 60 to 70 percent area of the country falls under moderate to severe intensity of earthquake.

18.10 ROLE OF CHECKING ENGINEERS[110]

Checking of structural design is mandatory in some European countries like Germany. A client has to engage an architect, a structural engineer and a contractor to build a structure. Likewise, he has to engage a checking engineer too, who will check the strcutural design and also supervise the construction at the site. A separate supervision agency may also be engaged to undertake site supervision. In India, it is not the general practice to get the structural design checked by another engineer. It is all based on professonal trust. In the medical field too, a patient consults a doctor with implicit faith. Rarely another doctor is brought in to give a second opinion in ordinary cases. In serious cases only, it is desirable to consult other doctors, all for the benefit of the patient. In the field of structural design, checking of structural design is insisted upon in the case of public buildings. In private construction, structural design often goes without checking, basing it on professional trust.

In the present situation, small buildings like houses, etc. are invariably designed by mistries or foremen who have only experience as their guide. Private constructions are designed by firms of architects and engineers with no outside checking of structural design. Public buildings are designed by PWD engineers with in-house checking of structural design. Large public building projects may be given to private firms of architects and engineers and the structural design is checked by prestigious institutions like IITs and other engineering colleges or engineering firms of repute.

18.10.1 Design and Checking Engineers

A design engineer enters a project at the initial stage itself and after a lot of thought, effort and time, he prepares the structural scheme, calculations and drawing. A checking engineer, on the other hand, enters the project much later and he should give due respect to the scheme and design brief of the design engineer. Checking of a structural design can be of great benefit to both the design and the checking engineers. In some cases, it can become an acrimonious exercise too. But, it should be taken as an opportunity to learn. There is no end to learning in any field, more so, in the field of structural design.

The purpose of checking of a structural design should be to seek out any weak points left in the structural arrangement proposed, structural loadings assumed, analysis and design procedures adopted and detailing methods used in the work. It is not difficult to find errors in a work. Whoever works, he only can commit an error. It is true that a design engineer should aim at an error-free work. But no engineer is perfect and a few errors may easily creep up in the work. It is the purpose of checking engineer to appreciate the good points of design and also to seek out its weak points so that the structure gets the benefit of his checking. The aim of checking of a structural design is not to demolish it but to improve it and make it error-free.

Checking process can be of, say, three types. One type may be called a mature type of checking wherein the checking engineer goes over the design and drawing and critically examines the structural arrangement, loads, design and detailing methods, etc. and speedily comes to a conclusion. The other type of checking can be of a hair-spliting type, wherein the checking engineer goes page by page and word by word and finds a lot of points of divergence. This type of checking process will take more time and effort to conclude. The third type of checking is the worst type wherein the checking engineer has no appreciation of the work of the design engineer and he superimposes his own ideas on the work completed. Thhis leads to a lot of acrimony and it may lead to no agreement at all.

There is always a gap between the position of the design engineer and that of the checking engineer. The gap becomes wider if the checking engineer happens to be the less mature of the two. The ideal situation will be to choose a checking engineer who has more experience than that of the design engineer. Then the gap will be less and it will take less time and effort to bridge the gap and come to a common position. In American parctice, checking of structural design is called a peer-review. This word indicates that both the design and the checking engineer should be comparable and equal in stature. If the checking engineer is inexperienced, he is likely to bring down the quality of the structural design and drawings. It is good to assume that we all engineers are equally good and that no engineer is inferior to the other and we must not have a know-all attitude.

18.10.2 Divergent Views of Engineers

It is quite natural that two engineers may have divergent views on structural design. This happens due to diverse backgrounds. In India, we all engineers learn from the same books, are taught similar syllabus by professors of similar calibre. Then, we should not have divergent positions on structural design. But as we have different backgrounds in terms of our experience, we tend to think differently on many matters of structural design. Many checking engineers come from teaching background; others may have design background with or without computer knowledge. Even with the same design background, each engineer has his own unique experience, on which he builds his approach and knwoledge. This is the reason, why two engineers may not agree on a given issue. The same happens in other fields too. Even two doctors may not have the same opinion or approach on a given case.

18.10.3 Aberrations in Structural Design

There are many aberrations in structural design observed in practice. Some designers may design only for vertical loads without taking any care of horizontal loads. Others may design the structural frame for vertical loads only and decide to give horizontal load to filler brick walls which resist these loads by way of box-action in plan. This sort of system may work well for short buildings. For tall buildings, reinforced concrete frames are efficient for resisting both the vertical and the horizontal loads. For still taller buildings, shear walls have to be provided to resist horizontal loads. Some designers consider only one frame under horizontal load in each principal direction and superimpose these results on all other frames, without considering the building as a whole, which is the way a building behaves under horizontal loads. In an actual building, it was observed that frames were provided in the transverse direction only, there being no beams in the longitudinal direction. Some designers prefer framed structure to a flat slab structures for multistoreyed buildings, others prefer the latter. It is known that flat-slab structure is efficeint to resist vertical loads and it is inefficient for horizontal loads. So, ideally flat-slab structures should be used for short buildings. Some designers give the entire horizontal shear to shear walls with frames being designed only for vertical loads. Other designers will go in for shear wall-frame interaction to get a realistic solution. In an existing building, it was observed that slab panels of 7.8 m × 7.8 m were designed as one way spanning with main steel in one direction and with only the distribution steel in the other. This practice does not conform to the actual behaviour of the slab panel.

There can be aberrations in foundation system too. There can be a difference of opinion on the use of raft or pile foundations. Further, a raft foundation works out costly when strip, combined or isolated footings may be adequate. Some designers feel that a raft foundation is necessary when basement is provided. Others may feel that a base slab of 100 mm to 150 mm thickness, with isolated and combined footings will work out adequate and economical too, for buildings with a basement. Concrete depth in foundation members is kept on the high side to save on steel consumption and also to make for safety in shear. These ideas come with time and one learns these ideas from the seniors in the profession. Because of the probability of aberrations in the design of super-structure and foundations of buildings, checking of structural design becomes necessary and profitable too.

18.11 COMPUTER AND STRUCTURAL ENGINEER[11]

Computers are being increasingly used for the structural design of all types of structures. Computers are not, however, infallible. They are as good as the program fed into them. The programs or the software are made by engineers who are as good as the rest of the engineers. Further, programs are based on structural theory and design codes which are used by other engineers in their manual methods. So programs and manual methods are based on the same principles. Therefore, any design based on a computer cannot be automatically taken as superior to the one done by manual method.

18.11.1 Deficiencies in Computer Programs

Many standard programs are currently in use but the underlying assumptions made in the formulation of these programs remain obscure. It is stated that stiffness matrix method is the basis of the 2-D and 3-D analysis of framed buildings. But, it is not clear whether the contribution of the floor slab in the stiffness of beams members has been considered, or only the rectangular beam sizes have been considered. The behaviour of buildings becomes realistic if this slab effect is included in the stiffness of beam members. The effect of floor-slab diaphragms in the analysis of structures under vertical and horizontal loads cannot be wished away and it will not be correct to do so for the sake of the realistic behaviour of structures under loads.

Further, it is not clear if the slab loads on beams are taken as triangular and trapezoidal distributions as given in IS: 456-1978, or some other equivalent uniformly distributed loading has been considered. This aspect can itself be a source of error leading to wrong column loads.

It is seen that the corner column load works out to be quite less in 3-D analysis of a building. This may be due to the fact that the negative elastic shears of beams on either side of a corner column have been considered, which is a correct theoretical position. But, in practice, negative elastic beam shears are always neglected and only the positive ones are included. This aspect, either the programmer does not know or he prefers to ignore it.

In column design, a computer program considers l_{ef} = 1.0l while the code requires l_{ef} = 1.2 l (minimum) for framed buildings. So, while using such computer programs, one should increase the value of l suitably to get the correct column design. The column design considers area of steel to be distributed as $A_s/4$ on each face of a column sections which is convenient for square or squarish column section. For long rectangular sections, this steel arrangement is not practical and some equivalent steel arrangement has to be devised which is compatible with the theoretical position assumed in the program.

In some computer programs of beam design, support moment at the centre-line of column is considered, which gives more steel bars on the top of the beam support section. The correct requirement would be to consider support moment at the column face for design of the beam support section. Such programs should be updated or the face-correction to the support moments should be applied manually to get the correct result.

Slab design based on a computer solution will give a large slab thickness in keeping with the requirements of moment and deflection in accordance with IS:456-2000. But experience has shown that the slab thickness should be as small as practicable, as about half the dead load of the structure of the building is on account of the slab self-weight. Fintel also states that "experience has shown that in tall buildings, the best economy is achieved with the thinnest slab, as the slab weight also affects the load on columns and on foundations". Further, in earthquake-prone areas, thin slabs will lead to less earthquake shear on buildings, leading to economy. This aspect should form a part of updated computer programs on slab and building design.

There is some confusion in the load factors given in Table 12 of IS:456-1978 and also in the codal provisions of deflection of members. These need to be sorted out and the correct approach is to be used in the updated computer programs.

Further, foundation members – slabs and beams – should be chosen on the high side in order to save steel consumptions. Also, minimum shear stress in concrete shall be taken 0.28 N/mm^2, otherwise, slab in footings and pile caps will work out to be unnecessarily thick. These aspects are not mentioned in the codes and these are likely to be ignored by the programmers. Computer solutions for raft foundations are available on the basis of the modulus of subgrade reaction, taking soil as an elastic medium. Manual methods on the same basis by Teng and other are tedious to apply. But the computer solutions lead to less steel in raft members and some designers like Gupta suggest design of raft foundations with $M = \pm$ WL/10 as minimum moments, so that some reasonable amount of reinforcement is provided in the raft foundations. This aspect too needs to be considered in the available computer programs.

18.11.2 Advantages of Computer Programs

In certain situations, the use of computers becomes a necessary requirement. When a beam rests on another beam, it becomes difficult to decide which is a subsidiary beam and which is the supporting (main) beam. But use of a computer program will give a correct and realistic solution, as these beams inter-support each other. When floating columns are used in frame, a computer solution will lead to correct result and manual methods will have to consider sinking supports leading to long and tedious calculations which become quite impractical.

For earthquake analysis of buildings, particularly with asymmetrical plans, 3-D analysis will lead to correct results and torsion in plan will be automatically taken into consideration. This is a great advantage of computer use. Frame analysis by computer is superior to approximate manual methods like portal and cantilever methods, which have now become obsolete.

Design of slender columns is neatly done by a computer program, giving the exact value of k, which, otherwise, has to be assumed at the first instance in manual methods leading to iteration.

The greatest advantage of computers is the speed of analysis. But data preparation and input feeding takes considerable time. Further, a practical difficulty is that computer may go out of order. So, these aspects do retard the speed of computer solutions.

Further, it is easy to repeat computer analysis when beam or column sizes are required to be revised. This is the greatest advantage of computers that many revisions can be easily performed.

Structural earthquake analysis by STAAD III uses alternative methods of finding value of T which works out more than that given by empirical relations like $T = 0.1n$, etc. thereby getting reduced values of earthquake force acting in building and it leads to economy. This approach is sanctioned by the code IS: 1893.

18.11.3 Computer versus Engineer

It is a misconception that computer is superior to engineer. Generally, young and brilliant engineers go in the field of computer programming. They may lack the experience and the understanding of the behaviour of structures which only comes with experience. They need to be guided to consider the practical aspects in the development of computer programs.

One may ask whether books giving ready-made tables and curves are at all required now when the standard computer programs are so readily available. Ready-made solutions of cylindrical shells derived by a computer program and given by Gibson and Cooper are still useful, thereby saving time and cost. In practical terms, such books will be always required for ready availability of solutions and also for effecting economy in the design effort. A computer may go out of order needing repairs, but a book is always available at our disposal for use.

An engineer is always required to test the veracity of a computer solution with a manual method. Otherwise, what is the guarantee of the correctness of a computer programme? So, an engineer is superior to a computer which is no better than an engineer who has developed the software fed into the computer.

CHAPTER 19

New Reinforced Concrete Code and Structural Design

19.1 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 the new code can be easily and correctly applied in structural design.

19.1.1 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 of 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 engineer has suggested to have a map of India with exposure zones clearly marked therein, on the analogy of 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: 456-1978 have been observed in the new code IS: 456-2000.

i) 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.

ii) Minimum concrete mix is changed from M-15 to M20, M25 and M30 in respect of mild, moderate and severe exposures zone respectively.

iii) Clear cover to outer bars has been increased. Previously, the clear cover was specified over main bars.

iv) Minimum slab thickness has to be 110 mm for 1-1/2 hour fire resistance. Previously, minimum slab thickness, in practice, was 100 mm.

v) Clear cover over outer slab bars has been increased from 15 mm to 20 mm, taking both tables 16 and 16A of the code into account.

vi) PCC shall be M10 (1:4:8).

viii) 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.

xi) Minimum shear strength of concrete shall be considered as given for $p_t \leq 0.15$ in Table 19 of the code. For M20, for example, $\tau_c = 0.28$ N/mm^2 for $p_t \leq 0.15$. Previously, were assuming $\tau = 0.35$ N/mm^2 for $p_t \leq 0.25$.

x) 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%.

Table 19.1 Requirements of IS: 456-2000 for durability and fire resistance in design of buildings (based on Tables 3, 5, 16, 16A of the code).

Exposure zone	Where applicable	Minimum concrete mix	Nominal cover for members (mm)	Maximum thickness of slab (mm)	Remarks
Mild	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 Column 40 Footing 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 Footing 50 Ret. wall 40	140	For structures immersed in sea water

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.

19.1.2 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.

i) Partial safety factors for loads, as given in Table 18 of the code (p. 68) are erroneous. Table 18 of the code gives,

$$U = (1.5\ (DL + LL) \quad (19.1)$$

$$U = (1.5 \text{ or } 0.9\ DL + 1.5\ WL) \quad (19.2)$$

$$U = 1.2\ (DL + LL + WL) \quad (19.3)$$

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 × 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.

ii) In analysis of structures, effect of slab on the moment of inertia of beams should be considered for the sake of realistic behaviour of reinforced concrete structures. Clause 22.3 of the code is silent on this aspect.

iii) 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 of 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 of the code (clause 23.2b) that the partial deflection after the erection of partitions and finishes including the effects of 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 requirement 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 of deflection of $L/250$ or $L/350$ is valid for these systems. This is the correct position but the code has not made this point clear.

iv) 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 mid-span 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.

v) 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 corner 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.

vi) 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 of 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.

vii) 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.

viii) Fig. 4 of IS: 456-2000 (p. 38) gives some additional values of f_s = 120, 145, etc. and these will be useful when $f_s < 0.58\,f_y$, 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)

$$MF = \frac{1}{\left[0.225 + 0.00322 f_s - 0.625 \log\left(\frac{(bd)}{100 A_{st}}\right)\right]}$$

where,

$$f_s = 0.58 f_y \times \frac{A_{st}\,(\text{reqd})}{A_{st}}$$

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.

ix) Clauses 25.12 and 39.7 of IS: 456-2000 give design of 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 of 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.

Appendix 1

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Appendix II

USEFUL TABLES: CHAPTER-WISE

In this Appendix the design engineer will find some useful information for the design of multistoreyed buildings. The first figure of the table relates to the chapter for which the information is relevant.

Table AII.1.1 Dead load: unit weights of some building materials.

S. No.	Material	Unit wt kg/m^3	S. No.	Material	Unit wt kg/m^3
1.	Coarse aggregate	1800	21.	Terra cotta	1900
2.	Fire aggregate (sand)	1600	22.	Timber	850
3.	Bitumen	1040	23.	Water, fresh	1000
4.	Bridis	1900	24.	Water, saline	1025
5.	Brick dust (Surkhi)	1010	25.	Aluminium	2800
6.	Cement	1440	26.	Copper	8960
7.	Plain cement concrete	2000	27.	Gold	19330
8.	Reinforced cement concrete	2500	28.	Iron	7700
9.	Foam concrete	1200	29.	Lead	11000
10.	Cinder fill	1000	30.	Silver	10550
11	Glass	2500	31.	Steel	7850
12.	Gypsum mortar	1200	32.	Ice	910
13.	Cement mortar	2080	33.	Cotton, compressed	1300
14.	Lime mortar	1760	34.	Jute in bundles	700
15.	Dry earth	1800	35.	Paper in bundles and rolls	700
16.	Moist earth	2000	36.	Newspaper bundles	400
17.	Fine sand dry	1600	37.	Thread in bundles	500
18.	Fine sand	2080	38.	Wooed, compressed	1300
19.	Stone	2400	39.	Rubber	1400
20.	Tar, crude	1010	40.	Coal	1000

Table AII.1.2. Live loads or floors and roofs.

S. No.	Type of floor or roofs	Min. live load kg/m^2	S. No	Type of floor or roofs	Min. live load kg/m^2
1.	Floors in dewelling houses, tenements, hospital wards, hostels, dormitoris	200	8	Floors of garages for light vehicles not exceeding 2.5t gross weight	400
2.	Office floors other than entrance halls	250-400	9.	Floors of garages for heavy vehicles not exceeding 4.0t gross weight	750
3.	Floors of banking halls, office entrance halls and reading rooms, floor of class rooms	300	10.	Stairs, landing, corridors, balconies for dewelling a) not liable to overcrowding, b) liable to some overcrowding, c) liable to large overcrowding	 300 400 500
4.	Shop floors, floors of work rooms, floors of assembly with fixed seating, resturants.	400	11.	Roops – flat, sloping or curved – with slope upto 10° a) access provided b) access not provided	 150 75
5.	Floors of ware houses, workshops, floors of assembly with fixed seating, public rooms in hotels, dance halls, waiting halls	500	12.	Horizontal loads on parapets and balustrades a) not liable to overcrowding b) liable is overcrowding	 75 225
6.	Floor of ware houses, workshops, factories for medium weight loads	700	13.	Floors supporting moving vehicles (a) cars, tempos (b) trucks (c) fire tender	 750 1500 2250
7.	Floors of ware houses, workshops, factories heavy weight loads, for floors of book stores, libraries	1000	14.	Surcharge loads due to a) pedestrain traffic b) vechicular traffic c) fire brigade traffic	 500 1000 1500

Table AII.1.3. Basic wind speed (V_b) at 10m height for some important places in India and also basic horizontal seismic coefficient (α_0).

Place	V_b (m/s)	α_0	Place	V_b (m/s)	α_0
Agartala	55	0.08	Jodhpur	47	0.01
Agra	47	0.04	Jorhat	50	0.08
Ahemdabad	39	0.04	Kanpur	47	0.04
Aizwal	55	0.08	Kohima	44	0.08
Ajmer	47	0.01	Kurnool	39	0.01
Almora	47	0.05	Lakshadweep	39	0.04
Ambala	47	0.05	Leh	55	0.04
Amritsar	47	0.05	Lucknow	47	0.04
Asansol	47	0.04	Ludhiana	47	0.05
Aurangabad	39	0.01	Madras	50	0.02
Bahraich	47	0.05	Madurai	39	0.02
Bangalore	33	0.01	Mandi	39	0.08
Barauni	47	0.05	Mangalore	39	0.04
Bareilly	47	0.04	Monghyr	47	0.05
Bhatinda	47	0.04	Moradabad	47	0.05
Bhilai	39	0.01	Mysore	33	0.01
Bhopal	39	0.02	Nagpur	44	0.02
Bhubaneshwar	50	0.04	Nainital	47	0.05
Bhuj	50	0.08	Nasik	39	0.04
Bikaner	47	0.04	Nellore	50	0.02
Bokaro	47	0.04	Panjim	39	0.04
Bombay	44	0.04	Patiala	47	0.04
Burdwan	47	0.04	Patna	47	0.05
Calcutta	50	0.04	Pondicherry	50	0.02
Calicut	39	0.04	Port Blair	44	0.08
Chandigarh	47	0.05	Pune	39	0.04
Coimbatore	39	0.04	Raipur	39	0.01
Cuttack	50	0.04	Rajkot	39	0.04
Darbhanga	47	0.08	Ranchi	39	0.02
Darjeeling	47	0.05	Roorkee	39	0.05
Dehradun	47	0.05	Rourkela	39	0.01
Delhi	47	0.05	Silchar	55	0.08
Durgapur	47	0.04	Simla	39	0.05
Gangtok	47	0.05	Srinagar	39	0.08
Gauhati	50	0.08	Surat	44	0.04
Gaya	39	0.04	Tezpur	50	0.08
Gorakhpur	47	0.05	Thiruchirappalli	47	0.02
Hyderabad	44	0.01	Trivandrum	39	0.04
Imphal	47	0.08	Udaipur	47	0.02
Jabalpur	39	0.04	Vadodara	44	0.04
Jaipur	47	0.02	Varanasi	47	0.04
Jamshedpur	47	0.02	Vijayawada	50	0.04
Jhansi	47	0.01	Visakhapatnam	50	0.02

Table AII.1.4. Strenghtening arrangements recommended for masonry buildings.

Building category	No. of storeys	Strengthening arrange-ments provided in all storeys
A (α_h = 0.04 < .05)	(i) 1 to 3 (ii) 4	a a, b, c
B (α_h = .05 to .06)	(i) 1 to 3 (ii) 4	a, b, c, f, g a, b, c, d, f, g
C (α_h > .06 < .08)	(i) 1 and 2 (ii) 3 and 4	a, b, c, f, g a to g
D (α_h = .08 < .12)	(i) 1 and 2 (ii) 3 and 4	a to g a to h
E (α_h = .12 > .12)	(i) 1 to 3	a to h

Notes 1. Strengthening arrangements.

a) masonry mortar; b) lintel band; c) roof band; d) vertical steel at corners and junction of walls; e) vertical steel at joints of openings; f) bracing in plan at thelevel of roofs; g) plinth band; h) dowel bars
2. = $\alpha_0.I.\beta$.

Table AII.1.5 Unit weights of stored materials and their angles of repose.

S.No.	Material	Unit weight kg/m^3	Angles of repose $\Phi°$
1.	Grain, corn, barley	800	27°
2.	Flour (grain)	600	40°
3.	Refined sugar	1000	35°
4.	Concrete aggregate dry wet	 1800 2000	 35° 25°
5.	Lime hydrated powder	700	35°
6.	Cement clinker	1400	33°
7.	Cement portland	1600	25°
8.	Earth dry	1500	30°
9.	Sand dry sand moist sand saturated	1600 1800 2000	35° 40° 25°
10.	Coal	900	40°
11.	Potatoes	750	30°
12.	Coke	600	40°

Table AII.1.6. Factor of safety for various load effects.

S.No.	Load effect	Factor of safety	Reference code
1.	Uplift	1.5	Fintel
2.	Overturning	2.00	IS: 1904
3.	Sliding	1.75	IS: 1904
4.	Safe bearing capacity of soil	2.5 to 3.0	IS: 1904
5.	Dead Load	1.5	IS: 456
6.	Live Load	1.5	IS: 456
7.	Earthquake or wind along with other loads	1.2	IS: 456
8.	Blast alongwith other loads	1.5	Varyani
9.	Concrete (f_{ck}/γ_m)	$\gamma_m = 1.5$	IS: 456
10.	Steel (f_y/γ_s)	$\gamma_s = 1.15$	IS: 456

Table AII.2.1. Optimum structural systems for RC buildings.

No of storeys	Structural systems
1 to 4	Load bearing brick walls with RC columns where required
5 to 12	RC framed structure
13 to 20'	RC frames combined with shear walls
21 to 30	RC core with frames and other shears walls if required
31 to 40	RC core in the interior combined with framed tube on the periphery

Table AII.3.1. Practical depth (*D*) bending members for moment, deflection, shear and economy.

Type of member and structural materials	Description of member	Depth (*D*)	Governing limit state
Beams M20 Fe415	Cantilever beams	$l/7$ to $l/5$	Economy
	Simply supported beams	$l/12$ to $l/10$	Economy
	Continuous beams	$l/12$ to $l/15$	Economy

Solid slabs	Cantileaver slabs	$l/10$	Deflection
M20	Simply supproted one-way slabs	$l/25$	Deflection
Fe415	Continuous one-way slabs	$l/30$	Deflection
	Simply supported two-way slabs	$l/30$	Deflection
	Continuous two-way slabs	$l/35$	Deflection
Flat plates M20 Fe415 $l = \frac{1}{2}(l_1 + l_2)$	Flat plate, i.e. slab without drop and without column capital	$l/28$	Deflection (shear heads to be provided or column capital to be privided for shear)
Flat slabs M20 Fe415	Flat slabs with drops and columns with capital	$D = l/35$	Deflection
$l = \frac{1}{2}(l_1 + l_2)$	Drop thickness = D_0	$D_0 = 1.5D$	Shear
Grids or waffle slabs	Simply supported two-way grid panels	$l/25$	Deflection
M20 Fe415 $l = \frac{1}{2}(l_1 + l_2)$	Flat-waffle slab with solid drops and columns with or without capitals	$l/20$	Deflection and shear

Table AII.4.1. Thickness (D) of stairs slabs of diffferent types.

Stairs type	D	Governing limit state
Dog-legged simply supported stairs slab	$l/30$	moment
Central-wall type cautileq stairs slab	$l/10$	moment
Central-column type spiral stairs slab	$l/10$	moment
Slabless or saw-tooth type stairs slab	$l/30$	moment
Helicoidal stairs slab of radius R in plan	$R/D = 9$	moment, shear, torsion
Free-standing stairs	$(L + W)/D = 25$	moment, shear, torsion

Table AII.5.1. Basic compressive stress in kg/cm^2 in masonry short walls (.75$H/t \leq 6$) for zero eccentricity of axial load.

S.No.	Concrete mortar	Crushing strength of bricks (kg/cm^2)				
		50	75	100	150	200
1.	1:3	5.0	7.5	10.0	12.5	16.5
2.	1:41/2	5.0	7.4	9.6	11.5	14.5
3.	1:6	5.0	7.4	9.6	11.0	13.0

Table AII.5.2 Reduction factors for slendress ratio and eccentricity of loading.

Slendness ratio (.75H/t)	Equivalent eccentricity of loading divided by thickness of walls						
	0	0.04	0.10	0.2	0.3	0.33	0.50
6	1.0	1.0	1.0	0.996	0.984	0.980	0.97
8	0.92	0.92	0.92	0.91	0.88	0.87	0.85
10	0.84	0.835	0.83	0.81	0.77	0.76	0.73
12	0.76	0.75	0.74	0.706	0.664	0.65	0.60
14	0.67	0.66	0.64	0.604	0.556	0.540	0.48
16	0.58	0.565	0.545	0.556	0.44	0.42	0.35
18	0.50	0.48	0.45	0.44	0.324	0.30	0.23
21	0.47	0.448	0.42	0.354	0.276	0.25	0.17
24	0.44	0.415	0.38	0.31	0.22	0.19	0.11

Table AII.6.1. Moments in columns.

Condition	Moments for frames of one bay	Moments for frames of two or more bays
External (and similarly loaded) columns:		
Moment at foot of upper column	$M_e \dfrac{K_u}{K_l + K_u + 0.5 K_b}$	$M_e \dfrac{K_u}{K_l + K_u + K_b}$
Moment at head of lower column	$M_e \dfrac{K_u}{K_l + K_u + 0.5 K_b}$	$M_e \dfrac{K_u}{K_l + K_u + K_b}$
Internal columns:		
Moment at foot of upper, column	–	$M_{es} \dfrac{K_u}{K_l + K_u + K_{b1} K_{b2}}$
Moment at head of lower column	–	$M_{es} \dfrac{K_u}{K_l + K_u + K_{b1} K_{b2}}$

Note 1:
M_e = bending moment at the end of the beam framing into the column assuming fixity at the connection,
M_{es} = maximum difference between the moments at the end the two beams framing into opposite sides of the column, each related on the assumption that the neds of the beams are fixed assuming one of the beams unlocated,
K_u = stiffness of the upper column,
K_l = stiffness of the lower column,
K_b = stiffness of the beam,
K_{b1} = stifness of the beam on one side of the column, and
K_{b2} = stifness of the beam on the other side of the column.

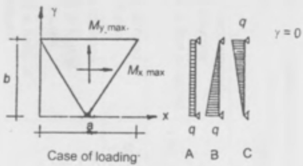
Note 2: For the purpose of this table, 'stiffness' of a member may be obtained dividing the moment of intertia of a cross-section by the length of the member provided that the member is of constant cross-section throughout its length.

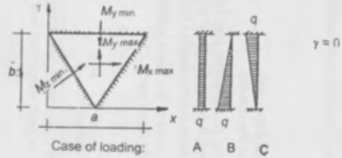
Note 3: The equation for the moment at the head of the lower column may be used for columns in a topmost storey by taking K_u as zero

Table AII.6.2. Approximate column load in buildings.

Location of column	loading intensity over tributary floor area at each supporting level
Corner columns	2.5 t/m^2
Exterior columns	2.0 t/m^2
Interior columns	1.5 t/m^2

Table 10.1. Bending moments in simply supported and totally fixed trianangular plates due to rectangular and triangular loads.





	a) Simply Supported Triangular Plate						b) Totally fixed triangular plate											
	Case A		Case B		Case C		Case A				Case B				Case C			
a/b	M_x max	M_y max	M_x max	M_y max +ve	M_x max +ve	M_y max +ve	M_x max +ve	M_y max +ve	M_y max ve	M_y max ve	M_x max +ve	M_y max +ve	M_y max -ve	M_z max -ve	M_y max +ve	M_y max +ve	M_y max ve	M_y max -ve
0.5	.0396	.0209	.0125	.0027	.0271	.0182	.0187	.0089	.0210	.0378	.0046	.0007	.0036	.0085	.0141	.0082	.0174	.0293
0.6	.0393	.0197	.0117	.0030	.0226	.0167	.0169	.0098	.0219	0355	.0053	.0022	.0042	.0087	.0116	.0076	.0177	.0248
0.7	.0343	.0186	.0108	.0032	.0195	.0154	.0150	.0100	.0220	.0297	.0053	.0028	.0044	.0087	.0097	.0072	.0176	.0210
0.8	.0200	.0178	.0100	.0035	.0170	.0143	.0131	.0096	.0215	.0264	.0048	.0028	.0046	.0086	.0083	.0068	.0169.	.0178
0.9	.0271	.0172	.0091	.0038	.0150	.0133	.0114	.0089	.0206	.0234	.0043	.0024	.0046	.0081	.0071	.0065	.0160	0.153
1.0	.0214	.0166	.0084	.004'	.0130	.0125	.0098	.0082	.0196	.0207	.0034	.0020	.0046	.0071	.0062	.0062	.0150	.0136
1.1	.0192	.0161	.0078	.0044	.0114	.0117	.0087	.0076	.0184	.0183	.0034	.0018	.0044	.0063	.0053	.0058	.0140	.0120
1.2	.0172	.0154	.0072	.0044	.0100	.0110	.0077	.0071	.0173	.0162	.0031	.0017	.0043	.0056	.0046	.0054	.0130	.0106

Table 10.1. *Contd.* Bending moments in simply supported and totally fixed trianangular plates due to rectangular and triangular loads.

1.3	.0155	.0145	.0067	.0041	.0088	.0104	.0069	.0067	.0163	.0145	.0029	.0017	.0042	.0051	.0040	.0050	.0121	.0094
1.4	.0141	.0135	.0063	.0038	.0078	.0097	.0061	.0063	.0153	.0131	.0026	.0017	.0041	.0048	.0035	.0046	.0112	.0083
1.5	.0128	.0126	.0058	.0035	.0070	.0091	.0055	.0059	.0144	.0120	.0024	.0017	.0041	.0046	.0031	.0042	.0103	.0074
1.6	.0118	.0118	.0055	.0033	.0063	.0085	.0049	.0056	.0136	.0111	.0021	.0017	.0041	.0044	.0028	.0038	.0095	.0067
1.7	.0108	.0111	.0051	.0031	.0057	.0080	.0044	.0053	.0128	.0103	.0019	.0017	.0040	.0042	.0025	.0035	.0088	.0061
1.8	.0199	.0105	.0047	.0030	.0052	.0075	.0039	.0050	.0120	.0096	.0017	.0017	.0038	.0040	.0022	.0033	.0082	.0056
1.9	.0090	.0099	.0042	.0029	.0048	.0070	.0034	.0048	.0112	.0091	.0015	.0017	.0035	.0039	.0019	.0031	.0077	.0052
2.0	.0081	.0094	.0036	.0028	.0045	.0066	.0029	.0045	.0103	.0087	.0013	.0026	.0025	.0038	.0016	.0029	.0072	.0049

Bending Moments = coeff. qa^2

Table AII.11.1. Safe bearing capacity of soils.

S. No	Type of soil	Safe bearing capacity (t/m^2)	Remarks
(a)	Rocks		
1	Rocks – hard without lamination and defects, for example, granite, trap and diorite	330	–
2	Laminated rocks, for example, sandstones and limestones in sound condition	165	–
3	Residual deposits of shattered and broken bed rock and hard shale, cemented material	90	–
4	Soft rock	45	
(b)	Non-cohesive soils (use half the values given below if water table is above or near the bearing surface of the soil.		
5.	Gravel, sand and gravel, compact and offering high resistance to penetration when excavated by tools	45	–
6.	Coarse sand, compact and dry	45	Dry means theground water level is at a depth not less than the width of foundation below the base of the foundation
7.	Medium sand, compact and dry	25	–
8.	Fine sand, silt (dry lamps easily pulverized figures)	15	–
9.	Loose gravel or sand gravel mixture, loose coarse to medium sand, dry	25	–
10.	Fine sand, loose and dry	10	–
(c)	Cohesive soils		
11.	Soft shale, hard or silt clay in deepbed, dry	45	This group is susceptible to long term consolidation settlement
12.	Medium clay, readily indented with a thumb nail	25	–
13.	Moist clay and sand dry mixture which can be indented with strong pressure	15	–

14.	Soft clay indented with moderate thumb pressure	10	–
15.	Very soft clay which can be penetrated several inches with the thumb	5	–
16.	Black cotton soil or other shrinkable or expansive clay in dry condition (50 % saturation)	–	Use under reamed piles or remove the soil entirely if practicable
17.	Peat	–	Use piles or remove soil entirely if practicable
18.	Made-up ground	–	Use sand piles, stone columns and other methods for improving soil capacity

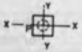
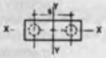
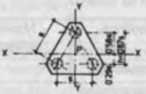
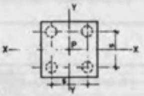
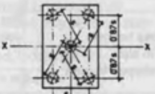
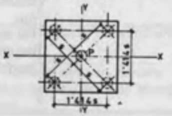
Table AII.11.2. Maximum total settlement for footings.

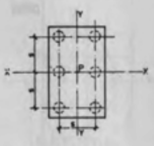
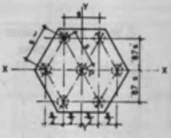
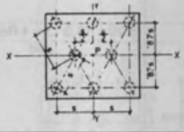
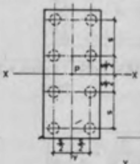
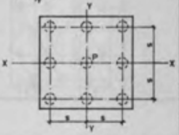
S.No.	Type of footings	Maximum total settlement (mm)
1.	Isolated footing on clay	75
2.	Isolated footing on sand	50
3.	Raft foundation on clay	100
4.	Raft foundation on sand	75
5.	Piles	12

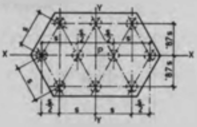
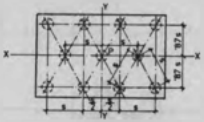
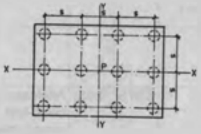
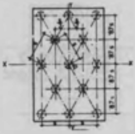
Table AII.11.3. A list of footing types.

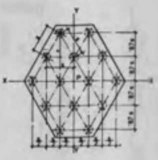
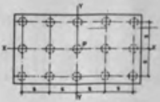
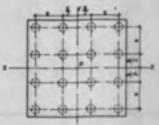
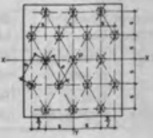
S.No.	Footing types	Application
1.	Isolated footing of a) uniform depth b) sloping depth c) stepped depth	For a single column
2.	Combined footing of a) uniform depth with hidden cross beams b) longitudinal inverted beam with slab c) rectangular, trapezoidal or T-shaped in plan d) strap beam type	For two columns
3.	Strip footing of beams-slab type	For a number of columns in a row
4.	Raft foundation of a) slab type b) beam and slab type c) annular in plan d) cellular in section	For a cluster of columns randomly placed

Table AII.11.4. A table of pile groups.

No. of pile	Pile layout	Z_{xx}	Z_{yy}	Remarks
1.		0	0	Incapable of taking moments either way
2.		0	1.0S	Incapable of taking moment about *x-x* axis
3.		1.74S	1.0S	Capable of taking moments both ways
4.		2.0S	2.0S	2 × 2 Rectangular grid
5.		3.48S	2.0S	This is closer knit than the following arrangement
5.		2.828S	2.828S	Alternate symmetrical arrangement

6.		3.0S	4.0S	3 × 2 Rectangular grid
7.		3.48S	3.00S	
8.		5.22S	4.50S	
8.		6.67S	4.0S	Alternate more close-knit arrangement 4 × 2 Rectangular grid
9.		6.0S	6.0S	3 × 3 Rectangular grid

10.		5.22S	6.0S	8 Piles + 2 more piles
11.		6.96S	8.0S	7 piles + 4 more piles
12.		8.0S	10.S	3 × 4 Rectangular grid
13.		12.18S	7.0S	11 piles + 2 more piles

14.		12.18S	6.67S	8 piles + 6 more piles
15.		10.0S	15.0S	3 × 5 Rectangular grid
16.		13.33S	13.33S	4 × 4 Rectangular grid
17.		13.92S	10.67S	13 Piles + 4 more piles

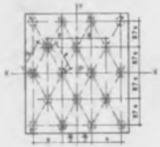
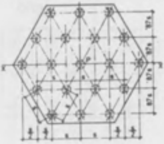
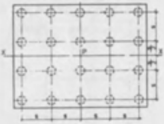
18.		16.53S	12.67S	14 Piles + 4 more piles
19.		13.92S	12.0S	17 Piles + 2 more piles
20		16.67S	20.0S	4 × 5 Rectangular grid

Table AII.11.5. Guide to selection of foundation types.

Structure Type	Site conditions			
	Deep firm bed	Firm bed over lying soft bed	Deep soft bed	Soft bed over lying firm bed
Light, flexible structure	Strip	Strip or raft	Friction piles or raft	Bearing piles or piers
Heavy, rigid structure	Strip	Raft or friction piles	Raft or friction piles	Bearing piles or piers

Table 12.1 Bending moment coffecients for rectangular panels supported on four sides with provision for torsion at corners

Case No.	Type of panel and moments considered	Short span coefficients α_x (values of l_y/l_x) 1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	Long span coefficients α_x for all values of l_y/l_x
1.	Interiour Panels:									
	Negative moment at continuous edge	0.032	0.037	0.043	0.047	0.051	0.053	0.060	0.065	0.032
	Positive moment at mid-span	0.024	0.128	0.032	0.036	0.039	0.041	0.045	0.049	0.024
2.	One Short Edge Discontinuous:									
	Negative moment at continuous edge	0.037	0.043	0.048	0.051	0.055	0.057	0.064	0.068	0.037
	Positive moment at mid-span	0.028	0.032	0.036	0.039	0.041	0.044	0.048	0.052	0.028
3.	One Long Edge Dsicontinuous:									
	Negative moment at continuous edge	0.037	0.044	0.052	0.057	0.063	0.067	0.077	0.085	0.037
	Positive moment at mid-span	0.028	0.033	0.039	0.044	0.047	0.051	0.059	0.065	0.028
4.	Two Adjacent Edges Discontinuous:									
	Negative moment at continuous edge	0.047	0.053	0.060	0.065	0.071	0.075	0.084	0.091	0.047
	Positive moment at mid-span	0.035	0.040	0.045	0.049	0.053	0.056	0.063	0.069	0.035
5.	Two Short Edges Discountinus:									
	Negative moment at continuous edge	0.045	0.049	0.052	0.056	0.059	0.060	0.065	0.069	–
	Positive moment at mid-span	0.035	0.037	0.040	0.043	0.044	0.045	0.049	0.052	0.035

Table 12.1. (Contd.) Bending moment coffecients for rectangular panels supported on four sides with provision for torsion at corners.

6.	Two Long Edges Discontinuous:									
	Negative moment at continuous edge	–	–	–	–	–	–	–	–	0.045
	Positive moment at mid-span	0.035	0.043	0.051	0.057	0.063	0.068	0.080	0.088	0.035
7.	Three Edges Discountinuous (One Long Edge Continuous):									
	Negative moment at continuous edge	0.057	0.064	0.071	0.076	0.080	0.084	0.091	0.097	–
	Positive moment at mid-span	0.043	0.048	0.053	0.057	0.060	0.064	0.069	0.073	0.043
8.	Three Edges Discountinuous (One Short Edge Continuous):									
	Negative moment at continuous edge	–	–	–	–	–	–	–	–	0.057
	Positive moment at mid-span	0.043	0.051	0.059	0.065	0.071	0.076	0.087	0.096	0.043
9.	Four Edges Discountinuous:									
	Positive moment at mid-span	0.55	0.064	0.072	0.079	0.085	0.089	0.100	0.107	0.056

Table AII.12.2. Effective length of compression members.

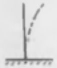
S. No.	Degree of end restraint of compression member	Symbol	Theoretical value of effective length	Recom-mended value of effective length
	(1)	(2)	(3)	(4)
1.	Effective held in position and restrained against rotation at both ends		0.5*l*	0.65*l*
2.	Effective held in position at both ends, restrained against rotation at one end		0.7*l*	0.80*l*
3.	Effective held in poistion at both ends, but not restrained against rotation			1.00*l*
4	Effective held in position and restrained against rotation at one end, and at the other restrained against rotation but not held in position			1.20*l*
5.	Effective held in position and restrained against rotaion at one end, and at the other partially restrained against rotation but not held in position			1.50*l*
6	Effective held in position at one end but not restrained against rotaion, and at the other and restrained against rotation but not held in position		2.00*l*	2.00*l*
7.	Effective held in position and restrained against rotaion at one end but not held in position nor restrained against rotation at the other end		2.00*l*	2.00*l*

Table AII.13.1. Development length of bars for different steel type and different concrete mixes.

$$\left(L_d = \frac{\phi\, \sigma_s}{4\, \tau_{bd}}\right),\ \sigma_s = 0.87\, f_y$$

= $(\phi.k)$, rounded off to the nearest integer

Type of steel	Value of k for concrete mixes				
	M20	M25	M30	M35	M40
Fe250 (plain bars)	45	39	36	32	29
Fe415 (deformed bars)	47	40	38	33	30
Fe500 (deformed bars)	57	49	45	40	36
Fe550 (deformed bars)	62	53	50	44	39

Table AII.14.1. Percentage cost of materials in buildings.

S.No.	Material	$\frac{\text{cost of material}}{\text{total cost of building}} \times 100$	Remarks
1.	Steel bars	15	
2.	Cement	13	
3.	Bricks	11	
4.	Sand	3	
5.	Coarse aggregate	3	
6.	Stone slabs	1	for kitchen top, stairs top, toilet floors, etc.
7.	Timber	6	
8.	Glass	11/2	
9.	Hardware	1/2	
10.	Labour	16	for civil works only
11.	Electrical works	20	including wiring, fitting and labour
12.	Plumbing works	10	including piping, fittings and labour
	Total	100	

Table AII.17.1. Spacing of expansion joints in metres in long concrete buildings.

Storey height (m)	Shape of building in plan	Column stiffness in long direction	Climate	
			Moderate ($t = -15 \pm 17$) $= -32°C$	Extreme ($t = -15 \pm 25$) $= -40°C$
3.0	Square (L/B = 1.0)	Rigid ($\alpha_1 < 2$)	20	15
		Flexible ($\alpha_1 \geq 2$)	25	20
	Rectangule (L/B = 2.0)	Rigid	30	25
		Flexible	35	30
	Long rectangule (L/B = 3.0)	Rigid	35	30
		Flexible	40	30
4.0	Square	Rigid	35	30
		Flexible	40	30
	Rectangule	Rigid	45	35
		Flexible	50	40
	Long rectangule	Rigid	50	40
		Flexible	60	50
5.0	Square	Rigid	45	35
		Flexible	55	45
	Rectangule	Rigid	55	45
		Flexible	65	50
	Long rectangule	Rigid	60	50
		Flexible	70	55
6.0	Square	Rigid	60	50
		Flexible	70	55
	Rectangule	Rigid	70	55
		Flexible	80	65
	Long rectangule	Rigid	80	65
		Flexible	90	70

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