Illustrated Design of Reinforced Concrete Buildings

(Design of G+3 Storeyed Buildings + Earthquake Analysis & Design)

Eighth Edition

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Dr. S. R. Karve

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Design of G+3 Storeyed Buildings and Earthquake Analysis and Design.

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Eighth Edition

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Preface to the First Edition

It gives the authors great pleasure and deep sense of satisfaction in presenting this book dealing with "Illustrated Design of Reinforced Concrete Buildings ". This Book is an outcome of persistent demand from Students, Practising Engineers and Building Designers. To fulfill the immediate need of Students, Teachers, the Authors have already published the Text Book, "Limit State Theory & Design of Reinforced Concrete" which is a nucleus to Limit State Theory. We suggest the readers to scan through this book to know the fundamental aspects of the Limit State Method. This would facilitate to study the design of Multistoreyed Buildings. It is the propose of this book to attempt to explain the basic principles and the method of Design of different types of buildings.

In Developing Cities, the Municipal Authorities do not give permission to construct high rise buildings to avoid congestion, pollution etc. They only give permission to construct ground plus two *or* three storeyed buildings. Hence, the authors have limited the design details for ground plus three storey structures.

First three Chapters are devoted to explain the need and the method of approach to structural planning, properties of constituent materials of reinforced concrete, critical load combinations, and fundamental aspects of structural analysis of residential / office buildings. It is suggested that the readers to go through these three Chapters so that further chapters on Design of various structures will become elucidative.

Chapter IV reviews the basic Limit State Theory while the procedure for Design of Structural Components is given Chapter V.

Three projects have been included, illustrating three different types of buildings. A single storey public building designed in details from first principles in Chapter VI. A public building having regular layout and which can be divided into a number of similar vertical plane frames, has been illustrated in Chapter-VII. The Chapter - VIII gives Design of Residential Building using Design aids.

At the end exhaustive Appendices have been given which include important design tables, Charts, Design aids, so that design of building can be done without further reference to any other Hand Book.

While writing this book we had extensive discussions with the practising design engineers, so that this book does not remain a theoretical model but a useful work which can assist practising engineers involved in the design of buildings.

In spite of meticulous care taken in writing this book some errors might have crept in the authors would highly appreciate if these are brought to our notice.

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Preface to Fifth Edition

Due to revision of IS:456, all the text , the projects, and all other chapters have been totally revised conforming to IS:456-2000. Even though the projects are the same, some useful comments, different practical methods used, have been included taking into account the latest practices. Further, in order that the structure should behave as designed and should not mar the appearance of the exposed surface due to excessive cracking, good detailing of a structure is of prime importance. Considering this aspect the original chapter on procedure for design of components has been replaced by, "Structural behavior of R.C. elements and detailing of reinforcement".

Recent earthquakes have perished many villages, damaged some urban areas, partially or totally damaged the structure. Loss of life, damage to structure can be reduced once the cause of earthquake and its action is understood. Keeping this in mind chapter on "Earthquake Analysis and Design", has been added. It commences explaining what is earthquake? methods of its measurement., behavior of a structure during earthquake, seismic analysis of forces acting on the structure, and the method of design has been included. Since damage controlled Limit state has been accepted as design method. Details of ductile detailing have been included to prevent brittle failure of structure.

Preface to the Sixth Edition

An example of three storeyed , single bey , commercial building with infilled masonry walls has been solved using equivalent static method and response spectrum method of dynamic analysis . The dynamic analysis includes *Absolute Sum Method* (*ABS*) , Square root of *Sum of Square Method* (*SRSS*) , *Complete Quadratic Combination Method* (*CQC*) . The example illustrates detailed procedure for dynamic analysis using response spectrum method for determination of lateral forces . The aim of giving the worked out example is to present a clause wise approach for determination of lateral forces as per IS: 1893 , so as to give insight in the method of design to enable one to prepare the programs for multistoreyed buildings.

The author thanks Prof. Mrs .Suhasini N. Madhekar for working out the design example using Response Spectrum Method .

Preface to the Eighth Edition

I have the feeling that once the basic theory is understood, it is easy to analyse any problem in practice. To achieve that the basic design principles contained in Chapters 1 to 6 have been revised adding plenty of new matters and 3-D figures for ease of understanding. Time saving analysis and design aids in the form of Tables and Charts, and formulae for B.M.,S.F.,deflection have been included for use in design. Text has been updated incorporating amendments 1 to 4 to IS:456-2000 , IS: 1893(part-1) - 2016 and IS: 13920 - 2016.

I take the opportunity to thank the consulting engineers, contractors, professors, and the students for their magnificent response and valuable suggestions.

The errors which had inadvertently crept in, in the subject matter and in figures have been rectified. The subject matter,its format, and presentation sequence, will prove to be dependable companion for teachers, students,and practsing engineers.

All efforts have been made to ensure the correctness of analysis and design. In spite of this it is inevitable that some errors or misprint may be found. I will be grteful to the readers conveying the same to me.

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APPENDICES

CHAPTER - 1

INTRODUCTION TO STRUCTURAL DESIGN

1.1 THE DESIGN PROCESS

Engineering is a professional art of applying science to the efficient conversion of natural resources for the benefit of mankind. Engineering, therefore, requires above all creative imagination to innovate useful application for natural phenomenon.

The entire process of structural planning and design requires not only imagination and conceptual thinking but also sound knowledge of science of structural engineering besides knowledge of practical aspects, such as recent design codes and byelaws, backed up by ample experience, intuition, judgment. and keen observation.

It may be clarified that Code of practice, which is compendia of good practices drawn up by experienced engineers, is intended to guide engineers and should never be allowed to replace the conscience and competence of the engineer. The purpose of standards is to ensure and enhance the safety, keeping careful balance between economy and safety.

It is emphasized that any structure to be constructed must satisfy the need efficiently for which it is intended and shall be durable during its desired life span.

The process of design commences with planning of the structure, primarily to meet its functional requirements. The design of any structure is categorized into the following two main types.

Design

Functional Design[1.1] *Structural Design.*

`1.1.1 Functional Design

(1) The structure to be constructed should primarily serve the basic purpose for which it is to be used.

The building should provide happy environment inside as well as outside. Therefore, the functional planning of a building must take into account the proper arrangements of rooms/halls to satisfy the need of the client, good ventilation, lighting, acoustics, unobstructed view in the case of community halls, cinema theatres etc., sufficient head room, proper water supply and drainage arrangements, planting of trees etc.

(2) Decide the type of structure.

Bearing all the above aspects in mind, the architect/engineer (*i.e.* designer) has to decide whether it should be a load bearing structure or R.C.C. framed structure or a steel structure. He should settle the system of covering the structure, whether the roof shall consist of steel roof trusses and girders *or* R.C.C. folded plates *or* R.C. shell *or* a beam-slab construction *or* a grid system *or* a prestressed concrete hanging roof *or* combination of above.

Since the scope of this book is restricted to R.C. Buildings, the discussions are limited to design of R.C. framed structures.

After deciding the tentative form of the structure the designer should select appropriate material for its construction. The properties of the available materials have to be determined to decide their suitability. Sometimes some material may be required to be imported due to which the cost may go high and may require change in the form selected. All these aspects are inter linked and final decision has to be taken considering requirements of the user, functional aspect, aesthetics and cost.

(3) It must satisfy the purpose for which it is constructed.

For example, in the case of assembly hall or cinema balcony, they are required to be supported at the ends only. In such a case the span may be very large and the depth of the supporting beam may work out to be very large, still the columns should be provided at the ends only so that the sight of the viewers is not obstructed.

(4) It must meet the requirements of the user.

The requirements proposed by the client should be taken into consideration. They may be vague, ambiguous or even unacceptable from engineering point of view because he is not aware of the various implications involved in the process of planning and design, and about the limitations and intricacies of the structural science. However, he should be convinced about his wrong views and then work should be undertaken.

(5) It must have a pleasant look and the aspects of aesthetics must be looked into.

1.1.2 Structural Design

Once the form of the structure is selected the structural design process starts.

Structural Design is an art and science of understanding the behavior of structural members subjected to loads and designing them with economy and elegance to give a safe, serviceable and durable structure.

The structural details of a R.C. building frame structure are shown in Fig.1.1.2

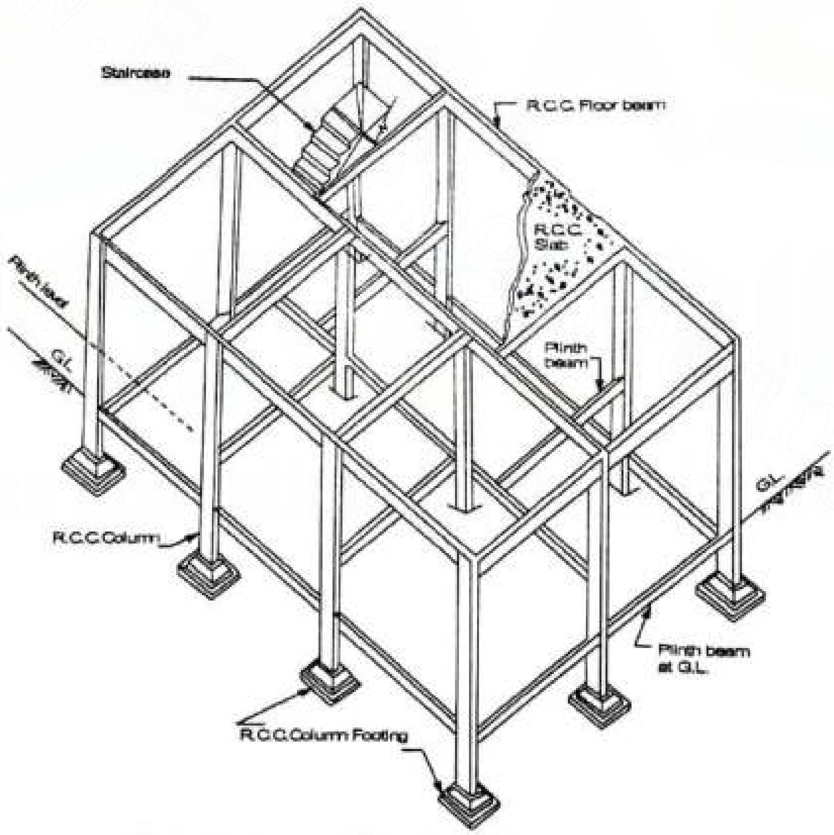

Fig.1.1.2 Structural Details of Framed Structure

The principal elements of a R.C. building frame consists of :

(i) Slabs to cover large area, (ii) Beams to support slabs and walls, (iii) Columns to support beams, (iv) Footings to distribute concentrated column loads over a large area to the supporting soil such that the bearing capacity of soil is not exceeded.

In a frame structure the load is transferred from slab to beam, from beam to column and then to the foundation and soil below it.

Comments: *A framed structure implies that first the frame should be erected and then the walls should be constructed. It is a wrong practice to cast the floor beam on the wall thereby saving in bottom formwork and further reducing the mid - span steel. In such a case some load from the floor beam is bound to get transferred to the wall and the system does not act as a framed structure in true sense.*

1.2 STAGES IN STRUCTURAL DESIGN

Once the form of the structure is selected the structural design process starts.

The process of Structural Design involves the following stages :

STAGES IN STRUCTURAL DESIGN

- **Computation of Design Loads**
- **Structural Planning**
- **Analysis**
- **Element Design**
- **Drawing, Detailing, Estimating**

The most important stage in design is computation of design loads, supporting conditions etc. If they are not accounted properly the structural design is meaningless. For this purpose the design engineer must have a clear concept of action of forcs.

1.3 ACTION OF FORCES/LOADS

In majority of the cases the principal forces acting on the structure consists of axial force, bending moment, shear force, torsion and their combinations.

1.3.1 Axial Force

This occurs in the case of *one dimensional (discrete) members* like columns, arches, cables and members of trusses, and it is caused by forces passing through the centroidal axis and inducing axial (tensile *or* compressive) stresses only.

(i) Fig.1.3.1a (i). shows external load *'P=0'* is applied in the centroidal plane. If tensile force $P=P_u$ is applied in centrodal plane acing on opposite faces pointing towards each other, produce internal compressive forces in the direction opposite to the applied forces so as to maintain equilibrium. They compresses the material to become shorter as shown in Fig.1.3.1a (ii). They induce compressive stresses.

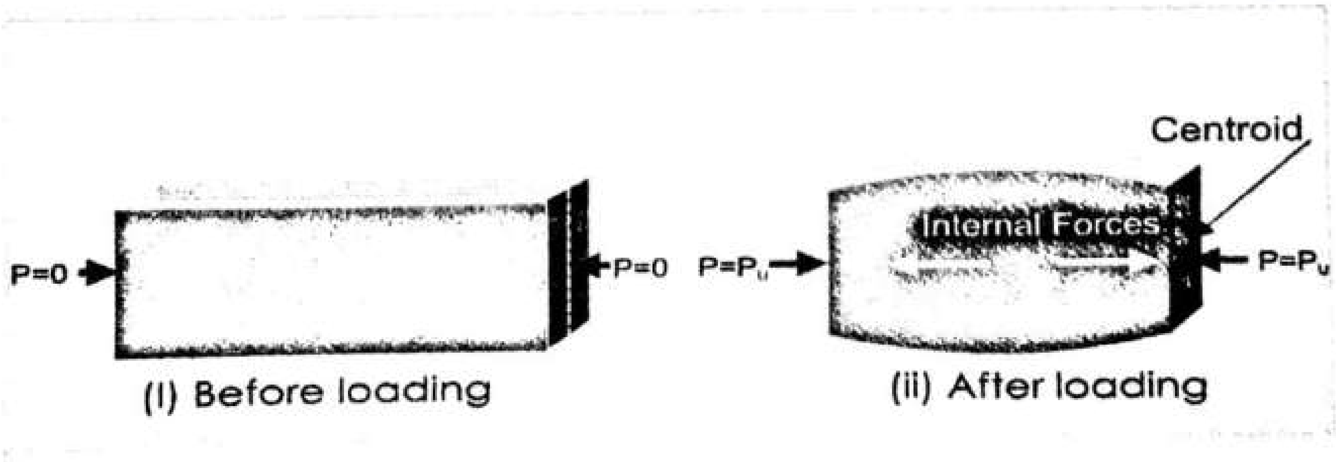

Fig. 1.3.1a

(ii) Fig.1.3.1b (i). shows external force *'P=0'* is applied in the centroidal plane. If compressive force $P=P_u$ is applied along the centroid, on opposite faces pointing away from each other, produce internal tensile forces in the direction opposite to the applied forces so as to maintain equilibrium as shown in Fig.1.3.1b (ii). They induce tensile stresses.

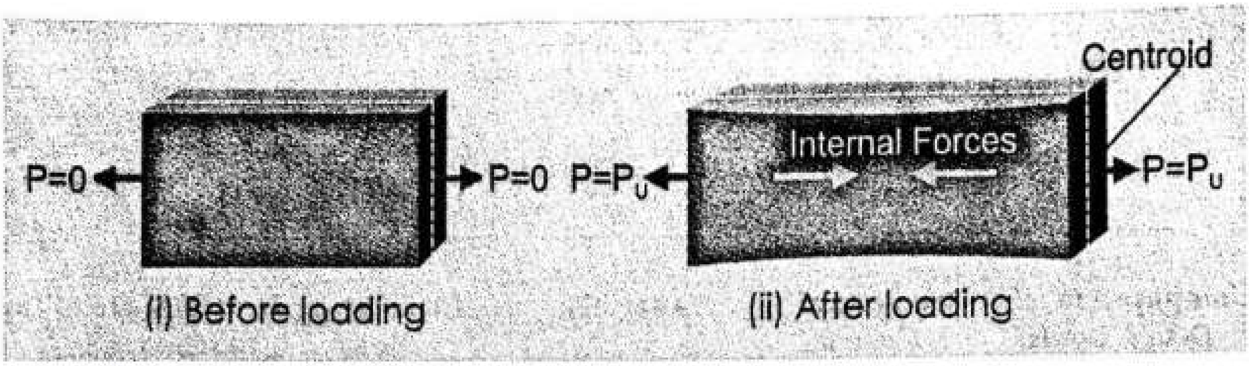

Fig. 1.3.1b

1.3.2 Forces Producing Bending compression and tension

The forces/moments either parallel or transverse to the member axis and contained in the centroidal plane of bending, induce internal bending tensile and compressive stresses.

Fig.1.3.2 (i) shows the neutral plane and centroidal plane before application of moment ($M = 0$). After application of moment ($M = M_u$) compression is induced on top face and tension at the bottom face as shown in Fig.1.3.2.(ii)

They induce compressive stresses above the neutral plane and tensile stresses below the neutral plane.

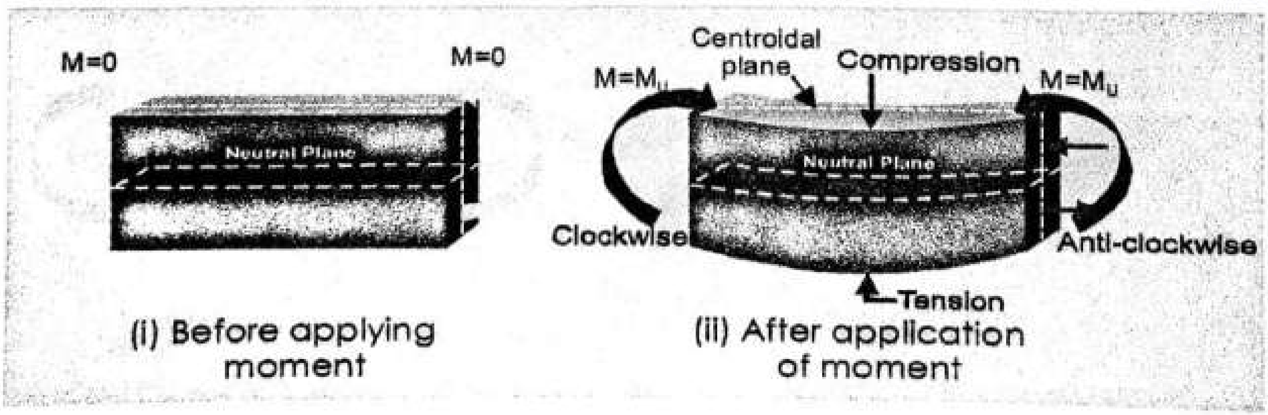

Fig. 1.3.2

1.3.3 Transverse forces Producing Shear

The shear forcec caused by in plane parallel forces and contained in the centroidal plane tend to slide the material in opposite direction as shown in Fig.1.3.3..

Fig.1.3.3 (i) shows the neutral plane and centroidal plane before application of shear (*S=0*). After application of shear ($S=S_u$) the material slide in opposite direction as shown in Fig.1.3.3 (ii) .They induce shearing stresses and produce diagonal cracks.

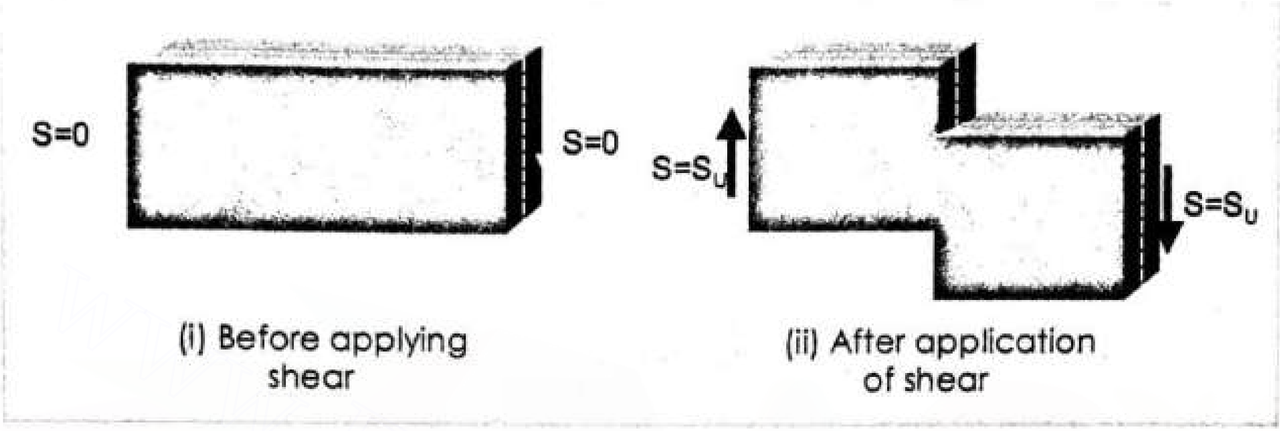

Fig. 1.3.3

1.3.4 Transverse Forces Producing Torsion

Transverse Forces/moments acting *in the plane of cross section or in the plane perpendicular to the axis of the member* produce Torsion or Twisting moment. They twist the material inducing spiral cracking shown in Fig. 1.3.4

Fig. 1.3.4 (i) shows before applying torsion (*T=0*). After application of torsion ($T=T_u$) spiral cracks are developed as shown Fig. 1.3.4 (ii). *Torsion produces shearing stresses.*

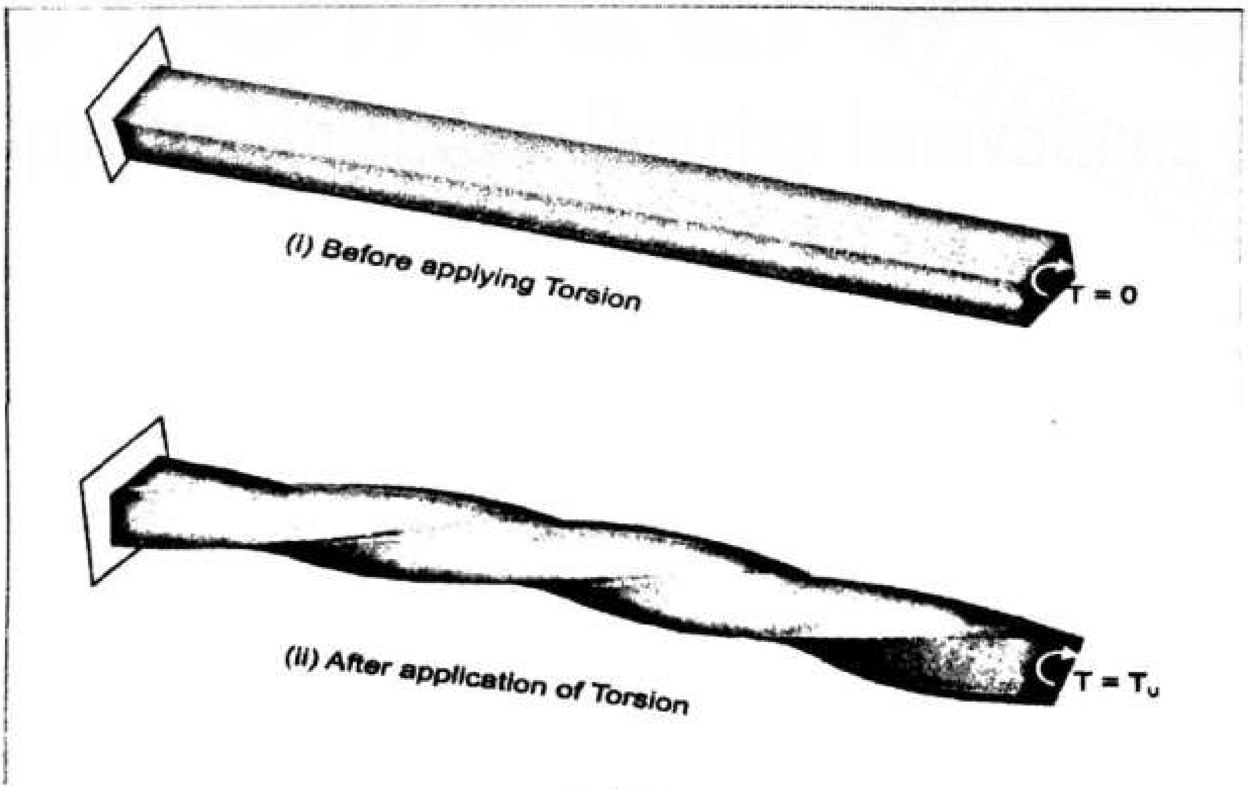

Fig. 1.3.4

1.3.5 Combination of Actions

It is a combination of one or more of above actions. They produce complex stress condition in members.

1.4 STRUCTURAL PLANNING OF R.C. BUILDINGS

After getting an architectural plan of the buildings, the structural planning of the building frame is done. This involves determination of the following :

(a) ***Positioning and Orientation of Columns.***
(b) ***Positioning of Beams.***
(c) ***Spanning of Slabs.***
(d) ***Layout of Stairs.***
(e) ***Selecting proper type of Footing.***

The basic principles in deciding the layout of component members is that the loads should be transferred to the foundation along the shortest path.

1.4.1 Positioning and Orientation of Columns

Following are some of the guiding principles which help in deciding the column positions.

Positioning of Column

1) Columns should preferably be located at or near the corners of a building, and at the intersections of beams/walls.

The positioning of columns at the intersection of walls and at the corners of a building is shown in *Fig. 1.4.1*. Since the basic function of the columns is to support beams which are normally placed under the walls to support them, their position automatically gets fixed. The commercial buildings have normally rectangular pattern of grid type shown in Fig.1.9.2 but especially for residential buildings the said type of pattern for columns does not become possible and different problems that arise are discussed further.

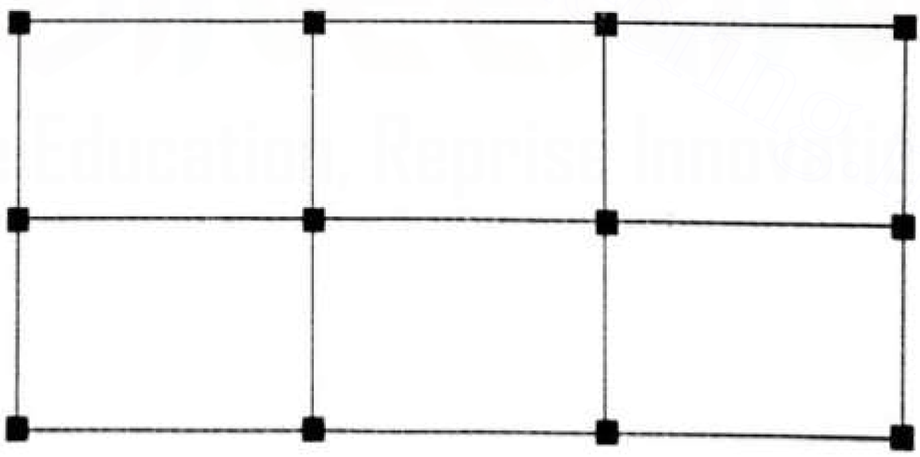

Fig. 1.4.1 Column Position for Rectangular Pattern Building

2. Avoid projection of column outside wall.

According to requirements of aesthetics and utility, projections of columns outside the wall in the room should be avoided as they not only give bad appearance but also obstruct the use of floor space, and create problems in placing furniture flush with the wall.

Therefore, as far as possible provide depth of the column in the plane of the wall to avoid such offsets. The problem of projection of column normally occurs in the internal walls since they are thinner. Now a days 150 mm thick walls are provided to get more floor space.

This has posed the problem for external walls too, because the width of column is required to be kept not less than 200 *mm* to prevent the column from being slender. Provide *L*-shaped columns at the corners or *T*-shaped columns at the intersection of intermediate cross walls.

Orientation of Columns

Normally, columns provided in a building are rectangular with width of the column not less than the width of the supported beam for effective load transfer. Restriction on the width of column necessitates the depth of column to be larger to get the desired load carrying capacity. This leads to the problem of orientation of such rectangular columns for which the following important points should be noted :

(a) The axis of bending is a transverse axis perpendicular to the plane of bending.

(b) The plane of bending is a plane of the frame or a member in which loads and longitudinal axis lie and in which deflection profile can be seen.

c) The unsupported length of the member is the length of the member contained in the plane of bending. The effective length of the column is a function of unsupported length.

(d).The major axis of bending x-x is taken as an axis bisecting the depth of the column or in other words the depth of the column is contained in the plane of bending.

(e) The properties viz. moment of inertia, deflection, stiffness are calculated about the axis of bending.

(1) ***Orient the column so that the depth of the column is contained in the major plane of bending or is perpendicular to the major axis of bending.***

Bearing in mind the guidelines given above, the principles governing orientation of columns given below can be easily understood. When a column is rigidly connected to beams at right angles, it is subjected to moments in addition to the axial load. In such cases, the column should be so oriented that the depth of the column is perpendicular to the major axis of bending (*i.e.* the depth of column shall be in the major plane of bending) so as to get larger moment of inertia and hence greater moment resisting capacity. It will also reduce L_{eff}/D ratio resulting in increase in the load carrying capacity of the column. *Fig. 1.4.2* shows the portal frame (in *xy*-plane) rigidly connected at top by beams B_2 , B_3 in the lateral direction. Since the bending moment will be very large in the plane of the frame (*i.e. xy*-plane) the depth of the column has been provided in the plane of bending to increase the moment resisting capacity of the column and reduce L_{eff}/D ratio effecting increase stiffness of column.

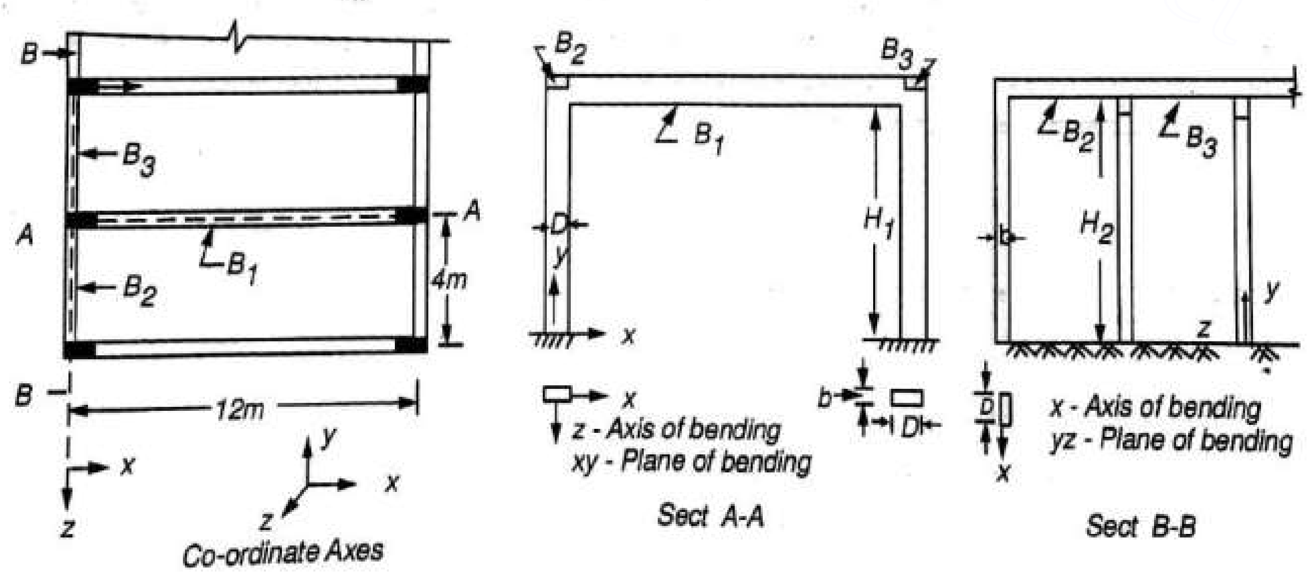

Fig. 1.4.2 Orientation of Columns from Stiffness and Effective length criteria.

It should be borne in mind that increasing the depth in the plane of bending not only increases the moment carrying capacity but also increases its stiffness, thereby more moment is transferred to the column at the beam column junction.

However, if the difference in bending moment in two mutually perpendicular directions is not large the depth of the column may be taken along the wall provided column has sufficient strength in the plane of large moment. This will avoid offset inside the rooms.

(2) Avoid larger centre to centre distance between columns.

Larger spacing of columns not only increases the span and the cost of beams but it increases the load on the column at each floor posing problem of stocky columns in lower storeys of a multistoreyed building. Heavy sections of column lead to offsets from walls and obstruct the floor area.

(3) Columns on property line.

The columns on property line need special treatment. Since column footing requires certain area beyond the column, difficulties are encountered in providing footing for such columns. In such cases, the column may be shifted inside along a cross wall to make room for accommodating the footing within the property line as shown in *Fig. 1.4.3a.* Alternatively, a combined footing *or* a strap footing (Fig.1.4.3b and 1.43c) may be provided which are further detailed in Chapter 12

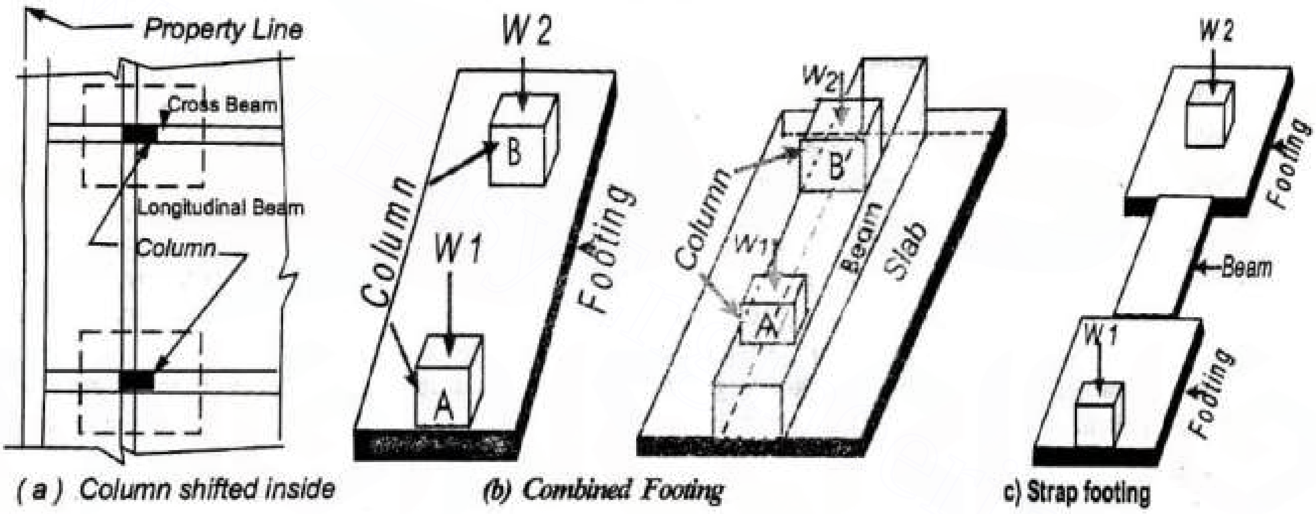

Fig.1.4.3 Columns on Property Line

(4) Select the position of columns so as to reduce bending moments in beams,

When the locations of two columns are very near (*e.g.* as it occurs when the corner of a building and the point of intersection of walls come very close to each other), then one column (either P or Q) should be provided instead of two at such a position so as to reduce the beam moment.

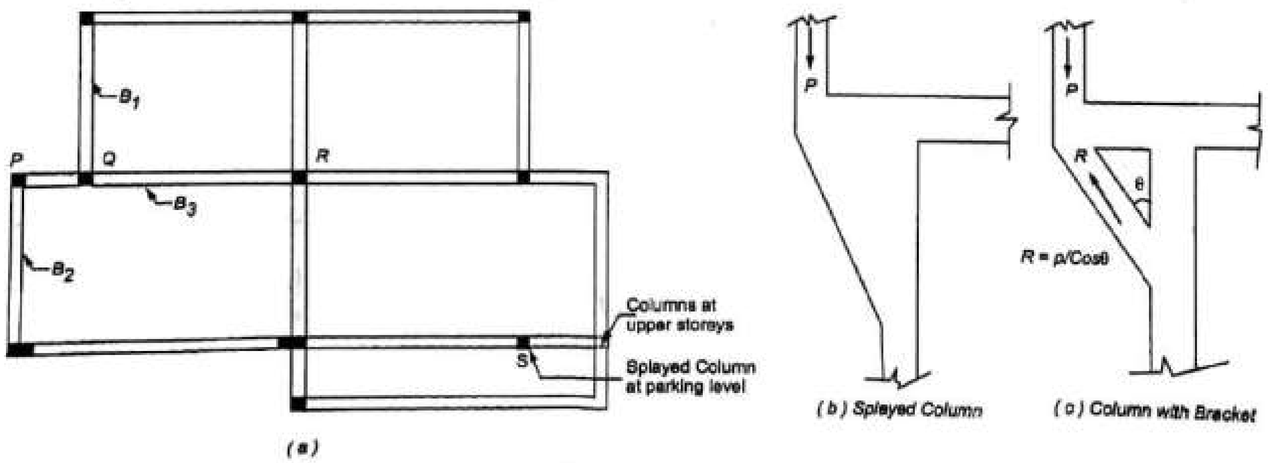

Fig 1.4.4

In buildings small offsets (such as *PQ*) are provided from architectural considerations. Now the question arises whether to provide the column at *P* or *Q*.

For illustration see *Fig. 1.4.4a*. In buildings small offsets (such as *PQ*) are provided from architectural considerations. Now the question arises whether to provide the column at *P* or *Q*. Consider only the point loads (excluding load transferred by floors) transferred by beams B_1 and B_2. If only column *P* is provided beam B_1 will transfer a concentrated load at *Q*. In such a case beam B_3 will have larger span and will be subjected to concentrated load at *Q* thereby there will be considerable increase in B.M. Instead of this if the column is located at *Q*, the cantilever moment due to the reaction of B_2 at *P* will relieve the bending moment in B_3 , thus providing a cheaper alternative.

Under certain rare circumstances, to satisfy the functional requirements, it may not be possible to provide upper storey columns above the columns at the parking level. Then the column at parking level is required to support the eccentric columns at upper storeys. In such a case the column at parking level is splayed as shown in *Fig. 1.4.4b or* provided with a bracket (see *Fig. 1.4.4c*) to support the columns at the upper storeys. However, the column at parking level will be subjected to heavy concentrated loads transferred from the columns of upper storeys.

1.4.2 Positioning of Beams

Following are some of the guiding principles for positioning of beams.

(1) Beams shall, normally, be provided under the walls or below a heavy concentrated load to avoid these loads directly coming on slabs.

Since beams are primarily provided to support slabs, their spacing shall be decided by the maximum spans of slabs. Slab requires the maximum volume of concrete to carry a given load (*i.e.* its volume/load ratio is very large compared to the other components). Therefore, the thickness of slab is required to be kept minimum. The maximum practical thickness of slab for residential/office/public buildings is 200 *mm* while the minimum is 100 *mm.*

The maximum and minimum spans of slabs which decide the spacing of beams are governed by loading and limiting thickness given above. In the case of buildings, with live load less than 5 kN/m^2, the maximum spacing of beams may be limited to the values of maximum spans of slabs given below.

Support Condition	*Cantilevers*		*Simply Supported*		*Fixed/Continuous*	
Slab Type	One-way	Two-way	One-way	Two-way	One-way	Two-way
Maximum recommended Span of slabs	1.5 *m*	2.0 *m*	3.5 *m*	4.5 *m*	4.5 *m*	6.0 *m*

(2) Avoid larger spans of beams.

When the centre to centre distance between the intersection of walls is large *or* where there are no cross walls, the spacing between two columns is governed by limitations on spans of supported beams, because spacing of columns decides the span of the beam. As the span (and the length) of the beam increases, the required depth of the beam, and hence its self weight, and the total load on beam increases. It is well known that the moment governing the beam design varies with the square of the span and directly with the load. Hence, with the increase in span, there is considerable increase in the size of the beam. On the other hand, in the case of a column, the increase in total load (and hence the increase in size) due to increase in length is negligible as long as the column is short. Therefore, the cost of the beam per unit length increases rapidly with the span as compared to that of column. Columns are, therefore in general, always cheaper compared to beams on the basis of unit cost. Therefore, large spans of beams should preferably be avoided for economy reasons. This aspect is illustrated in *Fig.1.4.5.* In this case, either one column at *C* can be provided making *ACB* a two span continuous beam *or* two columns can be provided at *E* and *G* to form *AB* a three span continuous beam.

In the first case, if columns are provided at *A, C, and B,* then spans *AC* and *CB* will be large and the beam has to carry two point loads, one at *E* and the other at *G*, transferred from secondary beams.

In the first case, if columns are provided at *A, C, and B*, then the span *AC and CB* will be lage and the beam has to carry two point loads, one at E and the other at G, transferred from secondary beams.

This will require heavier section for the beam. In the latter case, when two columns are provided, one at *E* and the other at *G*, the beam becomes a three span beam. Length of beam is reduced and it is required to carry only one concentrated load and that too on central span which further reduces the moment in beams in outer spans *AE* and *GB* without appreciable increase in design moment in portion *EG* leading to considerable reduction in the cost of beam. On the other hand since the cost of column is nearly proportional to the load on it, increase in cost of columns and footings due to provision of two columns at *E* and *G* (carrying half the load), over the cost of providing single column at *C* will be comparatively less than the increase in the cost of beam due to providing single column. Thus, the second alternative is likely to work out to be cheaper. This is more true in the case of multistory building frames.

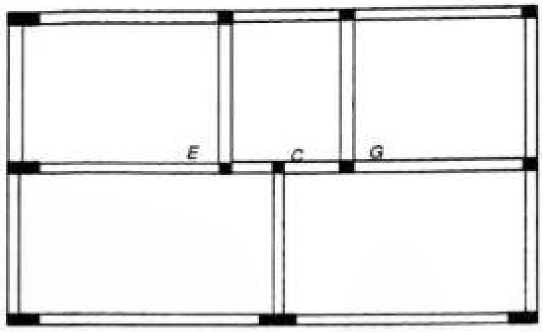

Fig. 1.4.5

In general, the maximum spans of beams carrying live loads up to 4 kN/m^2 may be limited to the following values.

Beam Type	*Cantilevers*	*Simply Supported*	*Fixed / Continuous*
Rectangular	3 meters	6 meters	8 meters
Flanged	5 meters	10 meters	12 meters

The upper limit shall be reduced by judgement for heavy loads (live load greater than $4kN/m^2$.

(2) Avoid larger spacing of beams from deflection and cracking criteria.

Larger spans of beams shall be avoided from considerations of controlling deflection and cracking. This is because deflection varies directly with the cube of the span and inversely with the cube of the depth (L^3 / D^3) as :

$$\delta = \frac{\alpha W L^3}{E I} = \frac{\alpha W L^3}{E \times b D^3 / 12} = \frac{\alpha\, 12\, W}{E \times b} \times \frac{(L^3)}{(D^3)}$$

In this case as *L* increases *D* does not increase in that proportion with the result δ increases considerably.

However, for large spans, normally higher *L/D* ratio is taken to restrict the depth from considerations of headroom, aesthetics and psychological effect (a long, heavy, deep beam creates a *psychological* feeling of crushing load leading to a fear of collapse). Therefore, spans of beams which require the depth of beam greater than one meter should as far as possible be avoided.

1.4.3 Spanning of Slab

This is decided by supporting arrangements. When the supports are only on opposite edges *or* only in one direction, then the slab acts as a *one - way* supported slab.

When the rectangular slab is supported along its four edges, it acts as a one-way slab when $L_y / L_x > 2$ (*i.e.* on the ratio of long span L_y to short span L_x) and as two-way slab for $L_y / L_x < 2$. However, the *two-way* action of slab not only depends on the aspect ratio L_y / L_x but also on the ratio of reinforcement in the two directions. Therefore, designer is free to decide as to whether the slab should be designed as *one- way* or *two-way*. This decision may be taken considering the following points.

(1) A slab normally acts as a one-way slab when the aspect ratio $L_y/L_x > 2$.

A slab with $L_y/L_x > 2$ is designed as one-way, since in that case one-way action is predominant. In one-way slab, main steel is provided along the short span only and the load is transferred to two opposite supports only. The steel along the long span just acts as distribution steel and is not designed for transferring the load but to distribute the load and to resist shrinkage and temperature stresses.

In practice, however, a slab having supports on all sides but having $L_y/L_x < 2$ is sometimes designed as one-way slab. Such slab is made to act as a one - way slab spanning across the short span by providing main steel along the short span and only distribution steel along the long span. In such a case, provision of more steel in one direction increases the stiffness of the slab in that direction. According to *elastic theory*, the distribution of load being proportional to stiffness in two orthogonal directions, major load is transferred along the stiffer short span and the slab behaves as one-way. Also, according to yield line theory, the load distribution in two orthogonal directions depends upon the ultimate moment capacities m_{ux} and m_{uy} in these directions. By providing more steel only in short direction m_{ux} is made far greater than m_{uy} and the slab is made to act as one-way. However, it should be noted that since the slab is also supported over the short edge, there is a tendency of the load near the support to get transferred to that support causing tension at top along the short supporting edge. Since there does not exist any steel at top across this short edge in a one-way slab interconnecting the slab and the side beam, cracks develop at top along that edge. The cracks may run through the depth of slab due to differential deflection between the slab and the supporting short edge beam/wall. Therefore, care should be taken to provide *minimum steel* at top across the short edge support to avoid this cracking.

(2) *A two - way slab having aspect ratio $L_y/L_x < 2$ is generally economical compared to one - way slab because steel along both the spans acts as main steel and transfers the load to all its four supports.*

The two-way action is advantageous essentially for large spans (greater than 3 *m*) and for live loads greater than 3 kN/m^2. For short spans and light loads, steel required for two - way slab does not differ appreciably as compared to steel for one-way slab because of the requirement of minimum steel.

(3) Spanning of slab is also decided by the necessity of continuity to adjacent slab.

For illustration, in *Fig. 1.4.6a.* if the slab S_1, is to be designed as a slab continuous over the support *AB*, then it is necessary that slab S_2 also spans across *AB*.

If slab S_2 is designed as one-way slab spanning only in the direction parallel to *AB*, then the slab S_1 will not get the desired fixity or structural continuity over *AB see Fig. 1.4.6b*. In such a case, even though full steel is provided at top across *AB* to cater for the support moment, the beam *AB* would simply rotate in absence of any balancing load coming from S_2 and S_1 simply acts as slab freely supported on *AB*.

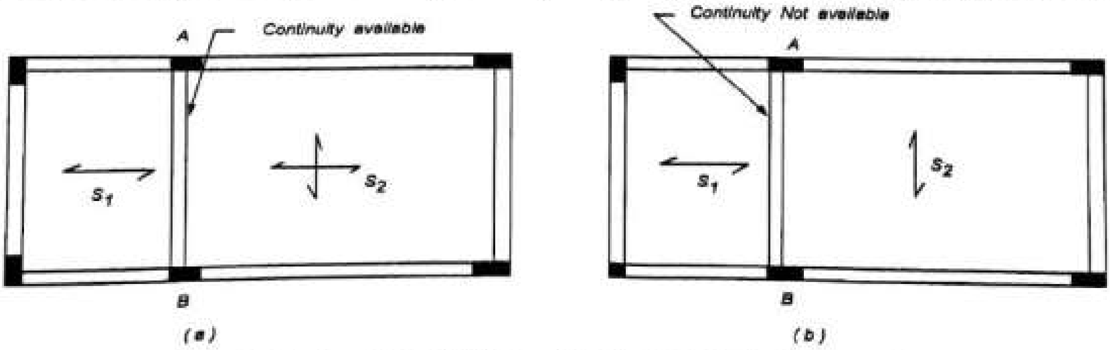

Fig. 1.4.6 Spanning of Slabs - Necessity of Continuity over Edge

4) *Decide type of slab.*

While deciding the type of slab, whether a cantilever or a simply supported or a continuous slab, loaded by UDL it should be borne in mind that the maximum bending moment in a cantilever slab *($M = wL^2/2$)* is four times that of a simply supported slab *($M = wL^2/8$)*, while it is five to six times that of a continuous *or* fixed slab *($M = wL^2/10$ to $wL^2/12$)* for the same span length.

Now , the deflection of a cantilever loaded by a uniformly distributed load is given by :

$$\delta = \frac{wL^4}{8\,EI} = \frac{48}{5} \times \frac{5wL^4}{384\,EI}$$ which is 9.6 times that of a simply supported slab $= \dfrac{5wL^4}{384\,EI}$

Therefore , for the same span and load , deflection is 9.6 times that of a simply supported slab (besides, additional reduction in deflection is obtained in simply supported slab due to partial fixity at supports). Further , in the case of cantilevers, on the contrary, there is a probability of increase in deflection due to probable rotation of the supporting beam due to lack of adequate end restraint for the beam. Therefore, in the case of balcony slabs the economic spanning is governed by the ratio of length of balcony (the longitudinal span for simply supported / continuous slab) to the width of balcony (which can act as transverse span for cantilever) and the availability of supporting transverse beams for longitudinal spanning.

Fig. 1.4.7 Shows different ways of supporting balconies. Normally balcony slab is designed as a cantilever as shown in *Fig. 1.4.7a*. But in the case of an isolated single balcony, S_1 in *Fig. 1.4.7(b)* if transverse beams are available at the ends and if the length of balcony is less than two times the width, it will be economical to design the balcony slab as simply supported spanning longitudinally across the transverse end beams instead of as a cantilever slab (see *Fig 1.4.7b)*. For a long balcony where number of transverse beams are available, this ratio of longitudinal span to width can even be 2.5.

If the width of balcony (L_x) is large and the transverse beams *DE* and *FG* are available at the ends, even a longitudinal beam *EF* can be provided along the free edge below the parapet wall and the slab S_2 could be made to span across the floor beam *DG. See Fig. 1.4.7c* (This principle is adopted in counterfort retaining walls by making the vertical stem to span longitudinally across the counterfort instead of transversely (*i.e.* vertically as cantilever) when the height is large). However, in all the cases illustrated above, it has to be seen whether the supporting transverse beams can be made available by extension of inner floor beams as brackets or not.

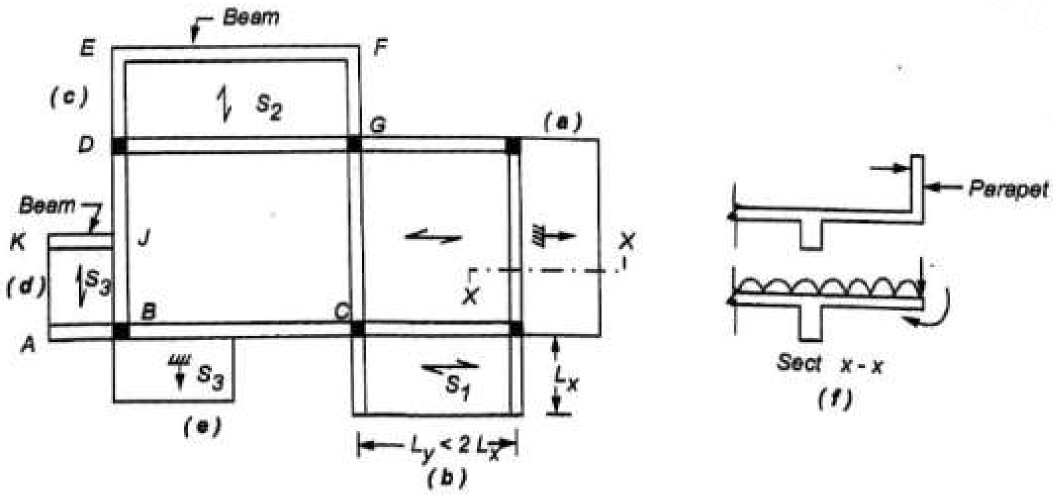

Fig. 1.4.7 Different Methods of Supporting Balconies

In the case of balcony S_3 in *Fig. 1.4.7d* which does not extend over the complete length of the room, transverse beam could be made available at *AB* by extending the beam *CB*. But it would not be available along *JK* as there is no floor beam inside in line with it. In such a case, the slab should preferably be designed as cantilever *(Fig. 1.4.7e)* because provision of a separate supporting beam *JK* shown in *Fig. 1.4.7d.* would induce large twisting moment in beam *BD*.

The presence of vertical parapet wall at the edge of the balcony makes the cantilever spanning further uneconomical because of the additional moments induced by the weight of the parapet acting at the free end as point load and due to accidental horizontal load acting at the edges of vertical wall *(See Fig. 1.4.7f)*

If the slabs are spanned longitudinally, the weight of parapet wall can be transferred directly to the supporting cross beams since the wall itself can act as a vertical deep beam provided of course it is supported transversely at top by either a transverse parapet wall or a hand rail.

(5) Canopy or Porch

While designing any slab as a cantilever slab, it is of utmost importance to see whether adequate anchorage to the same is available or not. For example, if a cantilever canopy slab S_1 in *Fig. 1.4.8* is to be provided outside the entrance instead of a column supported porch and that too at a different (lower) level than that of the floor beam *AB*, then adequate anchorage will not be available because slab S_1 cannot be extended inside the hall due to level difference between S_1 and S_2. In such a case, the beam *AB* will either be required to be made very deep, with depth equal to level difference between S_1 and S_2 and canopy slab connected to its bottom, or separate beam will have to be provided below *AB* at the level of S_1 if the projection of the canopy is large. In both cases supporting beams will be subjected to large torsional moment and in such a case it is necessary to ensure that beams are properly anchored at supports to prevent their rotations. The different types of layouts and their design of porches/canopies have been given in *details in Chapter -11.*

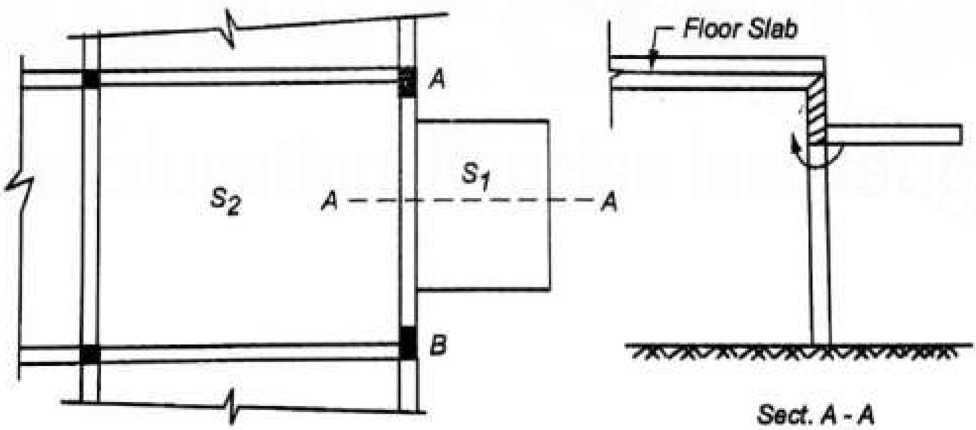

Fig. 1.4.8 Canopy / Porch Slab

(6) Corner Balconies

Another common problem in case of balconies is that of a corner balcony S_3 *See (Fig. 1.4.9(a))*. If balconies S_1 and S_2 are both spanning longitudinally across transverse beams *AB* and *AD*, corner slab S_3 can just be overhanging extensions of slabs S_1 and S_2 with 50% load transferred in each direction. This is economical. On the other hand, if both S_1 and S_2 are cantilever balconies with no beams at *AB* and *AD*, corner slab S_3 does not get any support except from S_1 and S_2 which themselves are cantilevers.

Since the transfer of load of S_3 on to S_1 and S_2 makes the design of S_1 and S_2 further uneconomical and complicated, slab S_3 should be supported by radial bars of minimum 12 *mm* diameter, and anchored backwards in slab S_4 through equal length. A diagonal bar *EF* should preferably be provided above the rear ends of radial bars and it should be anchored in beams below top bars of supporting beams B_1 and B_2 to prevent lifting of the radial bars as shown in *Fig. 1.4.9(b)*

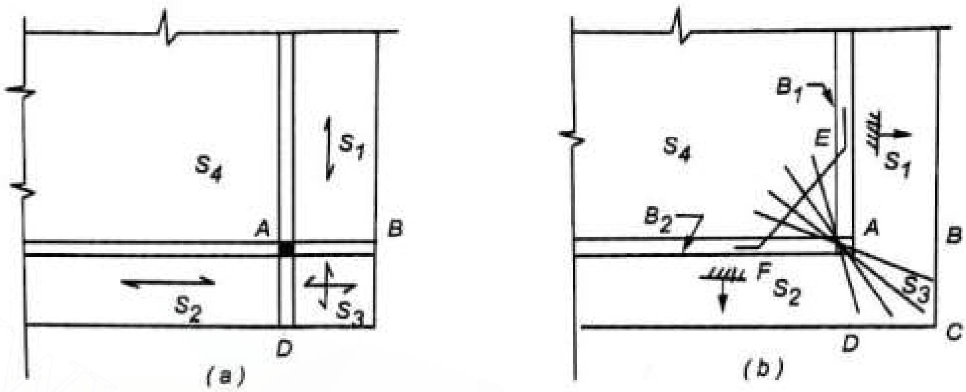

Fig. 1.4.9 Corner Balconies

1.4.4 Layout of Stairs

Initially, it is necessary to know the different parts of stairs and guide lines for fixing their dimensions. The component parts of the stairs are shown in *Fig. 1.4.10*

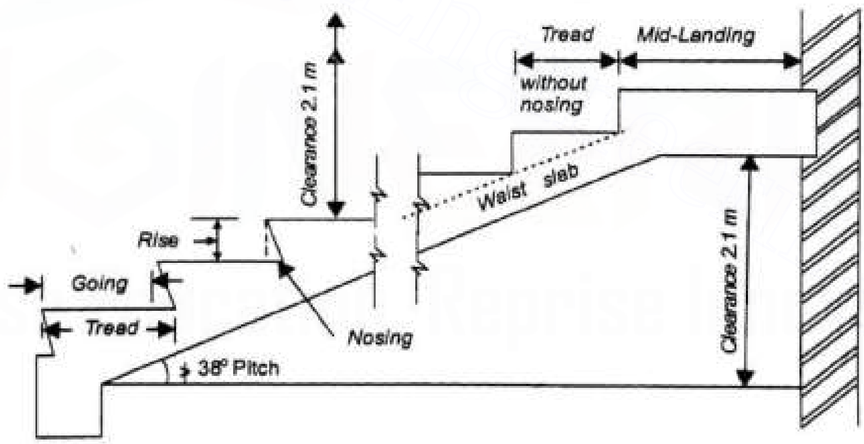

Fig. 1.4.10 Different Parts of Stairs

The guide lines for fixing dimensions of the component parts of stairs are as under :

1. The rise *R* should not be more than 200 *mm* and tread *T* not less than 200 *mm*.
   - for residential buildings the riser (*R*) may be between 150 *mm* to 180 *mm* and tread (*T*) between 220 *mm* to 250 *mm*.
   - for public buildings the riser (*R*) may be kept between 120 *mm* to 150 *mm* and tread (*T*) between 250 *mm* to 300 *mm*.
2. The sum of the tread plus twice the rise ($T + 2R$) should be between 500 *mm* to 650 *mm*.
3. The width of the stairs is dependent on its usage and shall be such as to avoid over crowding.
   - for residential buildings the width of the stairs should be between 0.8 *m* to 1 *m*.
   - for public buildings it should be between 1.8 *m* to 2 *m* or more.
4. The width of the landing should not be less then width of stairs.
5. For comfortable ascend on stairs, the number of steps in each flight should not be greater than 12.

6. The pitch of the stairway should not be greater than 38°.
7. The head room measured vertically above any step or below mid-landing shall not be less than 2.1*m*.
8. Avoid winders as far as possible.

The type of stairs and its layout is governed essentially by the available size of staircase room and the positions of beams and columns along the boundary of the staircase.

Following are some useful guide lines in deciding the layout and type of stairs :

(1) The stair slabs, in general, are heavy compared to floor slabs because of (i) heavy dead load due to inclined length of slab acting over horizontal span, and due to additional weight of steps, (ii) greater live load on stairs than that on floors. Therefore, longer spans for the flights be avoided as far as possible.

(2) Stair flights shall preferably be supported on beams or walls. Supporting the flight on landing slab should be avoided as far as possible especially when the span of the landing slab exceeds twice the width of stair, because this causes stress concentrations in the supporting landing slab at their junction.

(3) For the case of dog-legged stairs the landing beams may be provided at the end of flight to reduce the span. For example, in *Fig. 1.4.11a* beams can be provided either at *AB or* at *EF* on one side and at *GH or* at *CD* on the other side. Beams at *EF* and *GH* not only reduce the span of stair slab but the landing slabs beyond *EF* or *GH* act as cantilevers which reduce the design moment at mid-span giving double benefit and hence this arrangement is most economical. Thus, supporting stair slabs along *AB* and *CD* is uneconomical. When the provision of a mid-landing beam, say at *EF*, is not possible due to non-availability of adequate headroom under the landing, the flight may be supported on landing slab itself. The landing slab may be made to span transversely across *AE* and *BF* on walls or on bracket beam taken out from the columns as shown in *Fig. 1.4.11b.*

(4) If the span of stair flight is greater than 4.5 meter, the flight may be supported on a central stringer beam spanning across *AB* and *CD* and the steps of the stair flight cantilevering out from the stringer beam on both sides. This arrangement is aesthetically excellent for public buildings like hotels, theatres, banks, etc. *See Fig. 1.4.11c*

(5) Open newel *or* Open-well stair consists of rectangular well *or* open space provided between forward and backward flights as shown in *Fig.1.4.11d.* The space serves the purpose of ventilation or used for lift. These types of stairs are normally provided in commercial buildings. For details of design see authors book[1.2]

(6) The spiral stairs (Fig.1.4.11e) consists of steps radiating from the central newel post in the form of winders. It is normally provided at the back side of the building for rendering access to floors for service purposes *or* where their use is limited as in the case of overhead tanks *or* when the space is limited.

(7) Skew support shall as far as possible be avoided since they induce torsion in the flight slab. Beams shall be provided over the skew support.

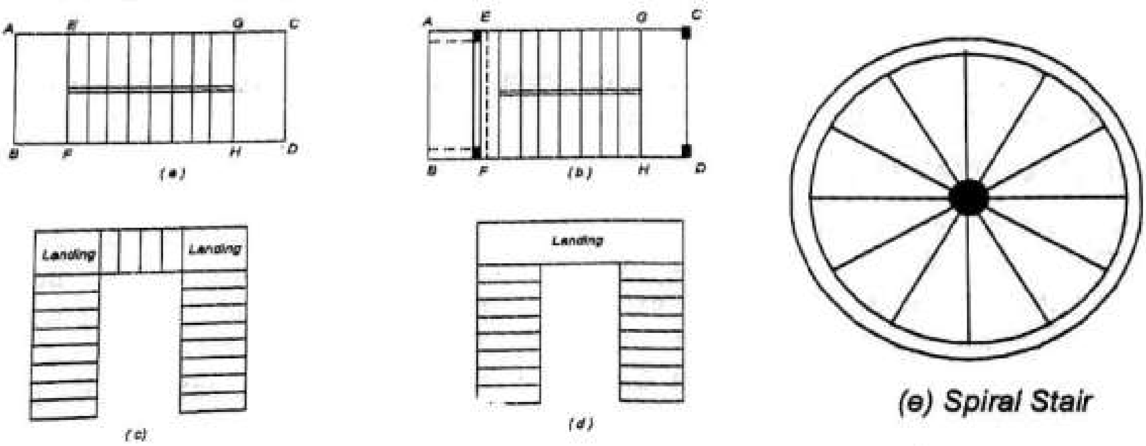

Fig. 1.4.11 Different Types of Stairs

1.4.5 Choice of Footing Type

The type of footing depends upon the load carried by the column and bearing capacity of the supporting soil. The representative values of the safe bearing capacities of the typical soils have been given in *Appendix-B, Table B-1*. It may be noted that the earth under the foundation is susceptible to large variations. Even under one small building the soil may vary from a soft clay to a hard murum. Also the nature and properties of soil may change with season and weather, swelling in wet weather. Increase in moisture content results in substantial loss of bearing capacity in case of certain soils which may lead to differential settlement. The permissible differential settlements and tilt of shallow foundation in soil have been specified in IS:1904[1.3]. It is necessary to conduct the survey in the area where the proposed structure is to be constructed to determine the soil properties. Drill holes and trial pits should be taken and in situ plate load test[1.4] (for big projects) may be performed and samples of soil tested in the laboratory to determine the bearing capacity of soil and other properties.

For framed structures under study, isolated column footings are normally preferred except in case of soils with very low bearing capacities. For such soil *or* if black cotton soil exists for great depths, pile foundations can be an appropriate choice.[1.6a] If columns are very closely spaced and bearing capacity of the soil is low, raft foundation can be an alternative solution. For a column on the boundary line, a combined footing or a strap footing may be provided, as detailed earlier.

1.5 COMPUTATION OF LOAD

As mentioned earlier, this is the most important step in design. The basic concepts of types of forces/loads are explained in Sect.1.3. The total load acting on a structure is calculated after carrying out the design of structural members (slab, beam, and column). However, for urgent work where footing details are required prior to detailed design, the rough estimate of loads can be made by other methods, the details of which are given in Chapter. 9.

1.6 ANALYSIS OF STRUCTURE

The main approaches to structural analysis are

(1) Elastic Analysis
(2) Limit Analysis

Elastic analysis is used in working stress method of design.

Limit analysis is further bifurcated as Plastic theory applied to steel structures and Ultimate load method of design, and its modified version namely Limit State Method for R.C. structures, which includes design for ultimate limit state at which ultimate load theory applies and in service state where elastic theory is used. For further details see *Sect. 3.1*.

1.7 MEMBER DESIGN

The member design consists of design of slab, beam, column and footing. These are undertaken once the basic theory of limit state method is understood.

1.8 DETAILING, DRAWING AND PREPARATION OF SCHEDULE

Detailing is a process of evolution based on understanding of structural behavior and material properties. The good detailing ensures that the structure will behave as designed and should not mar the appearance of the exposed surface due to excessive cracking. The skillful detailing will assure satisfactory behavior and adequate strength of structural members. The normal detailing rules and some typical problems in detailing are given in *Chap.5*. Normally it is sufficient to give details of reinforcement in the schedule but detailed drawings should be prepared for typical cases.

1.9 MARKING OF FRAME COMPONENTS

Before starting the structural design of R.C. frame components, it is always necessary to mark or designate them first to facilitate identification, listing, and scheduling. The different schemes adopted for marking or identification are given below.

(a) Column Reference Scheme

(b) Scheme as recommended by IS:5525[1.5] : "Recommendations for detailing of reinforcement in Reinforced Concrete Works". This scheme of marking is called as a *Grid Reference Scheme*.

(c) Scheme followed by the private sector.

1.9.1 Column Reference Scheme

In this scheme, columns are first of all numbered serially starting from the column at top left corner proceeding rightward and then downwards as shown in *Fig.1.9.1*. Beams are designated as *Bij* in which suffix i refers to column number from which the beam starts and suffix j refers to the direction in which it runs. ($j = 1$ for beams going northwards in (y) direction, $j = 2$, for beams going eastwards in (x) direction, $j = 3$ is used for cantilever beam going southwards with no column beyond, while $j = 4$ is used for cantilever beam going westwards with no column beyond). Thus, beam B_{51} is a beam starting from column No. 5 and running northward in y direction, while beam B_{52} is a beam starting from the same column 5 but running eastwards in x-direction. *(See Fig. 1.9.1)*. The secondary beams which are not supported on columns but on other main beams and which run in y direction are given odd numbers such as B_{15}, etc. while those in x direction are designated by even numbers such as B_{16}, B_{76}, B_{86} etc. See Fig. 1.9.1

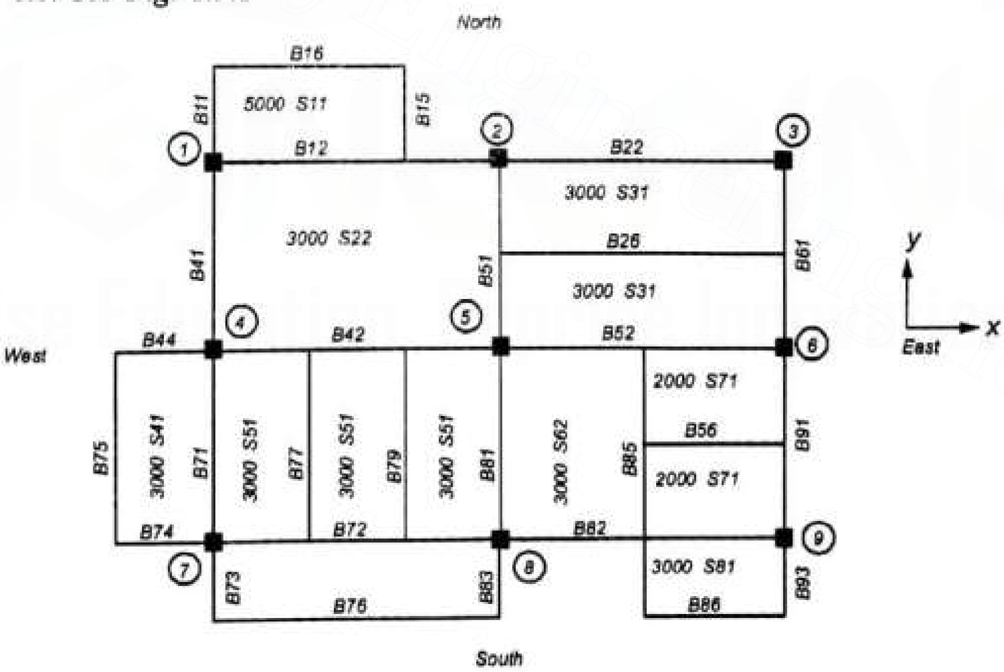

Fig. 1.9.1 Column Reference Scheme

This scheme is followed by Public Works Department of some states and by steel structures fabricators and erectors. It is not very common with R.C. designers in privatae sector. The Government Departments which adopt this marking scheme, designate slabs as wS_{ij} in which prefixing letter w indicates class of live load (value in *kg /sq.m* or in *N/sq.m*) for which the slab is designed, suffix i indicates category number of the slab, while suffix j indicates the type of slab whether one-way or two-way ($j = 1$ for one - way slab and $j = 2$ for two-way slab). Category of slab is known by the specifications of the slab, namely, depth and amount (diameter and spacing) of bars. For illustration, 2000S32 indicates slab of category No.3 designed as two-way for a live load of 2000 N/m^2. This practice is useful and advantageous for maintaining a proper record especially when different slab panels are designed for different loads. This record is helpful to avoid wrong usage or overloading of the room in future due to change of user which is very common in Government departments or public sector.

1.9.2 Grid Reference Scheme

In this scheme of marking, starting from the column at the bottom left corner, series of imaginary horizontal grid lines passing through each column are marked as *A-A, B-B, C-C*, etc, and vertical grid lines passing through each column are marked as 1-1, 2-2, 3-3 as shown in *Fig. 1.9.2*. The columns are designated as C_{ij} in which suffix *i* and *j* refer to horizontal (*i th*) and vertical (*j th*) grid lines intersecting at the column. Thus, the column at *x* in *Fig. 1.9.2* is marked as C_{D3}. Beams are marked as B_{m1}, B_{m2} etc. serially starting from the top left corner and proceeding downwards and then rightward (bay wise) sequentially. Slabs are designated serially as S_{b1} , S_{b2} , starting from panel in top left corner, proceeding vertically downwards bay wise and then rightward. *See Fig. 1.9.2*. This scheme is partially followed in practice. Scheme of marking columns in this way is very common, but that for beams and slabs is not very much favored (especially writing suffixes *m* and *b* to mark beam and slab respectively, is considered to be superfluous).

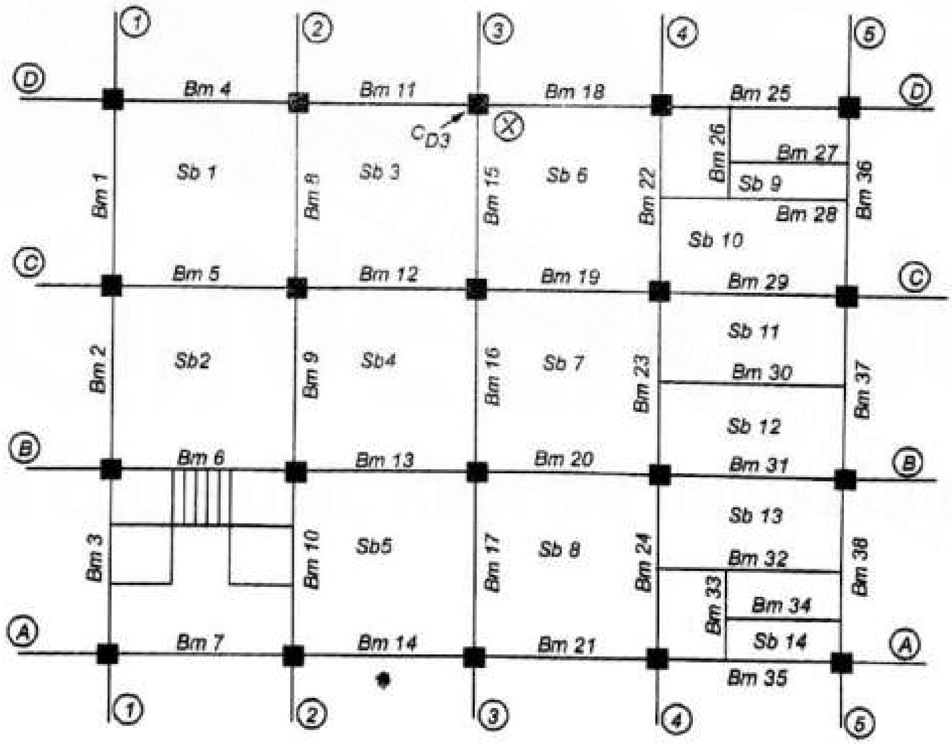

Fig. 1.9.2 Grid Reference Scheme

1.9.3 Scheme used in Private Sector

In this scheme, the columns are marked serially as *C1* , *C2*, *C3*, etc. *or* by encircled numbers such as 1, 2, 3, etc. by the side of the column starting either from top (or bottom) left corner and moving right ward and downwards (or upwards as the case may be). See *Fig. 1.9.3*. Beams are marked serially as *B1*, *B2*, etc. starting from first column and moving right ward first and then downwards (or upwards as the case may be), thus numbering first all the beams in horizontal or *x*-direction and then numbering upwards in *y*-direction starting from left most beams as shown in *Fig. 1.9.3*

However, the slabs are not marked serially but are marked according to their categories based on design specifications (namely the thickness, diameter, and spacing of reinforcement along two perpendicular spans). This facilitates scheduling of slabs. Nevertheless, it requires grouping of slab panel first having nearly equal spans, end conditions and the load so that categories of slabs required to be designed are reduced to a minimum. The spanning of slabs is shown, by arrows on the plan and specifying separately in the schedules under remarks column.

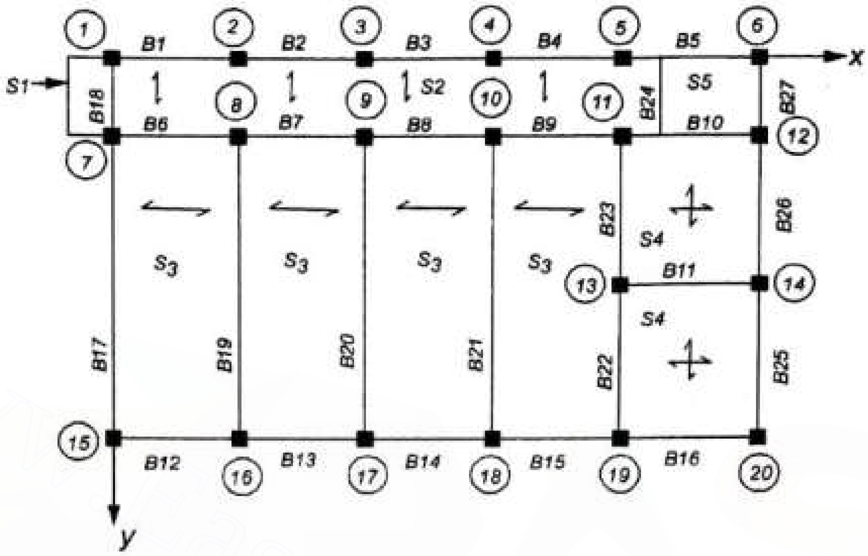

Fig.1.93

At present, the loads for which the slabs are designed are many times not shown on the drawings. However, since these drawings form a permanent record with the user *or* with the licencing bodies like municipal corporations, it is advisable to record the design live load along with the specification of grades of concrete and steel in the notes on the drawings. Rather, authors feel that this should be made obligatory by the licensing bodies to avoid change of use of the premises in future, and avoid possible failures. (One case of failure of a structure has been reported in the newspaper when a 40 year old building meant for residential purpose was converted into a marriage hall and it collapsed due to overload.)

1. 10 DESIGN PHILOSOPHIES

Reinforced concrete structures can be designed by using one of the following design philosophies.

(1) Working Stress Method *(WSM)*,

(2) Ultimate Load Method *(ULM)*,

(3) Limit State Method *(LSM)*

(1) Working Stress Method *(WSM)*

Working stress method is based on elastic theory. But neither the concrete nor the high yield stress deformed bars behave elastically . The fallacy using elastic theory to reinforced concrete structures was brought out by research in the middle of 20th century with the result that working stress method used over decades is now practically outdated. It is not being used at all in advanced countries of the world including India, because of its inherent drawbacks.

(2) Ultimate Load Method (ULM)

The working stress method was phased out giving way to Ultimate Load Method. In this method the structural element is proportioned to withstand the ultimate load , which is obtained by enhancing the service load by some factor referred to as Load factor , for giving desired margin of safety. The ultimate method ensures safety but disregards serviceability aspect.

(3) Limit State Method (LSM)

The limit state method ensures safety against loads, deformations and has adequate margin of safety during the anticipated service span of the structure. The condition or limit state at which the structure or its part becomes unfit for its intended use is called as Limit State Method. Thus , limit state method ensures the safety at ultimate load and serviceability at service load rendering the structure fit for it intended use.

For detailed comparison of the three methods, refer to author's text book[1.6b]

In this book, therefore, the limit state philosophy of design has been followed throughout using SI units. The Government departments and large consulting firms and engineers in the private sector have already switched over to this method.

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(1.6a) Chapter - 13, pp 727-742

(1.6b) Chapter - 1, Sect.1.2, *pp* 11-16

CHAPTER - 2

LOADS AND MATERIALS

2.1. INTRODUCTION.

Loads and properties of materials constitute the basic parameters affecting the design of a R.C. structure. Both of them are basically of varying nature. The correct assessment of loads/forces on a structure is a very important step for the safe and serviceable design of structure.

2.2 TYPES OF LOADS

The loads are broadly classified as *vertical loads, horizontal loads* and *longitudinal loads*. The vertical loads consist of dead load, live load and impact load. The horizontal loads comprises of wind load and earthquake load. The longitudinal loads. (*viz.* Tractive and braking forces are considered in special cases of design of bridges, design of gantry girders etc.)

2.2.1 Dead Load

Dead loads are permanent or stationary loads which are transferred to the structure throughout their life span. Dead load is primarily due to self weight of structural members, permanent partition walls, fixed permanent equipment and weights of different materials such as brick, stone etc.

The dead weight of materials are given in *Appendix - A, Table A-1*

2.2.2 Imposed Loads *or* Live Loads

Live loads are either movable or moving loads without any acceleration or impact. These are assumed to be produced by the intended use or occupancy of the building including weights of movable partition *or* furniture etc. The imposed loads to be assumed in buildings are given in *Appendix-A Table A-2* as per IS:875 (Part-2)[2.1].

The floor slabs have to be designed to carry either uniformly distributed loads and/or concentrated loads whichever produce greater stresses in the part under consideration. Since it is unlikely that any one particular time all floors will not be simultaneously carrying maximum loading, the code permits some reduction in imposed loads in designing columns, load bearing walls, piers their supports and foundations. They are given in *Fig. 2.2.1*

2.2.3 Impact Load

Impact load is caused by vibration or impact or acceleration. A person walking produces a live load but soldiers marching or frames supporting lifts and hoists produce impact loads. Thus, impact load is equal to imposed load increased by some percentage (called impact factor or impact allowance) depending on the intensity of impact.

2.2.4 Wind Load

Wind load is primary horizontal load caused by movement of air relative to earth. The details of design wind load are given in IS : 875 (Part - 3)[2.2]

Wind load is required to be considered in design especially when the height of the building exceeds two times the dimensions transverse to the exposed wind surface. For low rise building say up to 4 to 5 storeys the wind load is not critical because the moment of resistance provided by the continuity of floor system to column connection and walls provided between columns are sufficient to resist the effect of these forces. Further in limit state method, the factor for design load is reduced to 1.2 *(DL + LL + WL)* when wind is considered, as against the factor of 1.5 *(DL + LL)* when wind is not considered.

2.2.5 Earthquake Load

Earthquake loads are horizontal loads caused by earthquake and shall be computed in accordance with IS:1893[2.3] For monolithic reinforced concrete structures located in seismic zone II, and III with not more than 5 storey high, the seismic forces are not critical (See IS:13920 *Sect. 1.1*[2.4]) .

2.3 CHARACTERISTIC LOAD

Since the loads are variable in nature they are determined based on statistical approach. But it is impossible to give a guarantee that the loads cannot exceed during the life span of the structure. Thus, the characteristic value of the load is obtained based on statistical probabilistic principles from mean value and standard deviation.

The characteristic load is defined as that value of that load which has 95% probability of not being exceeded during the service span of the structure. However, this requires large amount of statistical data. But since such data are not available, Code recommends to take the working loads or service loads based on past experience and judgement and are to be taken as per IS:875[2.1] and IS:1893[2.3] Codes.

2.4 DESIGN LOAD

The variation in loads due to unforeseen increase in loads, constructional inaccuracies, type of Limit State etc. are taken into account to define the design load.

The design load is given by : Design load = γ_f x characteristic load *(clause:36.3.2)*

where , γ_f = partial safety for loads given in *Table 2.4.1*

Table 2.4.1 : Partial Safety Factor (γ_f) for Loads (according to IS : 456-2000)

Load Combination	*Limit State of Collapse*			*Limit State of Serviceability*		
	DL	*IL*	*WL*	*DL*	*IL*	*WL*
DL + IL	1.5	1.5	--	1.0	1.0	--
DL + WL	1.5 or 0.9 *	--	1.5	1.0	--	1.0
DL + IL + WL	1.2	1.2	1.2	1.0	0.8	0.8

* *This value is to be considered when stability against overturning or stress reversal is critical.*

Notes: *(1) DL = dead load , IL = Imposed load or Live load , WL = wind load.*
(2) While considering earthquake effects, substitute EL for WL
(3) Since the serviceability relates to the behavior of structure at working load the partial safety factors for limit state of serviceability are unity
(4) For limit state of serviceability, the values given in this table are applicable for short term effects. While assessing the long-term effects due to creep, the dead load and that part of the live load likely to be permanent may only be considered. For details see " Limit State Theory and Design of Reinforced Concrete" Chapter 8

2.5 CRITICAL LOAD COMBINATIONS *(clause 22.4.1)*

While designing a structure, all load combinations, in general, are required to be considered and the structure, is designed for the most critical of all.

As discussed in the earlier section, since for buildings up to 4 storeys, wind load is not considered, the elements are required to be designed for critical combination of dead load and live load only.

For deciding critical load arrangements, we are required to use maximum and minimum loads. For this, Code prescribes different load factors as given below :

Maximum load $= w_{max} = 1.5\ (DL + LL)$ ... ...(2.5.1)

Minimum load $= w_{min} = DL$ ... ...(2.5.2)

The *maximum positive moment* producing tension at the bottom will occur when the deflection is maximum *or* curvature producing concavity upwards is maximum. This condition will occur when maximum load (*i.e* both *DL* and *LL*) covers the whole span while minimum load *(i.e. only DL)* is on adjacent spans.

The *negative moments* producing tension at the top will be maximum when the curvature at support producing convexity upwards is maximum which requires maximum load should be applied on adjacent spans. Accordingly, IS:456 recommends the following loading arrangements on structural frames :

(a) Consideration may be limited to combination of :

(1) Design dead load on all spans with full design live loads on two adjacent spans (for obtaining maximum hogging moment. *Fig. 2.5.1b*

(2) Design dead load on all spans with full design imposed load and on alternate spans (to get maximum span moment.) . *Fig. 2.5.1a*

(3) When design imposed load does not exceed three-fourths of the design dead load, the load arrangement may be design dead load and design imposed load on all the spans.

The loading arrangement giving maximum span moment, say span *AB* is shown in *Fig. 2.5.1a* and *Fig. 2.5.1b* gives the loading arrangements for maximum negative moment at support *B*.

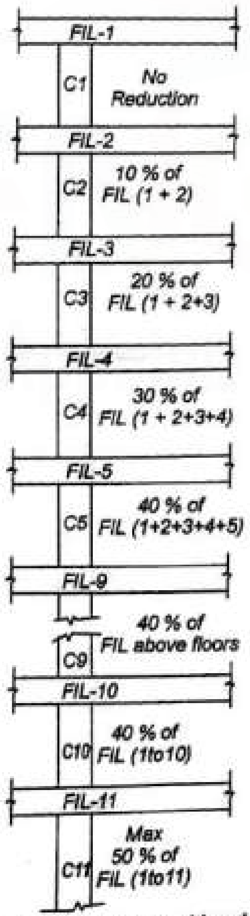

Fig. 2.2.1 Reduction in Imposed Loads

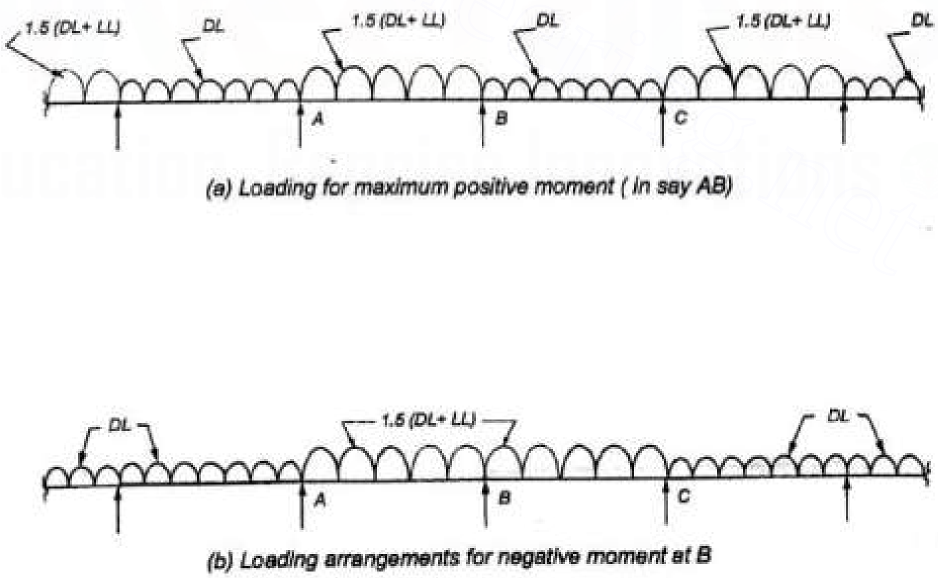

Fig. 2.5.1 : General Loading arrangements for maximum moments

2.6 GENERALIZED METHOD FOR COMPUTATION OF MAXIMUM SPAN MOMENT AND POINTS OF CONTRAFLEXURES [2.5]

When the loading arrangement specified by the code is considered for the analysis of a continuous beam simply supported at ends and loaded by *UDL*, the maximum bending moment does not occur at the mid-span of penultimate span but it occurs at a small distance away from the mid-span towards the simply supported end.

In general, the analysis of a structure using any method of analysis gives the end forces consisting of axial force, bending moment and shear force (*or* reaction). The next step is to analyze the beam to determine the maximum span moment and points of contraflexures, if any, and then to design the beam at different sections. The end sections subjected to hogging moments require provision of steel at top while the mid span section will require steel at bottom face to resist sagging moment. In some cases the end moments are so large that the negative (*or* hogging) moment may prevail over the whole span and it would be necessary to provide steel at top only *or* in some cases even negative reaction may develop at one of the ends for which proper anchoring arrangements may become necessary. Further, in some cases the end moments may be zero *or* only one end may be subjected to moment because of the other end being simply supported (*e.g.* penultimate span of a continuous beam) *or* end moments having the same or different magnitudes may act at both ends (*e.g.* intermediate span of a continuous beam). In order to consider all these probabilities it is necessary to derive generalized equations for calculation of span moment and points of contraflexures for a beam loaded by a uniformly distributed load subjected to end moments.

Consider a beam *AB* of span *L* loaded by a uniformly distributed load of intensity *w, and* subjected to end moments M_A and M_B as shown in *Fig. 2.6.1.* It is assumed that M_A is greater than M_B.

x_1, x_2 = distances of the points of contraflexures from end *A*.
L_o = Length of beam between points of contraflexures
x_{max} = distance of the section from *A* at which sagging moment is maximum.
M_{max} = maximum sagging moment occurring at distance x_{max} from end *A*.
R_A, R_B = end shear forces.

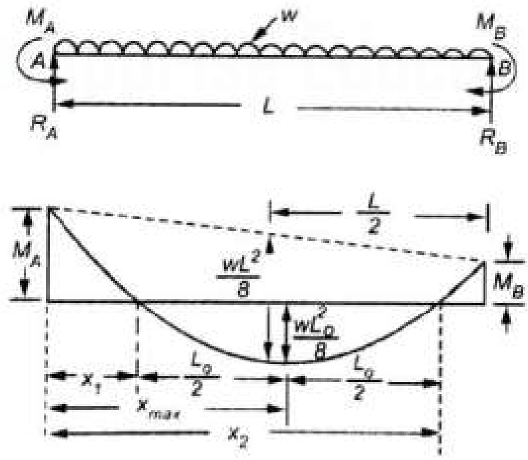

Fig.2.6.1 Sagging and Hogging Moments in Beam

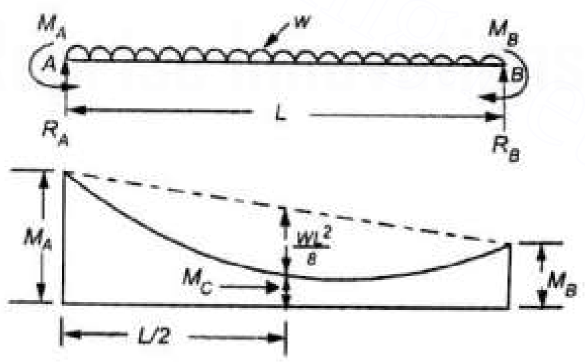

Fig.2.6.2: Negative moments in Beam

$$x_{max} = x_1 + L_o/2 = R_A/w \quad or \quad R_A = w\ x_{max} \quad or \quad x_{max} = R_A/w \qquad \text{..... (2.6.1)}$$

$$M_{max} = R_A\ x_{max}/2 - M_A = wL_o^2/8 \quad \text{For } M_A = 0, \quad M_{max} = R_A\ x_{max}/2 \qquad \text{..... (2.6.2)}$$

$$L_o = 2\sqrt{x^2_{max} - \frac{2\,M_A}{w}} \quad \text{... ... (2.6.3)}$$

$$x_1 = x_{max} - \sqrt{x^2_{max} - \frac{2\,M_A}{w}} = \frac{R_A}{w} - \sqrt{\left(\frac{R_A}{w}\right)^2 - \frac{2M_A}{w}} \quad \text{... ... (2.6.4)}$$

$$x_2 = x_{max} + \sqrt{x^2_{max} - \frac{2\,M_A}{w}} = \frac{R_A}{w} + \sqrt{\left(\frac{R_A}{w}\right)^2 - \frac{2M_A}{w}} \quad \text{... ... (2.6.5a)}$$

$$FOR \quad M_A = 0,\ x_1 = 0 \text{ and } x_2 = 2 \times x_{max} \quad \text{... ... (2.6.5b)}$$

The bending moment will be negative throughout the span, (See Fig. 2.6.2), if one of the following conditions is satisfied :

Condition - 1 : If $\left[\left(\frac{R_A}{w}\right)^2 - \frac{2M_A}{w}\right]$ is negative. ... ... (2.6.6a)

Condition - 2 : If either R_A *or* R_B acts in the downward direction irrespective of *condition-1* is satisfied *or* not. ... ... (2.6.6b)

In these cases the span moment is calculated at mid-span at a distance *L/2* from the support.
For details of derivation of equations see reference [2.5]

2.7 PROPERTIES OF CONCRETE

2.7.1 Grade of Concrete

Concrete is known by its grade which is designated as M15, M20, M25 to M100 at difference of 5 N/mm². in which letter *M* refers to concrete Mix and the number 15, 20, 25 etc. denotes the specified compressive strength (f_{ck}) of 150*mm* size cube at 28 days, expressed in *N/mm²*. Thus, concrete is known by its compressive strength. In R.C. work, M20, M25 grades of concrete are common, but higher grades of concrete should be used for severe, very severe and extreme environment as specified in *Appendix-C, Table C-1*

2.7.2 Compressive Strength

The strength of concrete is also a quantity which varies considerably even though concrte is cast from the same mix. Therefore, a single representative value, known as characteristic strength, is arrived at using statistical probabilistic principles.

***(a) Characteristic Strength** (clause: 36.1)*

It is defined as that value of the strength below which not more than 5% of the test results are expected to fall, (*i.e.* there is 95% probability of achieving this value, *or* only 5% probability of not achieving the same).

(b) Characteristic Strength of Concrete in Flexural Member

It may be noted that the strength of concrete cube does not truly represent the strength of concrete in flexural member because factors namely, the shape effect, the size effect, the prism effect (ratio *h/a* of the specimen), state of stress in a member, and casting and curing conditions for concrete in test specimen differ considerably from those of concrete in the member. Taking this into consideration, the characteristic strength of concrete in a flexural member is taken as 0.67 times[2.6] concrete cube strength.

(d) Design Strength (f_d) and Partial Safety Factor (γ_m) for Material Strength *(clause:36.3.1)*
The strength to be taken for the purpose of design is known as *design strength* and is given by

$$\text{Design Strength } (f_d) = \frac{\text{Characteristic strength } (f_{ck})}{\text{Partial Safety factor for material strength } (\gamma_m)}$$

The value of γ_m depends upon the type (in fact, reliability) of material and upon the type of limit state. According to I.S.Code,

$\gamma_m = 1.5$ for concrete , and $\gamma_m = 1.15$ for steel.

$\therefore$ Design strength of concrete in member $= 0.67 f_{ck} / 1.5 = 0.446 f_{ck} \cong 0.45 f_{ck}$

2.7.3 Tensile Strength (f_{cr}) *(clause: 6.2.2)*

The estimate of flexural tensile strength or the modulus of rupture or the cracking strength of concrete from cube compressive strength is obtained from the relation :

$$f_{cr} = 0.7 \sqrt{f_{ck}} \; N/mm^2 \qquad \text{... ... (2.7.1)}$$

The tensile strength of concrete in direct tension is obtained experimentally by split cylinder strength as described in IS:5816. It varies between 1/8 to 1/12 of cube compressive strength.

The test for flexural strength of concrete shall be carried out as per IS : 516

2.7.4 Creep *(clause:6.2.5)*

Creep is defined as the plastic deformation under sustained load.

The ultimate creep strain is estimated from the creep coefficient θ given by :

$$\theta = \text{creep strain/elastic strain} = \varepsilon_{cc} / \varepsilon_i \qquad \text{... ...(2.7.2)}$$

Creep strain ε_{cc} depends primarily on the duration of sustained loading. According to the Code, the value of ultimate creep coefficient is 1.6 at 28 days of loading.

2.7.5 Shrinkage *(clause: 6.2.4)*

The property of diminishing in volume during the process of drying and hardening is termed shrinkage.

It depends mainly on the duration of exposure. If this strain is prevented, it produces tensile stress in the concrete, and hence concrete develops cracks. The shrinkage is measured by shrinkage strain, ε_{cs}

I.S.Code prescribes the ultimate shrinkage strain $\varepsilon_{cs} = 0.0003$ for design purposes.

2.7.6 Short-term Modulus of Elasticity (E_c) *(clause: 6.2.3.1)*

The secant modulus obtained by testing a concrete specimen at 28 days under specified rate of loading is known as *short-term modulus of elasticity* because inelastic deformations under this loading are practically negligible.

According to the Code, short-term static modulus of elasticity of concrete is given by :

$$E_c = 5000 \sqrt{f_{ck}} \; N/mm^2 \qquad \text{... ...(2.7.3)}$$

2.7.7 Long-term Modulus of Elasticity (E_{ce}) *(clause: C - 4.1)*

The effect of creep and shrinkage is to reduce the modulus of elasticity of concrete with time. Therefore, the *long-term modulus of elasticity* of concrete takes into account the effect of creep and shrinkage and is given by :

$$E_{ce} = \frac{E_c}{1+\theta} \qquad \text{... ... (2.7.4)}$$

where, E_{ce} = long-term modulus of elasticity
E_c = short-term modulus of elasticity
θ = creep coefficient

Effect of the reduction in E_{ce} with time is to increase deflections and cracking with time. It, therefore, plays a very important role in limit state of serviceability and in calculations of deflection and cracking.

It is further noted that as E_c changes, modular ratio E_s / E_c charges with time. Thus, the working stress method which takes a single value of modular ratio m does not represent the true strength and behavior of concrete members where E_s = modulus of elasticity of steel.

2.7.8 Modular Ratio

Modular ratio consists of (a) Short-term modular Ratio and (b) *Long-term modular Ratio*

(a) *Short-term modular Ratio* is the ratio of modulus of elasticity of steel to modulus of elasticity of concrete (clause 6.2.3.1).

$$\text{Short term modular ratio} = E_s / E_c \qquad \text{... ... (2.7.5)}$$

where, E_s = modulus of elasticity of steel = 2×10^5 N/mm^2

E_c = short term stastic modulus of Elasicity $5000\sqrt{f_{ck}}$ N/mm^2

(b) *Long-term modular Ratio* (clause B-2.1.2d)

As the modulus of elasticity of concrete changes with time, age at loading etc., the modular ratio also changes accordingly. I.S. Code gives the following expression for the *Long-term modular ratio*, taking into account the effects of creep and shrinkage partially.

$$\text{Long-term modular ratio} = m = \frac{280}{3\,\sigma_{cbc}} \qquad \text{... ... (2.7.6)}$$

where, σ_{cbc} = permissible compressive stress due to bending in concrete in N/mm^2.

This modular ratio is useful only in the working stress design. It is also required for calculating the properties of transformed R.C. members for serviceability.

The values of modular ratio based on *Eq. 2.7.5* and that based on *Eq. 2.7.6* for different grades of concrete are given in *Table 2.7.1*.

Table 2.7.1 : Modular Ratio for Different Grades of Concrete

Grade of Concrete	*Modular Ratio 'm'*	
	Short-term	*Long-term*
M 20	8.9	13.3
M 25	8.0	11.0

2.7.9 Poisson's Ratio

Poisson's ratio varies between 0.1 for high strength concrete and 0.2 for weak mixes. It is normally taken equal to 0.15 for strength design and 0.2 for serviceability criteria. Many times the Poisson's ratio is totally neglected with little error in strength calculations.

2.7.10 Durability [2.7, 2.8] *(clause: 8)*

Durability of concrete is its ability to resist its disintegration and decay.

One of the chief characteristics influencing durability of concrete is its permeability to ingress of water and other potentially deleterious materials. The desired low permeability in concrete is achieved by having an adequate cement, sufficiently low water/cement ratio, by ensuring full compaction of concrete and by adequate curing. Hence, according to exposure conditions minimum cement content and maximum water/cement ratio, have been specified by IS:456 in *Table 5.*

The exposure conditions in working life, nominal cover, and minimum grade of concrete for R.C.C.work *(Contained in Table.3, Table.5, Table.16 and cl. 26.1.4* of IS:456) are all given in *single Table C-1* in *Appendix C*

2.7.11 Unit Weight of Concrete *(clause: 19.2.1)*

The unit weight of reinforced concrete depends on percentage of reinforcement, type of aggregate, amount of voids and varies from 23 to 26 kN/m^3. The unit weight of plain and reinforced concrete, as specified by IS:456 are 24 kN/m^3 and 25 kN/m^3 respectively.

2.7.12 Stress - Strain Curve

The typical idealized stress-strain curve adopted by IS code is shown in *Fig. 2.7.1.* It consists of a parabola for the initial ascending part up to a strain of 0.002 followed by a horizontal line terminating at an ultimate strain of 0.0035

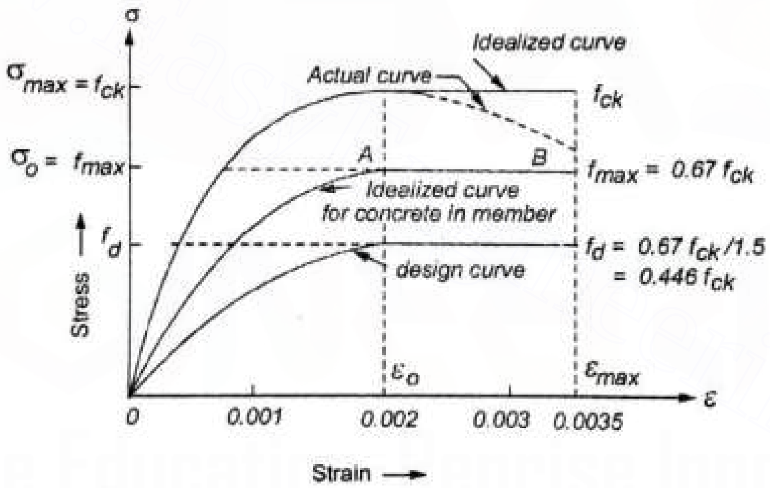

Fig. 2.7.1 Stress-Strain Curves for Concrete

The equation of an idealized stress-strain curve is given by :

$$\sigma = \left[\frac{2\,\varepsilon}{\varepsilon_o} - \left(\frac{\varepsilon}{\varepsilon_o}\right)^2\right] \sigma_o \quad \text{for } o < \varepsilon < \varepsilon_o$$

$$\text{and} \quad \sigma = \sigma_o \quad \text{for } \varepsilon_o \leq \varepsilon < \varepsilon_{max} \qquad \dots\dots (2.7.7)$$

where, ε = strain at any point

σ = stress at any point

ε_o = strain at which parabolic part ends = 0.002

σ_o = idealized maximum stress corresponding to ε_o

2.8 CONCRETE MIX PROPORTIONING *(clause: 9)*

The structural designer specifies the strength and properties of concrete assumed in design. The engineer is required to proportion the various ingredients making concrete so that the resulting mix has proper *workability for placing and gives the desired strength.*

The proportioning of concrete is done by any of the following ways :

(a) By designing concrete mix; such a concrete is called *Design mix concrete* 2.9, 2.10, 2.11

(b) By adopting nominal concrete mix; called *as Nominal mix concrete* to be used for M20 and lower.

The former is used for large and important works while the later is used for medium type routine concrete construction. Normally, it is desirable to proportion the ingredients by weight. However, for small routine jobs, it is conveniently being done by volume.

Mixing shall be continued until there is a uniform distribution of materials and mass is uniform in color and consistency. As a guidance, the mixing shall be at least 2min for conventional free fall batch type concrete mixtures.

2.9 CURING AND STRIPPING TIME FOR STRIKING OF FORMWORK *(clause:13.5 and 11.3)*

The concrete after casting shall be cured under moist condition for a minimum period of 7 days and it shall be kept in forms till the concrete attains the strength of at least twice the stress to which the concrete would be subjected at the time of removal of the formwork. In normal circumstances, and where ordinary portland cement is used, forms may be removed after the expiry of the periods given in *Table 2.9.1*. The props under the slabs and beams shall be removed in such a sequence as to effect the same type of structural action and support condition as envisaged in design. *For example*, the props under the cantilever beam shall be removed starting from free end of cantilever towards the fixed end sequentially.

Table 2.9.1 : Minimum Curing Period and Stripping Time for Striking of Formwork

Sr. No.	*Members*	*Period*
1.	Walls, columns and vertical faces of all structural members	16 to 24 hours
2.	Soffit formwork to slabs (Props to be refixed immediately after removal of formwork)	3 days
3.	Soffit formwork to beams (Props to be refixed immediately after removal of formwork)	7 days
4.	Removal of props under slabs :	
	(a) Spanning up to 4.5 *m*	7 days
	(b) Spanning over 4.5 *m*	14 days
5.	Removal of props under beams and arches :	
	(a) Spanning up to 6 *m*	14 days
	(b) Spanning over 6 *m*	21 days

2.10 REQUIREMENTS FOR STATISTICAL DETERMINATION OF CHARACTERISTIC STRENGTH *(clause: 15)*

(a) Sampling : A random sampling procedure shall be adopted to ensure that each concrete batch has reasonable chance of being tested. The sampling shall be spread over the entire period of concreting. The frequency of sampling will depend upon the nature of work , the volume of concrete, and the importance of the location of concrete from view point of stress condition. For Example, higher rate of sampling will be required for highly stressed structural member (like column). Also it will be appropriate to have higher rate of sampling and testing at the start of the work to establish the level of confidence in the quality of concrete at the earliest. The minimum frequency of sampling of concrete of each grade at each time shall be decided from the volume of concrete as given in *Table 2.10.1*.

Table 2.10.1 : Number of Samples for Testing

Quantity of concrete in work in m^3	1-5	6-15	16-30	31-50	Above 50
Minimum number of samples	1	2	3	4	$4 + x$

where, x is the number based on the rate of 1 additional sample for each additional 50 m^3 or part thereof.

(b) Test Specimens : Minimum of three test specimens shall be made from each sample of concrete for testing strength at 28 days. Additional specimens may be made for other tests like 7 days test or modulus of rupture test, etc. The average strength of the 3 specimens shall be called the sample test strength. The specimen shall be tested as described in IS : 516.[2.12]

(c) Standard Deviation : When sufficient test results for a particular grade of concrete are not available, the standard deviation given in *Table 2.10.2* may be assumed for design of concrete mix to start with (clause. 9.2.4.2). As soon as the results of samples are available, actual calculated value of standard deviation should be used. However, when past records of similar mix or grade of concrete are available, the standard deviation obtained from these records may be allowed.

Table 2.10.2 Assumed Standard Deviation

Grade of Concrete	*Assumed Standard deviation* N/mm^2	*Remarks*
M10, M15	3.5	These values have been specified assuming that there is site control, proper storage of cement, weight batching, controlled addition of water, regular checking of all materials, workability and strength. Where there is deviation from the above, the values of standard deviation shall be increased by 1 N/mm^2.
M20, M25	4.0	
M30, M35, M40, M45, M50, M55, M60	5.0	

(d) Characteristic Strength : This shall be obtained from known value of mean strength and the standard deviation.

2.11 ACCEPTANCE CRITERIA FOR CONCRETE *(clause :16)*

(a) Compressive strength : The concrete shall be deemed to comply with the strength requirements when both the following conditions are met.

(i) The mean strength determined from any group of four non-overlapping consecutive test results complies with the appropriate limits in column *A* of *Table 2.11.1*

(ii) Any individual test result complies with the appropriate limits in column *B* of *Table 2.11.1*.

Table 2.11.1 Characteristic Compressive Strength Compliance Requirement

Specified Grade	*A*	*B*
	The mean of the group of 4 non overlapping consecutive test results in N/mm^2	Any individual test results in N/mm^2
M 15	$\geq (f_{ck} + 0.825s)$ or $\geq (f_{ck} + 3)\ N/mm^2$	$\geq (f_{ck} - 3)\ N/mm^2$
M 20 or more	$\geq (f_{ck} + 0.825s)$ or $\geq (f_{ck} + 3)\ N/mm^2$	$\geq (f_{ck} - 3)\ N/mm^2$

where, s = established standard deviation rounded off to nearest 0.5 N/mm^2

Note:
(i) If established standard deviation is not known, the values given in Table 2.10.2 may be assumed.
(ii) Efforts should be made to establish the value of standard deviation from test results of 30 samples as early as possible.

(b) Flexural Strength : When both the following conditions are met the concrete complies with the specified flexural strength.

(i) The mean strength determined from any group of four consecutive test results exceeds the specific characteristic strength by at least 0.3 N/mm^2.

(ii) The strength determined from any test result is not less than the specified characteristic strength by less than 0.3 N/mm^2.

2.12 NON - DESTRUCTIVE TESTING OF STRUCTURES[2.12]

The nondestructive testing consists of following tests :

(a) Core test : Minimum three cores of concrete shall be prepared and tested as per IS:516[2.12]. The average cube strength of core should not be less than 85% of cube strength of concrete grade specified for the corresponding age and no individual core has strength less than 75%. If the above mentioned requirements are not satisfied or where it is not possible to take the cores the load test should be performed.

(b) Load test : The load test consists of actually loading the part of the structure and observing the deflection. The details are covered up in *Clause 17.6* of IS:456.

(c) The other nondestructive tests carried out near the surface of the structural member comprises of:

(1) Ultrasonic pulse velocity measurement at surface, IS:13311[2.13] (Pt.-1)

(2) Rebound Test using Schmidt hammer IS :13311(Part-2)[2.14]

2.13 DESIGN STRENGTH OF CONCRETE

The values of characteristic strengths and design strengths of concrete for the limit state of collapse and the limit state of serviceability for different structural actions, namely, axial tension and compression, bending tension and compression, shear, bond and bearing are given in *Table 2.12.1*.

Table 2.13.1 Characteristic Compressive Strength and Design Strength of Concrete for Limit State Method

Type of Structural Action	*Grade of Concrete*	
	M20	*M25*
Characteristic compressive strength (f_{ck})	20.00 N/mm^2	25.00 N/mm^2
Design strength in -		
- direct compression ($0.4 f_{ck}$)	08.00 N/mm^2	10.00 N/mm^2
- bending compression ($0.446 f_{ck}$)	08.92 N/mm^2	11.15 N/mm^2
- flexural tension ($0.7\sqrt{f_{ck}}$)	03.13 N/mm^2	03.50 N/mm^2
- average bond for plain bars in tension	01.20 N/mm^2	01.40 N/mm^2
- bearing ($0.45 f_{ck}$)	09.00 N/mm^2	11.25 N/mm^2

Notes :

1. *Direct compressive strength 0.4 f_{ck} given above is for axially loaded columns only, taking into account the effect of a minimum eccentricity. For pure axial compression, value is the same as that for bending compression, namely, 0.446 f_{ck}.*
2. *For deformed bars confirming to IS : 1786 value of the bond stress shall be increased by 60%. For bars in compression, the value of bond stress shall be increased by 25%.*
3. *Design strength in diagonal tension or in shear is a function of percentage of tension steel and grade of concrete. The values of design shear strength of concrete for different percentages of steel and grades of concrete have been given in Table 4.4.1*

2.14 PROPERTIES OF REINFORCING STEEL

Reinforcing steel is known by its grade, and the reinforcement, consisting usually of round bars, is known by the type of bar (whether plain *or* deformed).

2.14.1 Grade of Steel

Grade of steel is known by its characteristic yield strength and is designated as Fe 250, Fe415, and Fe500 where *'Fe'* stands for Ferrous metal and the number following it represents guaranteed yield strength in N/mm^2. At present, steel is commercially available in the above three grades.

2.14.2 Types of Bars *(clause: 5.6)*

Bars used as reinforcement in R.C. construction are available in the following types :

(i) Plain round bars of mild steel (grade Fe250),

(ii) High Yield Strength Deformed (HYSD) bars[2.15] (of grades Fe415 and Fe500, Fe550).

HYSD bars have ribs, lugs on their surfaces. They are manufactured by the process of hot rolling *or* by cold working.

In the cold working process the bars having deformations are twisted and twisting process is continued till the strains are in the elasto-plastic range and then unloaded. During this process the bars get hardened with the result stresses increase but ductility decreases.

The hot rolled bars are termed as TMT (Thermo-mechanically treated bars) are manufactured by sophisticated and controlled cooling , converting the outer surface of the bar in to hardcore and inner portion softens . This process increases yield strength, bond characteristicsand ductility . Due to higher ductility and weldability they are more economical and useful in R.C. structures and earthquake prone zones .

Fig. 2.14.1 shows typical stress-strain curve obtained from tension test for cold worked high strength deformed bars.

The HYSD bars do not exhibit a well defined yield point, and hence 0.2% proof stress is considered as yield stress. It is that value of the stress corresponding to the point of intersection of the stress-strain curve and the line *BC* drawn from residual (initial) strain of 0.002 (i.e. 0.2%) at *B* and parallel to the line *OA* which is tangent to the curve at the origin. as shown in *Fig. 2.14.2.*

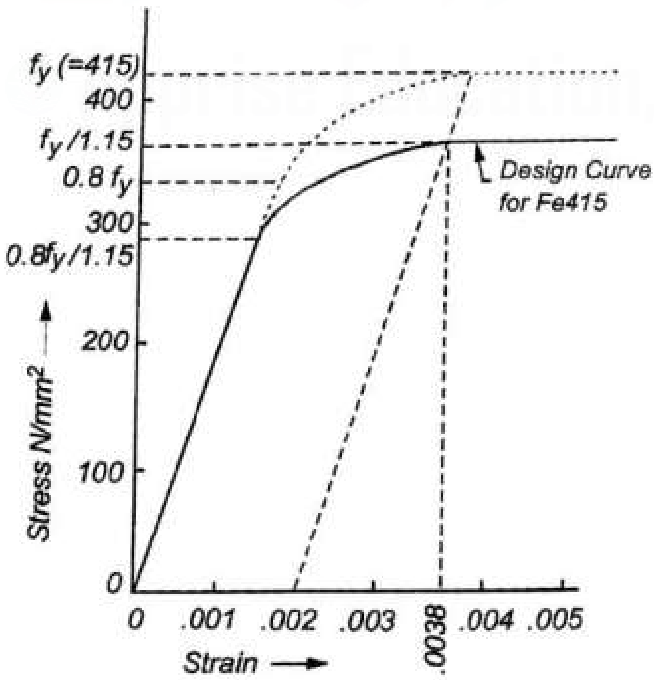

Fig. 2.14.1 Stress-strain curve for Fe415

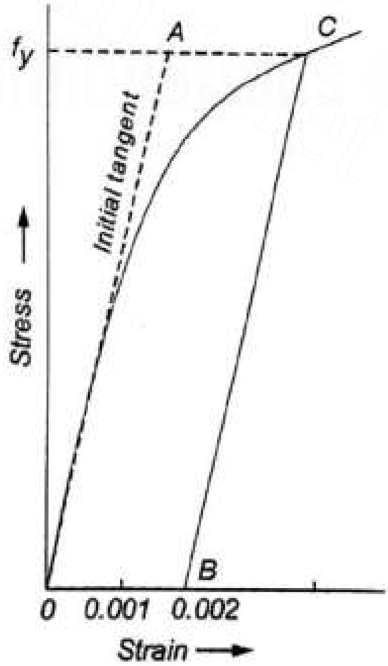

Fig. 2.14.2 Determination of Yield stress from Tension Test

2.14.3 Structural Specifications

The minimum yield stress (or 0.2% proof stress), the minimum percentage elongation at failure, ultimate stress etc. for the different grades of available steel are given in *Table - 2.14.1* for ready reference. The modulus of elasticity of steel is taken as 2×10^5 N/mm²

Table 2.14.1 Types and Grades of Reinforcing bars

Sr. No.	Type of steel	Indian standard	Bar Diameter	Yield stress N/mm²	Minimum elongation	Ultimate stress N/mm²
1.	Mild steel grade-I	IS: 432[2.16] Part I	≤ 20 *mm*	250	23%	410
			Over 20 *mm*, up to and including 50 *mm*	240	23%	410
2.	Mild steel grade-II	IS: 432 Part I	≤ 20 *mm*	225	23%	370
			Over 20 *mm*, up to and including 50 *mm*	215	23%	370
3.	Medium Tensile steel	IS: 432 Part I	≤ 16 *mm*	350	20%	540
			Over 16 *mm*, upto and including 32 *mm*	340	20%	540
			Over 32 *mm*, Up to and including 50 *mm*	330	20%	510
4.	High Strength Deformed Bars	IS: 1786[2.15]	All sizes	415	14.5%	1.1 x *y.s.* but ≮ 485 *N/mm²*
				500	12%	1.08 × *y.s.* but ≮ 545 *N/mm²*
				550	8%	1.06 × *y.s.* but ≮ 585 *N/mm²*

2.14.4 Stress Strain Relation

For steel grade Fe250, $f_s = \varepsilon_s . E_s$

For steel grade Fe415 $f_s = \varepsilon_s . E_s$ up to a strain of 0.00144

For steel grade Fe 500, $f_s = \varepsilon_s . E_s$ up to a strain of 0.00174

Where, E_s = modulus of elasticity of steel = 2×10^5 *N/mm²*

For Fe 415 for strains above 0.00144, the stresses corresponding to strains are given in *Table. 2.14.2*

Table - 2.14.2 Values of Stresses f_s (N/mm^2) for different values of strains for Steel Grade Fe 415

$\varepsilon_s \times 10^5$	f_s	$\varepsilon_s \times 10^5$	f_s	$\varepsilon_s \times 10^5$	f_s	$\varepsilon_s \times 10^5$	f_s	$\varepsilon_s \times 10^5$	f_s	$\varepsilon_s \times 10^5$	f_s
141	282.10	181	317.93	221	335.45	261	347.94	301	353.99	341	357.49
142	284.30	182	318.56	222	335.82	262	348.20	302	354.07	342	357.57
143	286.50	183	319.18	223	336.19	263	148.46	303	354.16	343	357.66
144	288.70	184	319.81	224	336.56	264	348.71	304	354.25	344	357.75
145	289.65	185	320.43	225	336.92	265	348.97	305	354.34	345	357.84
146	290.59	186	321.06	226	337.29	266	349.22	306	354.42	346	357.92
147	291.54	187	321.68	227	337.66	267	349.49	307	354.51	347	358.01
148	292.49	188	322.30	228	338.02	268	349.74	308	354.60	348	358.10
149	293.44	189	322.93	229	338.39	269	350.00	309	354.69	349	358.19
150	294.38	190	323.55	230	338.76	270	350.26	310	354.77	350	358.27
151	295.33	191	324.17	231	339.13	271	350.51	311	354.86	351	358.36
152	296.28	192	324.80	232	339.49	272	350.77	312	354.95	352	358.45
153	297.23	193	325.17	233	339.86	273	351.03	313	355.04	353	358.54
154	298.17	194	325.53	234	340.23	274	351.29	314	355.12	354	358.62
155	299.12	195	325.90	235	340.60	275	351.54	315	355.21	355	358.71
156	300.07	196	326.37	236	340.96	276	351.80	316	355.30	356	358.80
157	301.02	197	326.64	237	341.33	277	351.89	317	355.39	357	358.89
158	301.98	198	327.00	238	341.70	278	351.97	318	355.47	358	358.97
159	302.91	199	327.37	239	342.06	279	352.06	319	355.56	359	359.06
160	303.86	200	327.74	240	342.43	280	352.15	320	355.65	360	359.15
161	304.81	201	328.11	241	342.80	281	352.24	321	355.74	361	359.24
162	305.75	202	328.47	242	343.06	282	352.32	322	355.82	362	359.32
163	306.70	203	328.84	243	343.31	283	352.41	323	355.91	363	359.41
164	307.32	204	329.21	244	343.57	284	352.50	324	356.00	364	359.50
165	307.95	205	329.58	245	343.83	285	352.59	325	356.09	365	359.59
166	308.57	206	329.94	246	344.09	286	352.67	326	356.18	366	359.67
167	309.20	207	330.31	247	344.34	287	352.76	327	356.26	367	359.76
168	309.82	208	330.68	248	344.60	288	352.85	328	356.35	368	359.85
169	310.45	209	331.04	249	344.86	289	352.94	329	356.44	369	359.94
170	311.07	210	331.41	250	345.11	290	353.02	330	356.52	370	360.02
171	311.70	211	331.78	251	345.37	291	353.11	331	356.61	371	360.11
172	312.32	212	332.15	252	345.63	292	353.20	332	356.70	372	360.20
173	312.94	213	332.51	253	345.89	293	353.29	333	356.79	373	360.29
174	313.56	214	332.88	254	346.14	294	353.37	334	356.87	374	360.37
175	314.19	215	333.25	255	346.40	295	353.46	335	356.96	375	360.46
176	314.81	216	333.62	256	346.66	296	353.55	336	357.05	376	360.55
177	315.44	217	333.98	257	346.91	297	353.64	337	357.14	377	360.64
178	316.06	218	334.35	358	347.17	298	353.72	338	357.22	378	360.72
179	316.69	219	334.72	259	347.43	299	353.81	339	357.30	379	360.81
180	317.31	220	335.09	260	347.69	300	353.90	340	357.40	380	360.90

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CHAPTER - 3

ANALYSIS AND DESIGN APPROXIMATIONS

The following sections deal with the analysis of the structure *i.e* determination of the internal forces like axial compression, bending moment, shear force, twisting moment etc. in the component members, for which these members are to be designed, under the action of given external loads. This process requires the knowledge of Structural Mechanics which includes Mechanics of rigid bodies (*i.e* mechanics of forces), mechanics of deformable bodies (*i.e* mechanics of deformations) and theory of structures (*i.e* the science dealing with response of structural system to external loads). A brief review is taken of structural analysis to refresh the basic principles.

3.1 METHODS OF ANALYSIS

The different approaches to structural analysis are given below.

(1) Elastic Analysis based on Elastic Theory,

(2) Limit Analysis based on Plastic Theory or Ultimate Load Theory.

Normally, the elastic analysis is used in Working Stress (*or* permissible stress) Method of design *(WSM)*, and the Limit analysis is used in Ultimate Load or Ultimate strength Method of design *(ULM)*. The *modified version* of ultimate load method is called *Limit State Method.*

The the Limit State Method of design includes design for ultimate limit state at which ultimate load theory applies, and also for service state at which elastic theory applies, thus requiring study of both the theories. At the same time, one should not get confused between the limit state philosophy of design and limit analysis. The latter is a method of analysing a structure at collapse, while the former is a method of design for different limit states.

In this section, both these approaches of structural analysis will be briefly reviewed.

3.1.1 Elastic Analysis

Elastic analysis deals with the study of strength and behavior of the members and structures at working loads.

It is based on the following assumptions :

(i) Relation between force and resulting displacement is linear. (*i.e* Hook's Law is applicable.)

(ii) Displacements are extremely small compared to the geometry of the structure in the sense that they do not affect the analysis.

Methods of elastic analysis can be broadly classified as under :

(i) Classical Methods :

(a) Method of Consistent Deformation, (b) Slope-deflection Method, (c) Strain Energy Methods.

(ii) Relaxation/Iterative Methods :

(a) Moment Distribution Method, (b) Kani's Method.

(iii) Computer Methods :

(a) Matrix Method, (b) Finite Element Method, (c) Finite Difference Method.

(iv) Approximate Methods :

(a) Substitute Frame Method, (b) Cantilever Method, (c) Portal Method.

(v) Coefficient Method : Coefficients given in design hand books *or* Codes can be used to obtain bending moment, shear force etc.

With the availability and easy access to computers, the above methods will now be divided into two major groups. First group includes those methods which are more suitable for hand calculations for small works.

Method of Consistent Deformations, Moment Distribution Method, Kani's Method, Approximate Methods, and Coefficient Method come under this group. Substitute Frame Methods are suitable for analysing a building frame for vertical loads, while the Cantilever Method and the Portal Method are suitable for analysing the effects of horizontal loads on frames.

The second group is of the methods requiring the use of computers. Matrix Methods and Computer Methods described in Parts (iii) and (iv) above come under this group.

Since the scope of this book is restricted to design of (G + 3) storeyed building, the discussion is limited to use of substitute frame method for analysis of building frames for vertical loading. The Coefficient Method or the approach of determining the design forces (*e.g.* bending moment, shear force, axial loads etc.) by use of coefficients available for standard loading cases, is very common in building design for analysing simple frames and standard beams such as cantilever, simply supported, and continuous beams and slabs, and single bay single storeyed rectangular portal frames.

Since in many cases redistribution of moments is carried out therefore it is necessary to know the limit analysis and redistribution of moments. This is cursorily reviewed in the next subsection.

3.1.2 Limit Analysis

It is an analysis dealing with the study of strength and behavior of members and structure at collapse. It is based on plastic theory for structures made up of perfectly plastic material like steel, while it is based on ultimate load theory for structures of reinforced concrete, the behavior of which is characterized by crushing of concrete and yielding of steel at collapse. It must be borne in mind that this ultimate state is never allowed to be reached by the use of appropriate safety factors. However, the knowledge of strength and behavior at collapse is absolutely necessary to know the exact margin of safety.

Consider the behavior of a statically indeterminate fixed beam of span *L*, fixed at both ends *A*, *B* and carrying a uniformly distributed load of intensity *w*. The *maximum* bending moment occurs at ends *A* and *B* rather than at the mid-span. Now, the load is gradually increased till the collapse occurs. Initially the beam behaves elastically till the stress at any section reaches its yield value or ultimate moment capacity. The plastic hinges develop at ends due to plastification of concrete in compression, and cracking of concrete accompanied by yielding of steel in tension. The *UDL* has increased from w to w_1 at this *Stage - I*. The plastic hinges destroy full fixity at *A* and *B*. At this stage the collapse does not occur but the beam behaves as a hinged beam with constant moment M_u acting at *A* and *B*. *(See Fig. 3.1.1a)*

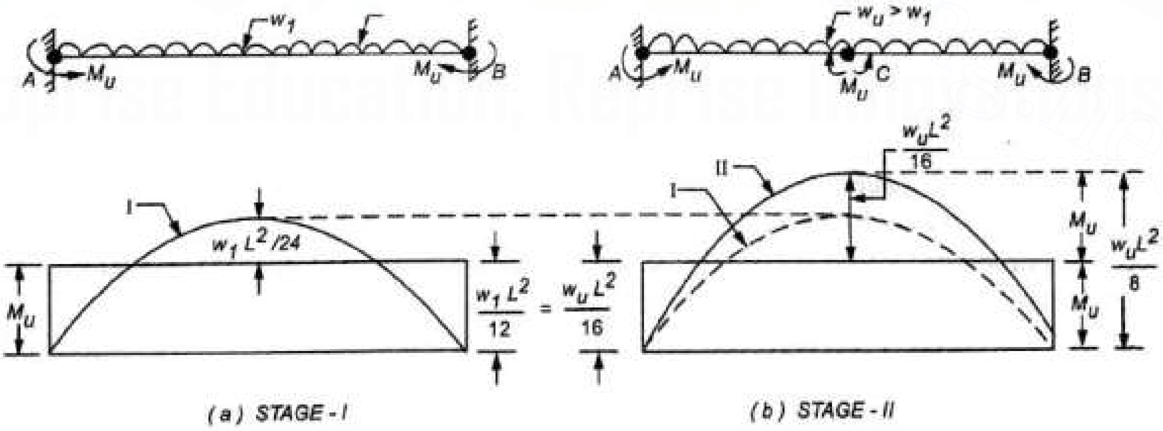

Fig. 3.1.1 Redistribution of Moments in Fixed Beam

$$M_A = M_B = w_1 L^2 / 12 = M_u \quad i.e.\ M_{uA} = M_{uB} = w_1 L^2 / 12 = M_{ur}$$

The corresponding moment at mid-span is just $= w_1 L^2/24$

On loading further, the moments at the fixed ends remain unchanged as the beam section at these locations cannot offer any additional resistance, but the moment in the span region increases. (Thus, additional load is resisted by span region only).

Stage - II is reached when the bending moment at mid-span reaches its ultimate moment capacity with the formation of hinge at *C* as shown in *Fig. 3.1.1b*. This causes division of beam into two segments *AC* and *CB* with rotations occurring at *A, B* and *C*. At this stage mechanism is said to have formed which leads to collapse of the beam.

The bending moment at mid - span $= M_{uC} = M_{ur} = w_u L^2/8 - M_{uA} = w_u L^2/8 - w_l L^2/12$

But $M_{uA} = M_{ur}$, $\therefore M_{ur} = w_u L^2/8 - M_{ur}$ or $M_{ur} = w_u L^2/16$

But M_{uA} is also equal to $w_l L^2/12$, $\therefore w_u L^2/16 = w_l L^2/12$

$$\therefore w_u = 1.33\, w_l$$

Thus, there is increase in load carrying capacity of the beam by 33% over the load w_l obtained by elastic analysis at *stage - I*. The merit of limit analysis lies in this fact that it gives higher load carrying capacity for an indeterminate structure due to redistribution of moments.

3.1.3 Advantages of Redistribution of Moments.

(1) In case of indeterminate structures, it helps to reduce bending moments in the peak regions, such as beam - column junction *or* supports of continuous beams, thereby the congestion of reinforcement is reduced making detailing and concreting easier.

(2) The reduction in support moment not only helps in reducing steel at supports but utilizes higher moment resisting capacity of a flanged section in the span region.

(3) It ensures under-reinforced failure since the depth of the neutral axis decreases with the increase in the percentage redistribution of moments.

(4) It gives better distribution of moments along the length of the member and makes detailing easier and gives economic design.

(5) It not only reduces the moment at support but, many times, it also does not increase the design moment at mid-span. This can be seen in the case of a continuous beam designed for maximum moments decided by bending moment envelope *i.e* maximum moment diagram obtained by consideration of all possible loading arrangements. (see Chapter-3 of Authors' Text book [3.1]).

The procedure for limit analysis involves the following. (clause 22.7 and 37.1.1)

(i) Elastic Analysis for ultimate (factored) loads.

(ii) Redistribution of moments subject to following conditions

(a) Equilibrium shall always be maintained *i.e* the sum of the support moment and the span moment shall be equal to the maximum span moment for a simply supported beam. Thus, for beam subjected to uniformly distributed load,

$$M_{sup} + M_{span} = w_u L^2/8$$

In brief, a reduction in support moment by dM must be accompanied by corresponding increase in the mid-span moment by $dM/2$.

(b) Amount of redistribution (dM = Elastic moment M_{EU} − assumed design moment M_{DU}) shall not exceed prescribed percentage given below.

- 30% in Limit state design

The limitations of 30% redistribution of moments have been imposed to avoid large rotation, and hence excessive deflection and cracking which affect the serviceability of the structure.

- 15 % to 20% redistribution may be normally taken as reasonable limit.[3.2 3.3]
- 10% in case of structures over 4 storey height to prevent lateral instability.

(c) The design moment M_{DU} shall not be less than 0.7 times the elastic moment M_{EU} at ultimate state.

Since $M_{EU} = 1.5 \times$ Working moment M_{Ew}, it means that

$$M_{DU} \geqslant 0.7 \times 1.5\, M_{EW} \;\; i.e \;\; M_{DU} \geqslant 1.05\, M_{EW}.$$

In other words the design moment after redistribution shall not be less than the working moment ($i.e. M_{EW}$) at service load.

(d) The depth of neutral axis shall be limited to

$$k_{u.limit} = x_{u.limit}/d <= (0.6 - dM/100) \quad or \quad k_{u.max} \text{ whichever is less.} \qquad \ldots\ldots (3.1.1)$$

This is required for satisfying the requirement of rotation capacity at a point where redistribution of moment is done.

3.2 ANALYSIS OF BUILDING FRAME

3.2.1 General

The structural frame of a building consists of floor and roof slabs, and supporting beams and columns. All the components of the frame are usually cast together forming a monolithic construction. The resulting frame acts as one integral unit. The monolithic casting of members and proper detailing enables to have rigid connections between the members so that every member acts integrally with the connected members. *The continuity between the members help to distribute the forces to number of connected members.* This enhances the reserve strength of the structure and eliminates the possibility of collapse of the structure due to failure of any component member on account of effect of localized loads and actions. Safety of the building as a whole is increased. The rigidity of the connection is also desirable *or* rather essential for resisting horizontal loads like wind load *or* earthquake load.

A typical frame of a multistoreyed building is shown in *Fig. 3.2.1a*. The frame consists of a continuous one-way slab S_1 cast monolithic with secondary beams B_1, B_2, and main beams B_3, B_4, B_5. The main beam is continuous over columns and is rigidly connected to them above and below that floor. The frame as a whole consists of number of members and number of rigid joints. The structure is highly statically indeterminate. The exact analysis of the entire frame by use of classical methods is beyond the capacity of manual/hand calculations. It can only be done by computer. Besides, formulation will involve large number of unknown displacements and large computer memory. The solution is also, therefore, costly and beyond the reach of a common designer. In fact such rigorous computer analysis will be required only for tall and unconventional irregular structures. Besides, in R.C. buildings, the actual conditions differ widely from the conditions assumed in theoretical analysis. The design of buildings of ground plus three storeys does not demand the use of rigorous classical analysis. The approximate methods are more than adequate.

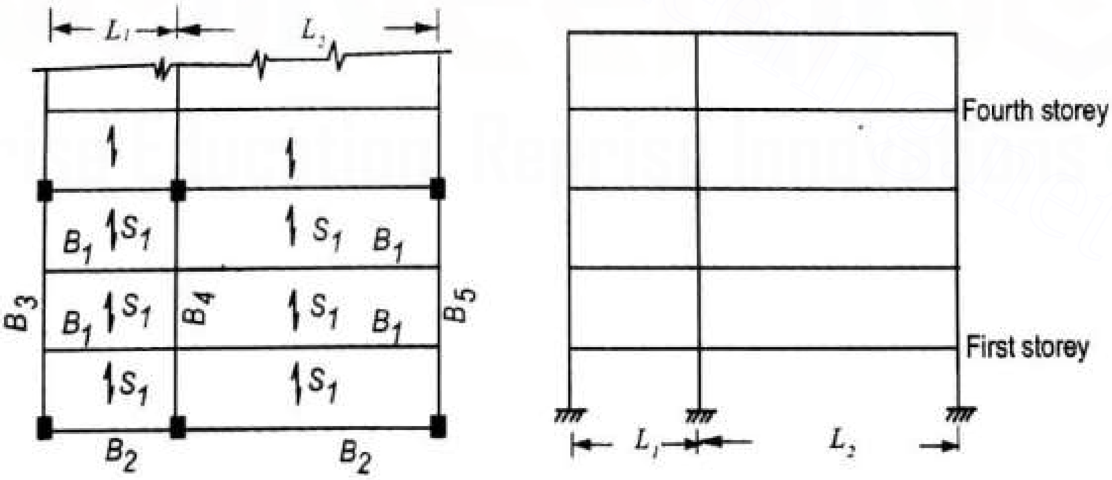

Fig. 3.2.1a Multistoreyed Building Frame.

The approximate methods are based on the principle of dividing the structure into parts and analysing only the part of interest, disregarding the effect of loads and resistances of members far away from the member of interest. The above simplification is based on the fact that a load on any member and its stiffness hardly affects a member which is two spans *or* two storeys beyond. For example, a unit moment applied at one end joint of a continuous beam produces a moment of only 27% at next *i.e* second joint, 7% at 3rd joint and only 2% at 4th joint *(see Fig. 3.2.1b)*. The assumed approximations reduce the computational efforts to a great extent without much affecting the accuracy, and the results obtained are on the safer side.

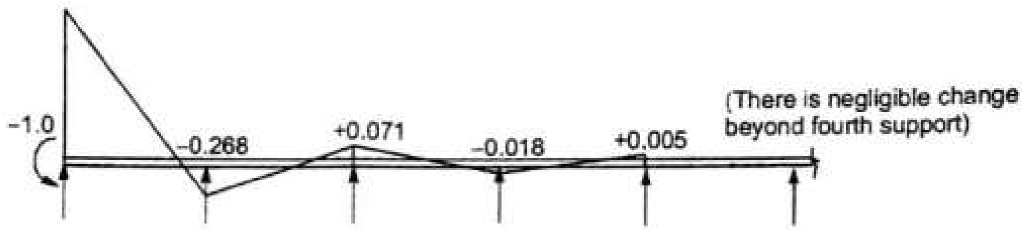

Fig. 3.2.1b

The approximate method, therefore, adopts some standardized small portions of the whole frame, known as *Substitute Frames* or Sub-frames principally consisting of the members of interest and other adjacent members connected to it. This method of analysis is known is *Substitute Frame Method*. Since the scope of this book is restricted to buildings of up to 4 storey height for which the effect of horizontal loads is not worth considering, the substitute frame method, suitable for vertical loads only will be discussed here.

3.2.2 Substitute Frames : Analysis for Vertical Loads *(clause 22.4.2)*

A building frame, in fact, is a three dimensional frame *i.e* a space frame shown in *Fig. 3.2.2.*

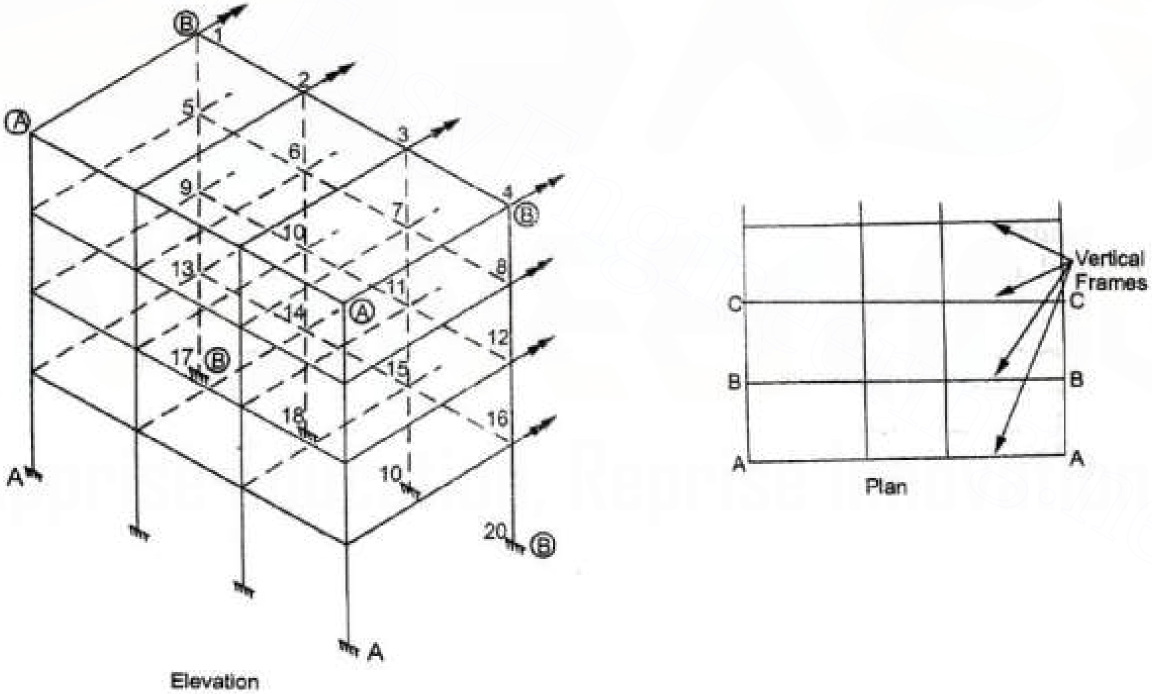

Fig. 3.2.2 Building Frame

The analysis of a space frame is complex, laborious, and also time consuming. Besides it is also not necessary (*or* not even justified) for the degree of accuracy required in R.C. construction. Therefore, as a first degree (*or* level) of approximation, the three dimensional space frame is divided into a number of two dimensional plane frames.

Each plane frame *(see Fig. 3.2.3)* is analyzed for the loads (vertical or horizontal) in the plane of the frame and is assumed to behave independently *i.e* disregarding its interconnection with the adjacent frames. This assumption holds good when all the parallel frames, say *A-A, B-B* etc. in *Fig. 3.2.2*, are subjected to identical loads, so that there is no relative deformation between the adjacent frames. This assumed condition is normally achieved, but if at all there is any difference in the loading conditions and the structural properties (the stiffnesses) of adjacent frames, the relative deformations which are caused due to them are ignored in the analysis by this first degree of approximation. As an illustration, the torsional and/or lateral bending stiffness of members (cross beams) at right angles are ignored. However, these members are assumed to give lateral support to the plane frame (*i.e* the cross members are assumed to be very rigid) with the result, the vertical frame which is plane before loading remains plane after loading.

Thus, the basic frame considered for analysis of a R.C. building is a vertical plane frame. *Fig. 3.2.3* shows the front elevation of vertical frame marked *B-B* in *Fig. 3.2.2*. This plane frame is further subdivided into substitute frames in different manners discussed below making further approximations. The method assumes that forces (*i.e B.M.* and *S.F.*) in the beam of any floor are influenced by the loading on that floor ignoring the effect of loading on the lower and upper floor. As stated earlier, the description of substitute frames given is restricted to those used in analysis for vertical loads only.

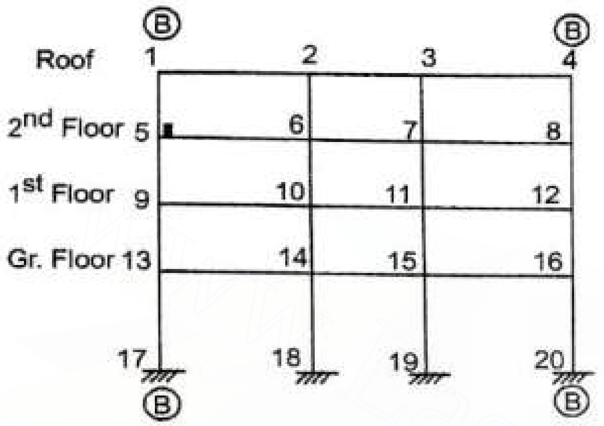

Fig. 3.2.3 Elevation of Building Frame

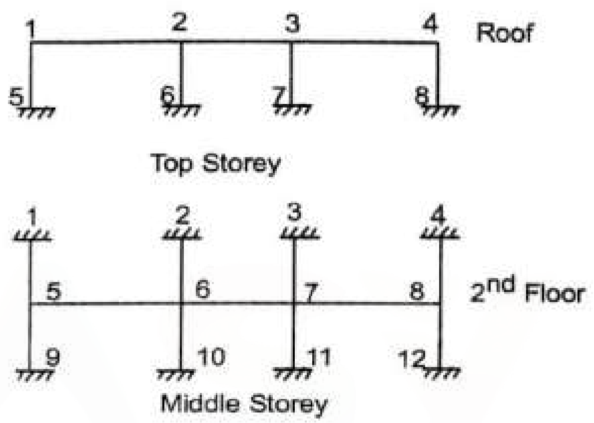

Fig. 3.2.4 Substitute Frame - I Floor Frame

(a) Substitute Frame - I : *Floor Frames*

In the second degree of approximation, the entire vertical frame is subdivided into number of two storey frames for each floor. *The floor frame or substitute frame-I at any floor consists of beams at the floor under consideration together with all connected columns in adjacent upper and lower storeys, assumed to be fixed at their far ends.* The substitute frames for the top floor and intermediate floors are shown in *Fig. 3.2.4*. This frame can be analyzed by any method for different loading cases to produce the maximum forces for design of members. The analysis is carried out for each floor frame, and moments and shears in all beams and columns are determined.

(b) Substitute Frames - II : *Bay Frames*

In the third degree (level) of approximation, instead of taking all beam segments and all columns in the adjacent two storeys, this frame is further subdivided into separate *bay frames each one consisting of the beam of interest together with connected columns and beams in the adjacent spans only, fixed at their far ends as shown in Fig. 3.2.5*. Such a frame is called Substitute Bay Frame. Since beams beyond the adjacent spans are not considered but assumed to be fixed, their stiffness get over estimated. Therefore, their stiffness are reduced to half to allow for the flexibility resulting from continuity [3.4, 3.5, 3.6].

This third degree approximation holds good, theoretically, for symmetric frames for symmetric loadings. The results are likely to differ from exact values in case of unsymmetrical frames and /or unsymmetrical loading. However, the difference is hardly beyond 10%. Such frame is also analyzed for different loading cases to get maximum forces in columns and beams as usual.

(c) Substitute Frame - III : *Beam and Column Systems*

A very conservative alternative to the preceding substitute frame arrangements consists of only continuous beam at each floor level with ends simply supported providing no restraint to rotation as shown in *Fig. 3.2.6. Since the rigidity offered by column is totally ignored, the moments in beams work out to be very large, even at times to the extent of 30% to 50% compared to the actual moments.* The critical loading arrangement should be in accordance with *Sect. 2.6. Thus,* though the analysis is simplified, the design proves to be very uneconomical. I.S.Code does not permit to use this method.[3.7]. However BS : Code[3.8] still permits this method of approach.

Even though, the interconnection between beams and columns has been ignored in calculation, actual rigidity does induce moments in the columns which cannot be ignored in design. Therefore, the moments in columns are obtained by considering only column systems made up of upper and lower column at a joint together with connecting beams fixed at their far ends as shown in *Fig. 3.2.6b*. The far ends of beams and columns are assumed to be fixed and stiffnesses of beams are reduced to half to compensate for the effects of bays beyond. The column sub - frames are analyzed for such loading on beams so as to cause maximum column moments.

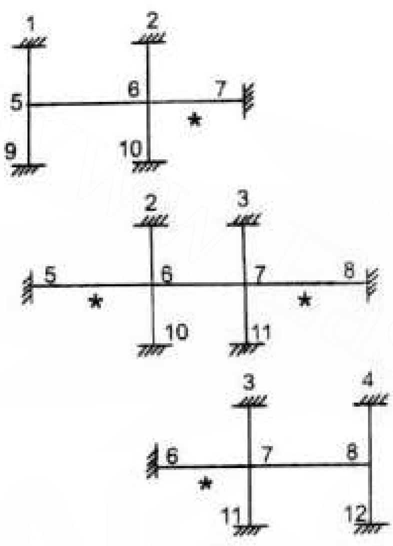

Fig. 3.2.5 Substitute Frame - II Bay Frame

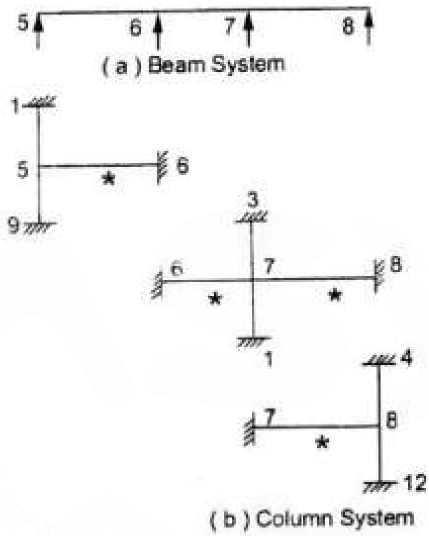

Fig. 3.2.6 Substitute Frame - III Beam Column System

If the results of this totally approximate method are required to be brought nearer to those of substitute frame - I the effect of column moments on beam moments and shear should further be considered and beam shear and moments be modified.

All these methods have been fully illustrated in Chapter - 8, Project-II , and the solutions are compared to bring out the merits and demerits of these methods in regards to the accuracy in relation to the simplicity of calculations.

In the frame analysis discussed above, whatever approximations are adopted, the analysis should be based on the fundamental fact that the joints between the members are rigid. It is, therefore, necessary to know how rigid connection is obtained in a R.C. construction and what are the types of connections. This has been discussed in subsequent *Sect. 3.2.3*.

Besides, the analysis is based on an important structural property, namely, the stiffness *(k)* which depends upon the ratio *I/L* and the nature of support conditions of the member at the far ends. In all substitute frames discussed above, the far end of the member is assumed to be fixed. If far end is hinged *i.e* rotation free, the stiffness of the member is taken equal to 0.75 *I/L*.

From above, it is evident that for computing the stiffness, it is necessary to decide whether the support is rotation free or not. The different types of supports in R.C. construction have been discussed in *Sect. 3.2.4*, and the different alternative methods of computation of stiffness (*I/L*) are presented in *Sect. 3.2.5*.

3.2.3 Types of Connections between Two Members

When two members (*viz.* slab-beam *or* beam-column) are to be connected, no relative translationally movement can be allowed between them. Therefore, connection between the two members are only of two types.

(a) ***A Simple or Hinged Connection :*** It allows relative rotation between the connected members. It does not transfer moment from one component to other but it does transfer the transverse shear and the axial load.

(b) ***Rigid Connection :*** It does not allow the relative rotation between the two connected members. It transfers the moment besides shear and axial force from one member to other.
For transfer of moment and hence for joint to be rigid, the following conditions are required to be satisfied.
(i) There should be interconnecting tension steel between the two members on the tension face with area sufficient enough to effect the transfer to forces (*i.e B.M.* and *S.F.*)
(ii) The interconnecting steel should be adequately anchored in both the connected members either by requisite development length or by mechanical anchorage.

If any one of the above conditions is not satisfied, the joint will not act as a rigid joint. For the joint to be rotation free, it should be seen that above conditions are not satisfied. If they are satisfied partially, the joint will act as a partially rigid (*i.e* semi - rigid) joint.

For illustration, consider a beam column connection shown in *Fig. 3.2.7(a)*. For the connection to be rigid adequate area of interconnecting steel A'_{st}, to resist moment M' at support must be provided on the tension face (*i.e* at top, for vertical downward loading) for a length equal to development length and at the same time it must be extended further in the column through a distance *BA* equal to anchorage length (which is equal to development length L_d). If it is found that this length is large, then instead of extending beam reinforcement into the column, the column bars should be bent and extended in the beam through a distance *BC* equal to the development length.

On the contrary, for a connection to be simple between the beam and column, no steel other than anchor bars be provided at top in the beam, and furthermore, these bars may simply be continued straight in the column through column width only, just for getting sufficient lateral support at top. When the beam deflects it has a tendency to rotate at support. This connection will allow the rotation by development of a vertical crack at the column face or at the junction of upper part of column cast with the beam and lower part of the column top cast earlier.

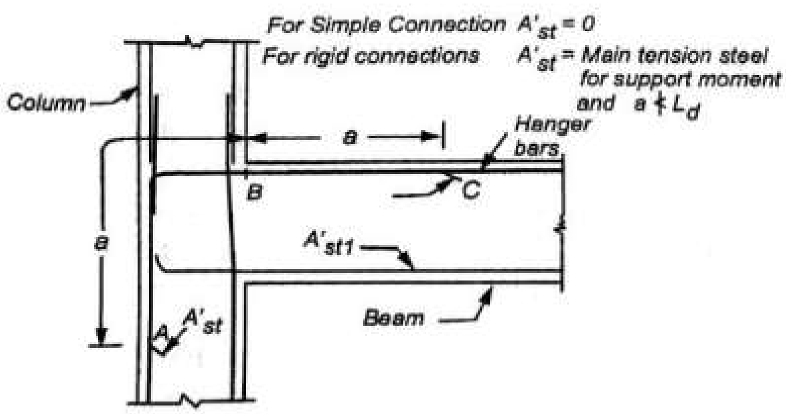

Fig. 3.2.7 (a)

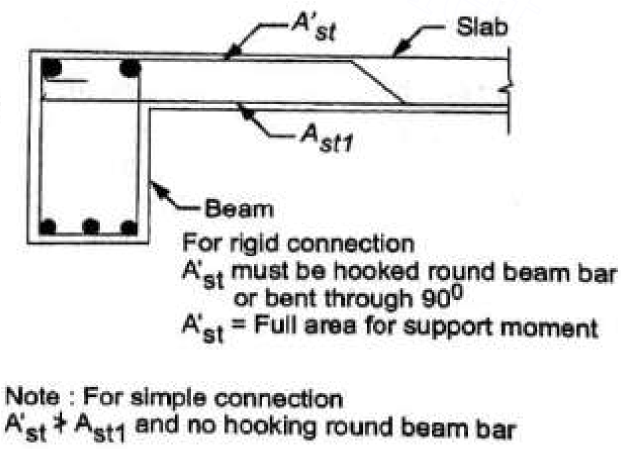

Fig. 3.2.7 (b)

It may be borne in mind that only monolithic casting of the two components does not ensure structural continuity. The structural continuity is obtained only by rigid connection. As an illustration, consider a monolithically cast slab-beam connection shown in *Fig. 3.2.7(b)*. The connection will not be a rigid one unless sufficient tension steel is available at the top of slab and which is adequately anchored by extending it by a bond length distance or mechanically hooked round the beam bar through 180°. If no separate tension steel is provided at top of the slab and if it is simply left over the beam, the connection will be a simple one. The slab only will rotate (and not the beam) by development of crack at the top of slab just at the beam face. If the connection is rigid and the beam itself is simply supported, it will also rotate (due to torsion) along with the slab.

3.2.4 Types of Supports or End Conditions

Following are three different types of ideal supports.

(a) Simple Support

It is the support which can allow the member to move in the direction of the plane of support. It also allows rotation. However, it does not allow movement in the direction perpendicular to supporting plane.

Thus, a simple support neither offers sliding resistance nor any resisting moment. It offers reaction only in a direction perpendicular to the support. A roller support is the only support of this type. It is only used in bridge bearings. In buildings, a member like slab or beam (cast separately from the supporting wall or column and) simply resting on wall or column can be called a simple support (neglecting the frictional resistance to sliding at the interface). A precast beam, or a slab resting on wall, or cast in-situ slab resting on steel beam are some more illustrations of this type of support.

(b) Hinged Support

It is a support which allows the supported member only to rotate but does not allow any translationally movement. This support offers reaction in any direction but does not resist moment. The support is known as rotation free support.

A slab having simple connection with the supporting beam as explained in *part (a) of Sect. 3.2.3*, is said to have rotation free support even though actual hinge is not provided at the support. Even a slab cast monolithically and rigidly connected with the supporting beam as described in *part (b) of Sect. 3.2.3* can be rotation free at support if the beam itself is free to rotate along with the slab. The rotation of beam is possible if that beam itself is simply supported at its ends. A hinged support or a rotation free support is also many times loosely termed as a simple support (though a simple support is truly a roller support)

(c) Fixed Support

It is the support which not only resists translation but also rotation. It resists moment and offers reaction in any direction. Thus, fixed support is a support which does not rotate.

A slab or a beam embedded in a rigid wall can be said to give a rigid support. Any support which can offer resisting moment so as to prevent rotation can be called a fixed support. It has already been made clear in part (b) above that a rigid connection should not be taken as to give fixed end condition. Rigid connection implies zero relative rotation between the connected members. It does not imply zero rotation of the joint or of the supported member. Thus, it must be noted that a simple connection between two members can be said to give a simple rotation free end condition to the supported member. But a rigid connection between two members does not necessarily give a fixed end condition to the supported member.

Consider a two span continuous beam carrying equal U.D. load on both the spans. The beam is simply supported over three supports *i.e* it is not even interconnected with the support. Still, the symmetry of the loading, span and end conditions, creates a condition of zero rotation at the intermediate support which can be considered as rotation fixed support condition for the purpose of analysis. However, still, the support is simple because rotation is possible due to change in the loads on two spans.

The question of end condition for the column at the end of the footing is a typical one. Consider a column subjected to axial load P and a moment M at the top. It is to be seen whether the footing end could be called hinged *or* fixed. As stated above, for the column base to be fixed, the footing should be able to offer a resisting moment equal to *M/2*, besides axial compression. This resisting moment can be made available either by nonuniform pressure distribution at the base of footing in case of a concentric footing as shown in *Fig. 3.2.8(a)*, or alternatively, resisting moment can also be made available by an eccentric footing having uniform pressure distribution at the base as shown in *Fig. 3.2.8(b)*. If the bearing capacity of the soil is low, it is many times not necessary to provide a fixed base *i.e* moment resistance footing. In such a case, the footing could be designed for axial load only. Since, the footing cannot offer resisting moment, the column has a tendency to rotate at the base. This rotation will be possible if the supporting soil yields more on one side and less at the other edge, thus creating a rotation free condition as shown in *Fig. 3.2.8(c)*. This rotation free condition is possible only with soils having low or medium bearing capacities. For soils with large bearing capacity, rotation of footing is not possible as such footings cannot be designed for axial loads only. They have to be designed as moment resistant or fixed. The moment at the footing can also be avoided or reduced by providing a heavy plinth beam in the plane of bending near the footing. This practice is common in R.C. building construction when the depth of footing below plinth is very large *i.e* when the cost of wall below plinth works to be greater than the cost of plinth beam. The plinth beam also helps in reducing the effective length of column at ground floor.

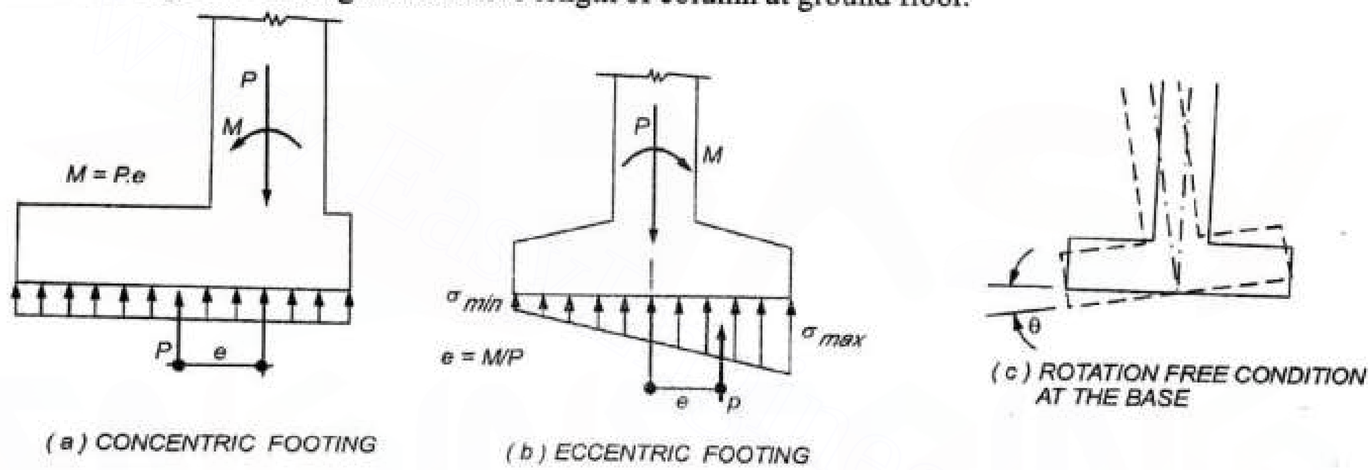

Fig. 3.2.8 End Conditions for column Footing

3.2.5 Stiffness of Members *(clause 22.3)*

For calculating the stiffness of a member, the moment of inertia *(I)* of the members meeting at a joint are required. Code allows to take any one of the following definitions of moment of inertia for determining the stiffness.

(1) Moment of inertia of Gross - section : Moment of inertia of concrete gross cross- section (I_{gr}) ignoring reinforcement is given by :

Rectangular section of size $b \times D$, $I_{gr} = bD^3/12$

(2) Moment of inertia of Transformed Gross Section : Moment of inertia of transformed gross concrete section including the reinforcement transformed on the basis of modular ratio, is obtained as under :

In the case of column of size $b \times D$ with the neutral axis lying outside the section, the whole section is under compression. In such a case all the steel will be in compression and the moment of inertia is given by :

$$I = bD^3 / 12 + \sum_{i=1}^{n} (m-1) A_{si} \times x_i^2$$

where, m = modular ratio ,

n = number of rows of reinforcement,

A_{si} = Area of steel in compression

x_i = distance of the steel at *i*th level from C.G. of section.

(3) *Moment of inertia of Transformed Cracked Section :* It is the moment of inertia of concrete section in compression (I_r) including area of reinforcement transformed on the basis modular ratio.

Whatever may be the basis adopted for calculation of I, it is required to be applied consistently to all members.

In preliminary design, since neither the moments are known nor the reinforcement, the question of using the second or third method of finding I (described above) does not arise at all. The common practice is, therefore, to take I of concrete gross section (I_{gr}) ignoring reinforcement.

The moment of Inertia of gross concrete section excluding reinforcement may be obtained using the following equations :

Rectangular Section : $I_{gr} = bD^3/12$

Flanged Section :

$$\text{Depth of N.A. } \bar{x} = \frac{b_w D^2/2 + (b_f - b_w)\, D_f^2/2}{b_w \times D + (b_f - b_w) \times D_f} \qquad (I)$$

$$I_{gr} = b_f \bar{x}^3/3 - (b_f - b_w)\,(\bar{x} - D_f)^3/3 + b_w\,(D - \bar{x})^3/3 \qquad (II)$$

$$\text{or} \quad I_{gr} = k_f b_w D^3/12$$

$$\text{where, } k_f = 4[k_1 k^3 + (1-k)^3 - (k_1 - 1)\,(k - k_2)^3\,] \qquad (III)$$

$$k_1 = b_f/b_w, \quad k_2 = D_f/D \ \text{ and } \ k = \bar{x}/D.$$

Alternatively, the value of k_f may obtained from *Fig. 3.2.9*

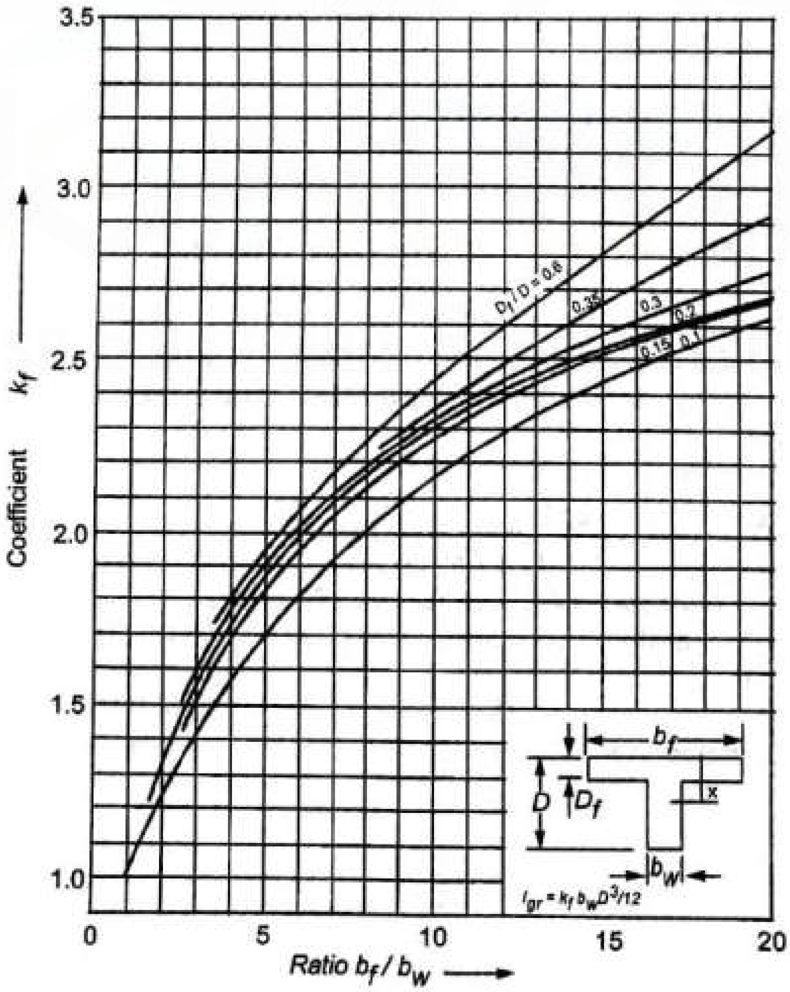

Fig. 3.2.9 Moment of Inertia of T-Beam

However, main difficulty in calculating I arises when the beam is continuous at both ends as in the case of beams in frames, because in that case the beam acts as rectangular beam in the negative moment region and a flanged beam in positive moment region. Thus, such a beam is a varying moment of inertia along the length. A single equivalent or effective value of I to be taken for stiffness would depend upon the ratio of region of positive moment (L_o) to length L of the beam. However, since value of L_o depends upon the end conditions, the loading and the moment developed at supports, it is not constant. It varies from 0.58 L for a beam fixed at both ends to L for a beam simply supported at both ends.

Since 100% fixity at supports is hardly ever possible because of rotational displacements at supports due to flexibility of supports, the ratio L_o/L is normally large, hence some designers take I_{gr} of a flanged section ignoring the difference between L_o and L.[3.10] The other method is to use moment of inertia of *T*-section equal to twice that of rectangular section [3.11]. The multiplying factor 2 corresponds to flanged section having $b_f/b_w = 6$ and $D_f/D = 0.2$ (see *Fig. 3.2.9*).

Since the section acts as a flange section in the major portion of beam span and normally the ratio of b_f/b_w is nearly 6, others consider I_{gr} of beam $= 2 \times b_w D^3/12$ *i.e.* two times that of a rectangular section to be more appropriate.

Institute of struct, E. Manual[3.12] recommends to consider actual flange width of *T*-beam *or* $(0.14L + b_w)$ whichever is less. Wang et.al[3.13] assumes it as an equivalent system approximating *T*-section having flange width equal to twice web width over the entire section. This will over estimate the beam stiffness giving higher moments in beam. The design of beam will be conservative but the computations of I of *T*-section will be too much involved. Alternatively, the assumption of rectangular section (instead of flanged section) is easy for calculation, gives higher moments in the columns[3.14]

The length of the member, in calculation of stiffness ($k = I/L$,) is taken equal to centre to centre distance between the supports for beams, and floor to floor height for columns except in case of ground floor column for which the length is taken from the top of footing to the top of floor level when there is no plinth beam, and from the top of plinth beam when the same exists.

3.2.6 Effect of Stiffness on Distribution of Moments in Beams and Columns

When a beam is connected to columns, fixed end moment M_e is first calculated on the assumption of zero rotation or clamping of the joint. This moment is known as unbalanced moment acting at the joint. The joint is then released or unclamped by applying an opposite moment. This moment is now distributed to various members meeting at the joint in proportion of their stiffnesses.

Thus, moments in columns are obtained as follows.

$$M_{cal.a} = \frac{k_{ca}}{k_{ca} + k_{cb} + k_b} \times M_e, \quad M_{cal.b} = \frac{k_{cb}}{k_{ca} + k_{cb} + k_b} \times M_e$$

$$M_{beam} = \frac{k_b}{k_{ca} + k_{cb} + k_b} \times M_e$$

where, k_{ca} = stiffness of Column Above the joint = I_{ca}/L_{ca},
k_{cb} = stiffness of Column Below the joint = I_{cb}/L_{cb},
k_b = stiffness of Beam = I_b/L_b.

If the beam is continuous beyond, the stiffness of beam k_b *is reduced to half*[3.4, 3.5] to account for the effect of loads on spans beyond.

It will be observed from the above relations, that if the column has large cross-section and is short compared to beam, its stiffness $k_c = I_c/L_c$ will be large. While, if a beam is of smaller cross-section and has a large span, its stiffness will be small. Consequently, negligibly small rotation will occur at the joint and the column is said to offer practically full fixity to beam. [See *Fig. 3.2.10(a)*]. The bending moments in the column is said to offer practicqlly full fixity to beam [see Fig. 3.2.10(a)]. The bending moments in the beam and column[3.15] will both be nearly $wL^2/12$.The fixity offered is more if columns are two in number while the beam is only one. This situation is common in lower storeys of multistoreyed frames having large span bays.

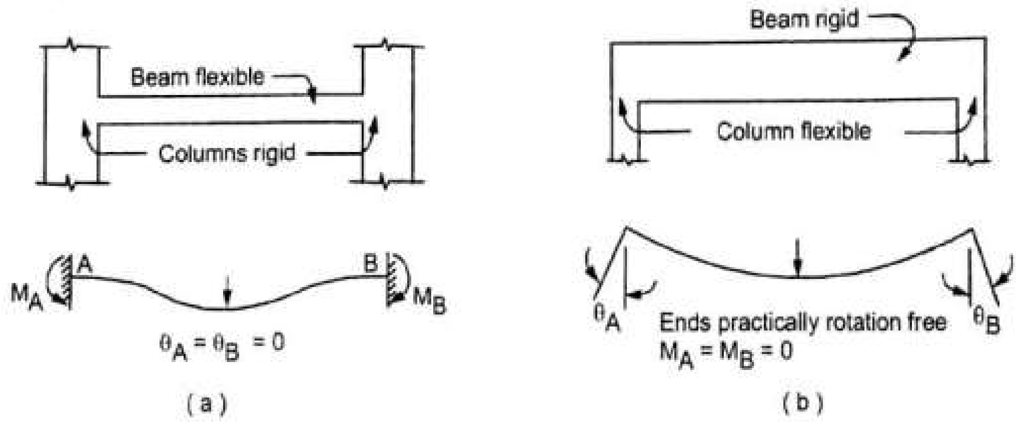

Fig. 3.2.10 Effect of Stiffness on Moments in Beams and Columns

On the other hand, if the cross-section of beam is large and span short, its stiffness will be large. Simultaneously, if the column cross-section is small and length large its stiffness will be small and consequently joint will rotate and practically no fixed end moment will develop either in the column or in the beam. (practically entire value of fixed end moment M_e at the joint gets released with no moment remaining in the beam). This situation gives rotation free simple support for the beam (See *Fig. 3.2.10b*). Thus, the bending moment in the beam and column at the joint lies between 0 and $wL^2/12$ in general; the actual magnitude depends upon the relative stiffnesses of the beam and the column. To satisfy the condition of equilibrium, the sum of the moments in the beams meeting at a joint must be equal and opposite to the sum of the moments in the columns meeting at that joint.

3.3 DESIGN ASSUMPTIONS AND APPROXIMATIONS

In practical design, a designer is many times required to make certain assumptions and approximations for making the analysis simple to save computational efforts and time. The design assumptions, of course, should be such as to make the design err on the safer side. If at all they are found to be on the unsafe side at certain places, an allowance should be made in the analysis based on earlier observations and judgement. Some of the assumptions and approximations are given below.

3.3.1 Assumptions regarding Support Condition

The first and the foremost assumption that is required to be made is about the support condition or the type of support for slabs and beams. Normally, though slab is cast monolithically with the beam, it is not necessary that it should be connected rigidly to supporting beams. Such a rigid connection does not necessarily ensure fixed end condition. It may cause rotation of the beam if the beam itself is simply supported at its ends. If the beam is fixed at the ends, the rigid connection between the slab and the beam induces torsion in the beam giving condition of partial fixity and not full fixity. Therefore, *it is commonly assumed that a slab is simply supported at discontinuous end and continuous over intermediate support.* The same assumption can be made applicable to beams also because whether a beam is connected rigidly or simply to a supporting column, it is generally rotation free at the ends and therefore *assumed as simply supported at the ends when one is not sure about the condition of rotational restraint at the ends.*

The above assumption is for ends of continuous beams or slabs. Similarly, assumption is many times, required to be made for an intermediate support also. For example, the exact analysis of a continuous beam or slab having large number of spans, (say more than 4) is extremely laborious, and besides, continuity of beam/slab for more than 4 spans has little advantage. The analysis is made simple by introduction of a discontinuity at suitable intermediate support (like the discontinuity at support in a multispan bridge). As an illustration, consider a beam in a public building shown in *Fig. 3.3.1*.

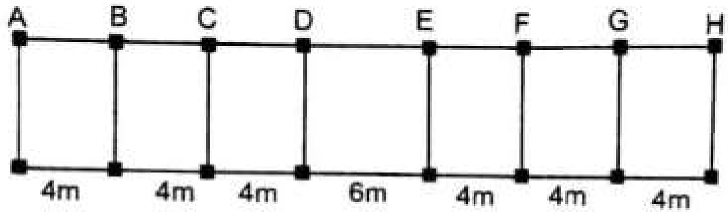

Fig. 3.3.1 Introduction of Discontinuity in a long unequal span continuous beam

A structural discontinuity can be introduced at supports D and E and the entire beam can be divided into 3 separate beams, *ABCD*-freely supported *A* and *D*, *DE* - simply supported at *D* and *E*, and *EFGH* freely supported at *E* and *H*. As the structural discontinuity is assumed at *D* and *E*, the same condition can be obtained by not allowing the top bars to extend from *CD* to *DE or* from *FE* to *ED*. This assumption of providing a simple support for beams has a backing of age-old practice of famous post-lintel construction adopted in the past over number of centuries. However, it should not be extended to each span of a continuous beam.

Alternatively, the beam in *Fig.3.3.1* can be divided into three segments *ABCD,DE* and *EFGH* for the purpose of analysis only and each segment analyzed independently assuming *fully fixity* at *D* and *E*. This assumption of treating structural continuity as fixity, though may not be rigorously correct, especially for beams with unequal spans and loads, can still be accepted since the difference on account of this approximation is found to be well within the degree of accuracy expected in reinforced concrete structures. It simplifies the analysis to a great extent.

It may be noted that if a continuous beam cast monolithically is designed by assuming it as made up of a number of simply supported beams, the moments in the beam work out to be very large, even at times to the extent of 20% to about 50%. In such a case, the negative moment that may structurally develop at intermediate support due to physical continuity of beam and/or rigidity of intermediate walls/ columns/supports would cause cracking of concrete at top at the face of support which could be quite objectionable, though it may not be a sign of structural unsoundness or lack of safety. As cracking occurs, the moment at support is transferred to the mid-span. The load is fully sustained if the mid-span section is designed corresponding to moment for a simply support condition. However, this is a crude design practice which makes the beam heavy and misuses the principle of compensatory resistance. Since *continuity is totally ignored, the moments in beams work out to be very large, even at times to the extent of 30% to 50% as compared to the actual moments.* Especially, such design though may not be unsafe for vertical loads, is definitely *unsound for resisting horizontal loads* and unacceptable from viewpoint of serviceability or performance of the structure, and it will not give any reserve strength at collapse, and consequences of collapse, whenever it may occur, are likely to be very serious.

3.3.2 Approximations regarding Bending Moments in Beams and Slabs

The analysis of a multispan continuous beam can be further simplified by analysing and designing each beam (span) separately considering approximate moments at continuous end based on redistribution of moments as explained in *Sect. 3.3.1*. In this approach, simplicity is achieved together with the desired economy, safety and a satisfactory structural performance.

Calculation of exact bending moments in single span slabs or beams do not pose any problem. They can be obtained directly using the coefficients for standard loading cases available in various design aids. Difficulty normally arises in determination of bending moments in continuous beams/slabs. Codes prescribe coefficients for continuous beams/slabs with approximately equal spans (variation between long and short span not exceeding 15% of long span) and carrying uniformly distributed loads. These are given in *Sect.5.1*.The coefficients for equal span continuous slabs/beams for other standard loadings like central point load or equal point loads at 1/4th or 1/3rd span points are also available in various design aids[3,9].

As mentioned earlier the continuous beams/slabs can be approximately designed by treating a continuous beam/slab as if made up of number of independent single span beams or a group of typical multispan beams. This approximation is an extension of the principle used in Substitute Frame Method of dividing a large structure into parts for the purpose of analysis, and then analysing each part independently. Here, it is applied to continuous beams of approximately equal spans. Each one is analyzed separately using the standard bending moment coefficients which are based on the ordinates of the bending moments envelope [*See Fig.3.3.2*] and allowing redistribution of moments. They give values within 30% of the exact theoretical values. It may be noted, that redistribution of moments is allowed to the extent of 30%, hence the difference of 30% is acceptable between the design moment and the elastic moment at support with corresponding increase in the mid-span moment. The results obtained by approximation under discussion lie nearly between the elastic moments and those obtained by limit analysis allowing redistribution of moments.

The design moment coefficients used for typical beams are as follows :

Moment at supports as well as at mid-span :

Uniformly Distributed Load (w) :

End spans $\pm\ wL^2/10.$ ... ...(3.3.1)

Intermediate spans $\pm\ wL^2/12.$ ... ... (3.3.2)

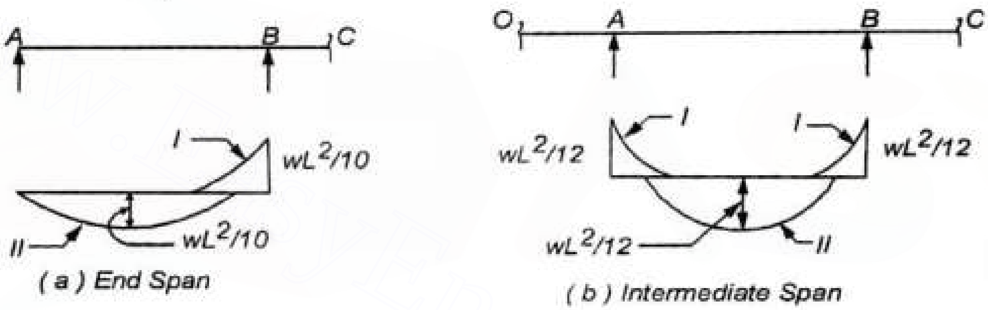

Fig. 3.3.2 Approximate Moment Coefficients for an Isolated Span

3.3.3 Assumptions regarding Beam Section

Another important assumption made in design of R.C. beams is about the type of the section whether to design a flanged section or a rectangular section. In general practice the beam and the slab are cast together with slab main steel going inside the beam section, and beam bars and stirrups extend into the slab. In such a case part of the slab acts along with the beam in resisting compressive forces, provided slab lies in the compression zone with respect to bending of the beam and the resulting section is called as a *flanged section*. .

When the slab, occurs on both sides of the beam as in case of an intermediate beam, resulting cross-section resembles a *T*-shape and hence called *'Tee-beam'*.On the other hand if the slab is only on one side of the beam it appears like an inverted *'L'* and hence the name *'Ell-beam' (Fig.3.3.3c) In reality, we can see only the slab and the beam below it. So, flange section is a virtual (not real) concept which actually does not appear as flange but stucturally acts as a flange providing compression.*

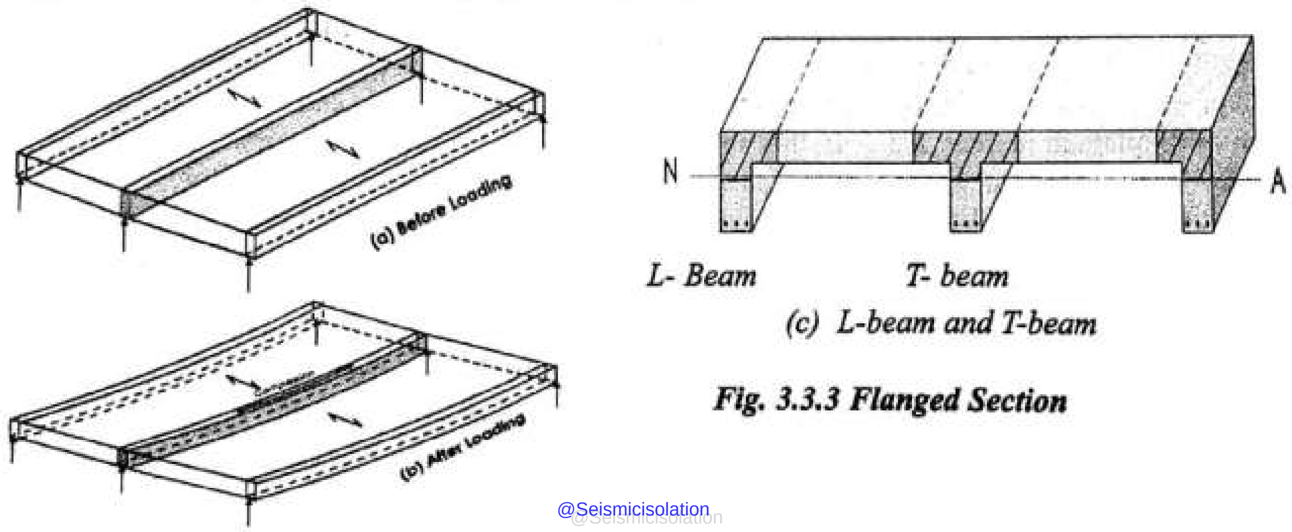

Fig. 3.3.3 Flanged Section

Bending of Beam

The initial position of the beam is shown in (Fig.3.3.3.a). When the load is applied the beam bends in the vertical plane producing compression at the top (Fig.3.3.3.b). Thus, the slab lies in the compression zone and forms the virtual (not real) flange of the beam.

Since, design of a flanged beam is complicated and laborious as compared to the design of rectangular beam, the design could be done assuming beam to be of rectangular section only when the design moment does not exceed $M_{ur.max}$ of rectangular balanced section. The beam designed on the basis of this assumption is always on the safer side when design moment is less than M_{ul} which is the value of M_{ur} of flanged section for $x_u = D_f$ (*i.e* when $x_u =< D_f$), and this condition is normally satisfied in all cases of beam design. The area of steel obtained on the basis of assumption of rectangular section is always greater than that required for flanged section, because the lever arm of the rectangular section [Fig.3.3.4(ii)] is always less than that of a flanged section [Fig.3.3.4(i)], since x_u in a rectangular section is always greater than x_u for flanged section *See Fig. 3.3.4.*

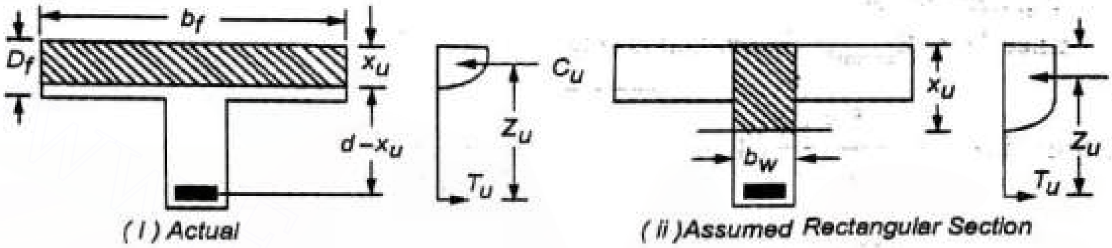

Fig. 3.3.4 Assumption Regarding Beam Section

The area of steel increases with the increase in the value of x_u and is maximum when $x_u = D_f$. In case of flanged beam, the quantity of steel required to balance the compression in outstanding flanged portions depends upon the ratio D_f/d and b_f/b_w and is a variable quantity. However to get rough idea, it may be mentioned that the maximum additional steel required due to assumption of rectangular section in place of a flanged section is about 10% to 20% for $x_u =< D_f$

However, when the design moment > moment of resistance of a balanced section $M_{ur.max}$, the assumption of rectangular section *should not be* made *i.e* the area of steel should not be worked out on the assumption of rectangular section, because in that case the rectangular section should be designed as a doubly reinforced. Area of steel required for doubly reinforced rectangular section will be much more than that required for flanged section. In such a case, the *beam shall be designed as flanged beam only provided flange lies in the compression zone with respect to bending of the beam.* The following example will clearly bring out the difference between the area of steel required when design is based on assumption of rectangular section in place of a flanged section.

Illustrative Example :

A two span continuous beam 4m span is fixed at both ends. It carries slab of 110mm thick. The section of the beam is 230mm x 380mm. Effective cover = 40mm. Compare requirments of steel (i)Intensity of UDL = 48 kN/m² (ii) Design moment $M_u = M_{ur.max}$ (iii) M_u=96 kN/m. Use M20 - Fe 415

Data Span = 4*m* , Slab 110 *mm* thick, Section 230*mm* × 380*mm*, Beam fixed at both ends

$f_{ck} = 20\ N/mm^2$, $f_y = 415 N/mm^2$, Loading (i) w_u = 48 kN/m² (ii) $M_u = M_{ur.max}$ (iii) M_u = 96 kN.m

Required : A_{st}.

Solution : $d' = 40$ mm,

$\therefore\ d = 380 - 40 = 340\ mm$, $x_{u.max} = 0.48 \times 340 = 163.2$ mm

$M_{ur.max} = R_{u.max} \times bd^2$. $R_{u.max} = 2.76\ N/mm^2$ *(Table 4.1.1)*

$\therefore\ M_{ur.max} = 2.76 \times 230 \times 340^2 \times 10^{-6} = \mathbf{73.3\ kN.m},$

(1) $w_u = 48\ kN/m^2$,

$M_u = w_u L^2/12 = 48 \times 4^2/12 = \mathbf{64\ kN.m}$

For T-section : For M_u=64 kN.m.

$$L_o = 0.7 \times L = 0.7 \times 4000 = 2800mm$$

$$b_f = L_o/6 + 6D_f + b_w = 2800/6 + 6 \times 110 + 230 = 1357 \ mm \qquad \text{(Eq. 4.3.1)}$$

$$(M_{ur1})_{x_u = D_f} = 0.36 f_{ck} b_f D_f (d - 0.42 D_f) \qquad \text{(Eq. 4.3.8)}$$

$$= 0.36 \times 20 \times 1357 \times 110 \times (340 - 0.42 \times 110) \times 10^{-6}$$

$$= 315.8 \ kN.m > 73.38 \ kN.m \ \therefore x_u < D_f$$

$$M_u = 64 \text{ kN.m} < M_{ur.max} \ (=73.3 \ kN.m)$$

For Flanged Section :

$$\text{Required } A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 64 \times 10^6}{20 \times 1357 \times 340^2}}\right] \times 1357 \times 340$$

$$= 534.5 \ mm^2$$

For rectangular section :

Since $M_u < M_{ur.max}$, the section is singly reinforced

$$\text{Required } A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 64 \times 10^6}{20 \times 230 \times 340^2}}\right] \times 230 \times 340$$

$$= 625.4 \ mm^2 \qquad \text{(Eq4.1.1b)}$$

$$\% \text{ increase over flanged section} = \frac{625.4 - 534.5}{534.5} \times 100 = 17\%$$

(II) $M_u = M_{ur.max} = 73.3 \ kN.m$

For flanged section :

:

$$\text{Required } A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 73.3 \times 10^6}{20 \times 1357 \times 340^2}}\right] \times 1357 \times 340$$

$$= 614.4 \ mm^2$$

For rectangular section :for balanced section x_u = 163.2 mm

$$\text{Required } A_{st} = \frac{73.3 \times 10^6}{0.87 \times 415 \times (340 - 0.42 \times 163.2}$$

$$= 748 \ mm^2 \qquad \text{(. 4.1.1b)}$$

$$\% \text{ increase over rectangular section} = \frac{748 - 614.4}{614.4} \times 100 = 21\%$$

(iii) $M_u = 96 \text{ kN.m} < M_{ur1}$

For flanged section:

$$\text{Required } A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 96 \times 10^6}{20 \times 1357 \times 340^2}}\right] \times 1357 \times 340$$

$$= 812 \ mm^2$$

For rectangular section

$x_{u.max} = 0.48 \times 340 = 163.2\ mm$,

$M_{ur.max} = 73.3\ kN.m$ as obtained earlier

$M_u = 96$ kN.m $> M_{ur.max}$

$\therefore$ *Section is doubly reinforced*

$$A_{st1} = \frac{73.3 \times 10^6}{0.87 \times 415 \times (340 - 0.42 \times 163.2)} \quad \text{(Eq.4.2.3a)}$$

$= 748\ mm^2$

$M_{u2} = M_u - M_{ur.max}$

$= 96 - 73.3$

$= 22.7$ kN.m

$$A_{st2} = \frac{(96 - 73.3) \times 10^6}{0.87 \times 415 \times (340 - 40)}$$

$= 209.6\ mm^2$

Total area of tension steel= $748 + 209.6 = 957.6\ mm^2$

$d_c/d = 40/340 = 0.12$,

$\therefore f_{sc} = 348\ mm^2$ (Table4.2.2)

$$A_{sc} = \frac{0.87 \times 415 \times 209.6}{(348 - 0.446 \times 20)}$$

$= 223\ mm^2$

Total area of steel $= 957.6 + 223$

$= 1180.6\ mm^2$ as against $812\ mm^2$

$$\% \text{ increase over flanged section} = \frac{1180.6 - 812}{812} \times 100 = 45\%$$

Comments *: It will be observed that percentage increase of steel over flanged section normally varies from about 12% to 20%. (In this case it is 17% to 21%). It varies with the position of neutral axis. If the neutral axis lies vary near to the top flange the percentage is less.*

But when $M_u > M_{ur.max}$ the doubly reinforced section becomes very costly because % increase becomes very high. It is 45% in this case.

Therefore, when the section becomes doubly reinforced it should be designed as flanged section only.

3.4 References

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CHAPTER - 4

LIMIT STATE THEORY FOR R.C. MEMBERS

A member in a R.C.framed structure is subjected to following structural actions : (a) Flexure, (b) Shear, (c) Torsion, (d) Axial Compression (Crushing and Buckling), and (e) Combination of above.

While designing a member, its strength at collapse and behaviour at working loads for each of the above structural actions are required to be known thoroughly. As stated earlier in the scope of this book, the object of this chapter is not to explain the Theory of Limit State Design in detail. On the contrary, it is presumed that the reader knows the theory well. However, a cursory review of the theory has been taken in this chapter and the relevant equations and the design requirements given according to IS : 456 - 2000. For detailed study of the Limit State Theory, the reader may refer the Authors'book on "Limit State Theory and Design of Reinforced Concrete"[4.1].

4.1 FLEXURE *(clause 38.1)*

4.1.1. Basic Assumptions

(1) A normal section which is plane before bending remains plane after bending.

This implies that longitudinal strain at any point in a section is proportional to the distance x of that point from the neutral axis. Mathematically, $\varepsilon \; \alpha \; x$. Graphically, the strain diagram across the section is triangular up to failure.

(2) Limit state of collapse in bending is said to have reached in flexure when the maximum compressive strain in concrete ε_{max} at the outer most fibre reaches the ultimate value $\varepsilon_{cu} = 0.0035$.

(3) The variation of compressive stress with strain in concrete in compression region, known as stress -block, is rectangular - parabolic as prescribed by I.S. Code and is shown in *Fig. 4.1.1a*. It consists of a parabola emerging from the neutral axis with its apex lying at a point corresponding to strain of 0.002 and rectangular terminating at the compression face where maximum strain is 0.0035.

(4) Concrete does not carry any tension. The tension is carried by steel only.

(5) Perfect bond exists between concrete and steel right up to collapse. Mathematically, $\varepsilon_s = \varepsilon_c$.

(6) The stress in the reinforcement is corresponding to the strain in steel at that point as obtained from the prescribed stress-strain curve for the type of steel used for reinforcement.

(7) The maximum strain in tension steel at ultimate state (*i.e.* at collapse) shall not be less than :

$$\varepsilon_s = 0.002 + f_y / (1.15\, E_s) \cong 0.002 + 0.87\, f_y$$

This condition ensures that the flexural failure is initiated by yielding of steel in tension.

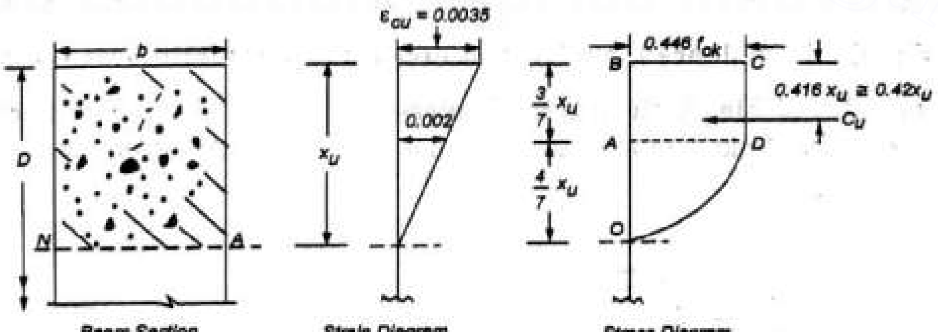


Fig. 4.1.1a Stress Block across a Beam Section

Let x_u = Depth of neutral axis below the compression face ,

Area of stress block = $0.361 f_{ck} x_u \cong 0.36 f_{ck} x_u$

Distance of the centroid of stress block from the compression face $= \bar{x} = 0.416 \times x_u \cong 0.42\, x_u$

4.1.2. Modes of Failure

Depending on the relative proportion of steel in the section three failure modes occur *viz.* balanced failure, under-reinforced failure and over-reinforced failure and the corresponding sections are called balanced, under-reinforced and over-reinforced sections Fig.4.1.1b.

(a) Balanced section

When the ratio of steel to concrete in a section is such that the strain in tension steel and strain in concrete reach their maximum values simultaneously the section is called a *balanced* or *critical section* and the percentage of steel in the section is known as *critical or balanced steel percentage.*

(b) Under-reinforced section

When the percentage of steel in the section is such that at ultimate limit state, the steel reaches the failure strain before concrete reaching ultimate compressive strain. The section is known as *under-reinforced* section. The steel yields before concrete crushes and the amount of steel is less than that in balanced section.Since the amount of steel is less than the balanced section, the neutral axis moves above the balanced neutral axis to satisfy equilibrium condition.

The failure of an under-reinforced section is characterized by substantial deflection and extensive cracking of concrete giving *ample warning* of impending failure. For this reason and economy point of view the under-reinforced sections are preferred.

(c) Over-reinforced section

When the percentage of steel in the section is such that at ultimate limit state, the concrete reaches the maximum strain before ultimate strain in steel is reached, the section is known as *over-reinforced* section. The concrete crushes before steel reaching its maximum strain and the amount of steel is more than that in balanced section.

Since the amount of steel is more than the balanced section, the neutral axis moves below the balanced neutral axis resulting in concrete reaching its ultimate strain before steel reaching its maximum strain . As the compressive strain in concrete reaches its ultimate value first *sudden failure* occurs *without giving any warning* due to crushing of concrete. Consequently, the depth of neutral axis is restricted to that of a balanced section and hence the *depth of critical neutral axis is called maximum depth of neutral axis,* $x_{u,max}$.

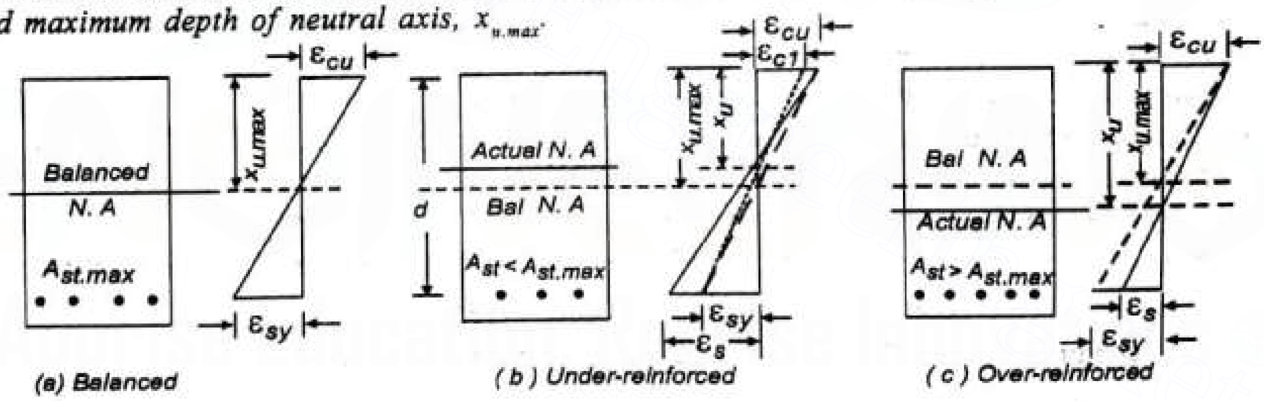
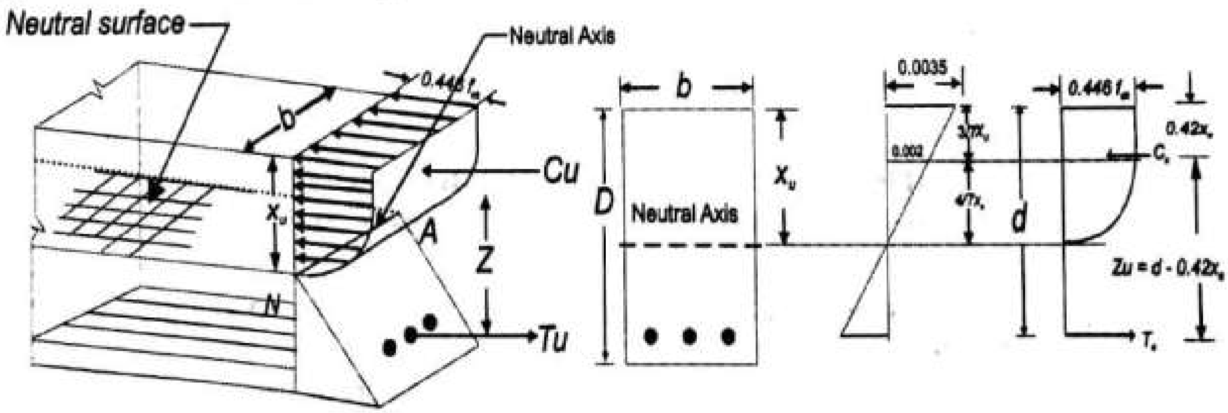


Fig. 4.1.1b Balanced, Under-reinforced and Over-reinforced section

4.1.3 Properties of Singly Reinforced Under-reinforced Rectangular Section

Fig. 4.1.2 shows the strain diagram and stress diagram of an under-reinforced rectangular section.



Note: I.S.Code has prescribed compression strain equal to 0.0035 even though it varies with the grade of concrete between 0.003 to 0.006

Fig. 4.1.2 Properties of Under-reinforced Section

a) *Depth of Neutral Axis*

Depth of neutral axis is obtained by considering equilibrium of internal forces.

Total compression, C_u = Total tension, T_u

$$\therefore 0.36\ f_{ck}\, bx_u = 0.87\ f_y A_{st}$$

$$\therefore x_u = \frac{0.87\ f_y}{0.36\ f_{ck}} \times \frac{A_{st}}{b} \quad \text{......(4.1.1.a)}$$

$$\therefore k_u = \frac{x_u}{d} = \frac{0.87\ f_y}{0.36\ f_{ck}} \times p_t \quad \text{... ... (4.1.2.a)}$$

or
$$A_{st} = \frac{0.36\ f_{ck} b\ x_u}{0.87\ f_y} \quad \text{or} \quad p_t = \frac{0.36\ f_{ck}\ k_u}{0.87\ f_y} \quad \text{... ...(4.1.6.b)}$$

where, $p_t = \dfrac{A_{st}}{bd}$ = steel factor

(b) *Ultimate Moment of Resistance*

The moment of resistance M_{ur}, is obtained by taking moment of total compression, C_u, about resultant tension T_u, and vice versa

$$M_{ur} = C_u \times Z_u \qquad = T_u \times Z_u$$

where, Z_u = lever arm $= d - 0.416\, x_u \cong d - 0.42\, x_u$

(i) M_{ur} from compression in Concrete

$$M_{ur} = 0.36 f_{ck}\, b.x_u\ (d - 0.42\, x_u)$$

or, $$M_{ur} = M_u = [\,0.36 f_{ck} k_u (1 - 0.42\, k_u)]\ bd^2 = R_u\ bd^2 \quad \text{... ...(4.1.4a)}$$

where, $$R_u = 0.36 f_{ck}\, k_u (1 - 0.42\, k_u) \quad \text{... ...(4.1.4b)}$$

(ii) M_{ur} from tension in steel

$$M_{ur} = T_u \times Z_u$$

where, T_u = design yield stress x Area of steel = $0.87 f_y A_{st}$

$$M_{ur} = M_u = 0.87\ f_y A_{st} (d - 0.42\, x_u) \quad \text{... ...(4.1.3a)}$$

Substituting value of x_u from *Eq. 4.1.1a* we get,

$$M_{ur} = M_u = 0.87\ f_y A_{st}\, d \left(1 - \frac{f_y}{f_{ck}} \times \frac{A_{st}}{bd}\right) \quad \text{... ...(4.1.3b)}$$

The solution of above equation gives A_{st} as :

$$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6\ M_u}{f_{ck}\ bd^2}}\right] bd \quad \text{.. ...(4.1.6a)}$$

or
$$p_t = \frac{A_{st}}{bd} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6\ R_u}{f_{ck}}}\right] \quad \text{... ...(4.1.6c)}$$

4.1.4 Properties of Singly Reinforced Balanced Section

The strain distribution and stress distribution diagrams of a balanced section are shown in *Fig. 4.1.3*

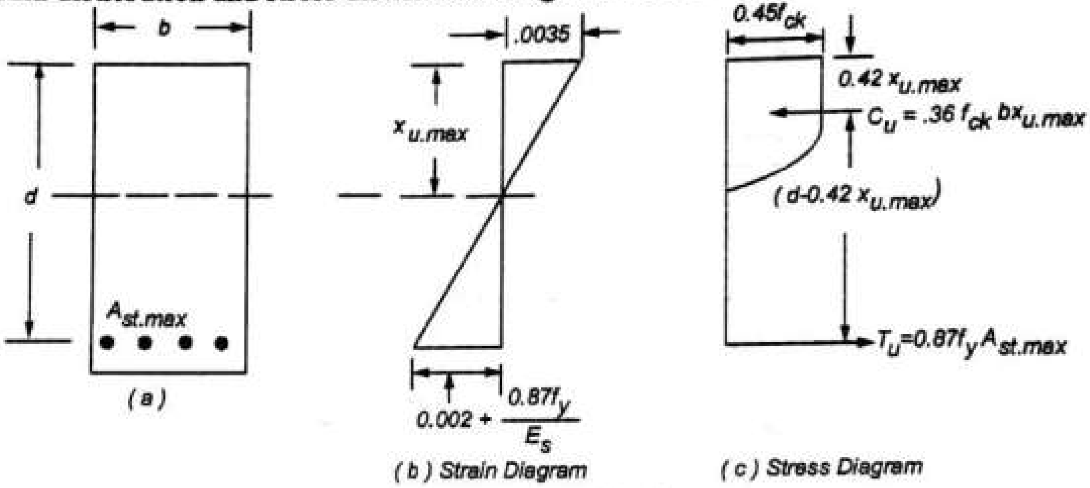


Fig. 4.1.3 Balanced Section

As mentioned earlier that I.S. Code does not allow over - reinforced section, the depth of balanced neutral axis will be the maximum depth of neutral axis, $x_{u.max.}$

From *Fig. 4.1.3b* the depth of neutral axis is given by :

$$\frac{x_{u.max}}{d - x_{u.max}} = \frac{0.0035}{0.002 + 0.87\, f_y / E_s}$$

Substituting $E_s = 2 \times 10^5\ N/mm^2$, $x_{u.max} = \frac{700}{1100 + 0.87\, f_y} \times d$ ... ... (4.1.1.c)

$$or \quad k_{u.max} = \frac{x_{u.max}}{d} = \frac{700}{1100 + 0.87\, f_y} \qquad \text{... ... (4.1.2.c)}$$

All the design parameters *viz.* x_u ,A_{st} , M_{ur} *etc* will have maximum values and they will be obtained by replacing x_u, A_{st}, M_{ur}, R_u by $x_{u.max}$, $A_{st.max}$, $M_{ur.max}$ and $R_{u.max}$ respectively in equations given in *Sect. 4.1.3*

$$M_{ur.max} = 0.36\, f_{ck}\, b x_{u.max}\, (d - 0.42\, x_{u.max}) = R_{u.max}\, bd^2 = M_u \qquad \text{... ... (4.1.5.a)}$$

where, $R_{u.max} = 0.36\, f_{ck}\, k_{u.max}\, (1 - 0.42\, k_{u.max})$ ... ... (4.1.5.b)

$$p_{t.max} = \frac{0.36\, f_{ck}}{0.87\, f_y}\, k_{u.max} \quad or \quad A_{st.max} = \frac{0.36\, f_{ck}\, b\, x_{u.max}}{0.87\, f_y} \qquad \text{... ... (4.1.6.d)}$$

The design parameters for a balanced section are given in *Table 4.1.1*

Table 4.1.1 : Design Parameters for a Balanced Section

Concrete Grade	M20			M25			M30		
Steel Grade	Fe 250	Fe 415	Fe 500	Fe 250	Fe 415	Fe 500	Fe 250	Fe 415	Fe 500
$k_{u.max}$	0.53	0.48	0.46	0.53	0.48	0.46	0.53	0.48	0.46
$R_{u.max}$	2.97	2.76	2.67	3.71	3.45	3.34	4.45	4.14	4.00
or $R_{u.max}$	$0.149 f_{ck}$	$0.138 f_{ck}$	$0.133 f_{ck}$	$0.149 f_{ck}$	$0.138 f_{ck}$	$0.133 f_{ck}$	$0.149 f_{ck}$	$0.138 f_{ck}$	0.133_{ck}
$p_{t.max}$ %	1.76	0.96	0.76	2.20	1.20	0.95	2.63	1.43	1.14

Note : Values of $R_{u.max}$ and $p_{t.max}$ % have been obtained from the rounded values of $k_{u.max}$

All design equations derived earlier are given in *Table 4.1.2* for ready reference.

Table 4.1.2 Singly Reinforced Sections - Summary of Equations

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b} \quad \text{... ... 4.1.1a}$$

$$k_u = \frac{0.87 f_y}{0.36 f_{ck}} \cdot \frac{A_{st}}{bd} = \frac{0.87 f_y}{0.36 f_{ck}} p_t \quad \text{... ...4.1.2a}$$

$$x_u = 1.2\left(1 - \sqrt{1 - \frac{4.62\, M_u}{f_{ck}\, bd^2}}\right) \times d \quad \text{... ...4.1.1b}$$

$$k_u = 1.2\left(1 - \sqrt{1 - \frac{4.62\, R_u}{f_{ck}}}\right) \quad \text{... ...4.1.2b}$$

$$x_{u.max} = k_{u.max} \times d = \frac{700}{1100 + 0.87 f_y} \cdot d \quad \text{... ... 4.1.1c}$$

$$k_{u.max} = \frac{700}{1100 + 0.87 f_y} \quad \text{... ... 4.1.2c}$$

$$M_{ur} = 0.87 f_y A_{st} (d - 0.42\, x_u) = M_u \quad \text{... ...4.1.3a}$$

$$M_{ur} = 0.87 f_y A_{st} d\left(1 - \frac{f_y}{f_{ck}} \cdot \frac{A_{st}}{bd}\right) \quad \text{... ... 4.1.3b}$$

$$M_{ur} = 0.36 f_{ck}\, b.x_u\, (d - 0.42\, x_u) = M_u = R_u\, bd^2 \quad \text{... ...4.1.4a}$$

$$R_u = 0.36 f_{ck}\, k_u (1 - 0.42\, k_u) = M_u \quad \text{... ...4.1.4b}$$

$$M_{ur.max} = R_{u.max}\, bd^2 = 0.36 f_{ck}\, b.x_{u.max}\, (d - 0.42\, x_{u.max}) \quad \text{... ... 4.1.5a}$$

$$R_{u.max} = 0.36 f_{ck}\, k_{u.max}\, (1 - 0.42\, k_{u.max}) \quad \text{... ...4.1.5b}$$

$$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6\, M_u}{f_{ck}\, bd^2}}\right] bd \quad \text{... ... 4.1.6a}$$

$$A_{st} = 0.36 f_{ck}\, b.x_u / (0.87 f_y) \text{ or } p_t = 0.36 f_{ck}\, k_u / (0.87 f_y) \quad \text{... ... 4.1.6b}$$

$$p_t = \frac{A_{st}}{bd} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6\, R_u}{f_{ck}}}\right] \quad \text{... ... 4.1.6c}$$

$$A_{st.max} = 0.36 f_{ck} b.x_{u.max} / (0.87 f_y) \text{ or } p_{t.max} = 0.36 f_{ck} k_{u.max} / (0.87 f_y) \text{... 4.1.6d}$$

4.1.5 Redistribution of Moments

The main feature of limit state design for collapse is the redistribution of moments in *statically indeterminate beams*. The phenomenon of transferring moments or forces to any other section which has reserved load/moment carrying capacity is called *redistribution of moments/forces*.

In Sect.3.1.2 and Sect.3.1.3 it is shown how redistribution of moment is advantages in carrying increasing load carrying capacity.

The R.C. members have limited rotation capacity and further large rotation canot be permited since it will affect serviceability *i.e* deflection and cracking. Hence limitations have been imposed on rotation and corresponding amount of redistribution.

The rotation capacity is an inverse function of depth of neutral axis *i.e.* as the rotation capacity goes on increasing, the depth of the neutral axis goes on decreasing. Therefore, the design codes impose limitations on the depth of neutral axis in proportion to the amount of redistribution of moments as given by the following equation :

$$\frac{x_u}{d} + \frac{dM}{100} = 0.6 \quad or \quad k_u = 0.6 - \frac{dM}{100}$$

where, dM = percentage reduction in moment < = 30%

∴ Limiting depth of neutral axis is given by :

$$x_{u.limit} = \left(0.6 - \frac{dM}{100}\right) \times d \quad or \quad x_{u.max} \text{ whichever is less} \qquad (4.1.1d)$$

$$\text{or } k_{u.limit} = \left(0.6 - \frac{dM}{100}\right) \quad or \quad k_{u.max} \text{ whichever is less}$$

The design examples involving redistribution of moments continue to be the same except that the depth of neutral axis to be used is $x_{u.limit}$ instead of $x_{u.max}$ provided $x_{u.limit} < x_{u.max}$ otherwise $x_{u.limit}$ is taken equal to $x_{u.max}$. The corresponding design parameters can be obtained replacing $k_{u.max}$, $R_{u.max}$, $p_{t.max}$ by $k_{u.limit}$, $R_{u.limit}$ and $p_{t.limit}$ respectively.

The values of $k_{u.limit}$, $R_{u.limit}$ and $p_{t.\ limit}$ are given by :

$$k_{u.limit} = \left(0.6 - \frac{dM}{100}\right) \not> k_{u.max}\ .\ else \text{ take } k_{u.limit} = k_{u.max} \qquad \dots\dots (4.1.2d)$$

$$R_{u.limit} = 0.36 f_{ck}\, k_{u.limit}\,(1 - 0.42\, k_{u.limit}) \qquad \dots\dots (4.1.5c)$$

$$M_{ur.limit} = R_{u.limit}\, bd^2 = 0.36 f_{ck}\, k_{u.limit}\,(1 - 0.42\, k_{u.limit})\, bd^2 \qquad \dots\dots (4.1.5d)$$

$$p_{t.limit} = 0.36 f_{ck}\, k_{u.limit} / (0.87 f_y) \qquad \dots\dots (4.1.6\ e)$$

4.1.6 Redistribution of Moments for Practical Applications

Consider a beam carrying moment $(M + dM)$ out of which additional moment dM can be reduced. It will be seen from the following figure that if moment dM is reduced at both the ends then there is increase in moment by dM at mid-span as shown in Fig. 4.1.5.

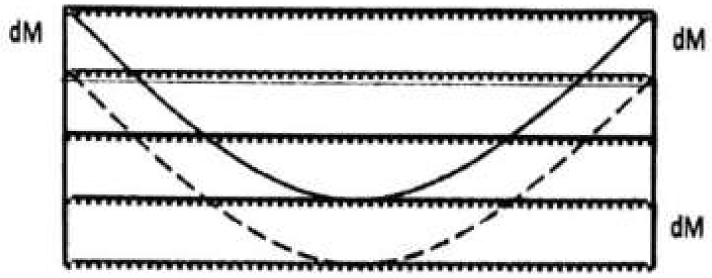


Fig. 4.1.5 Redistribution of Moments at Both Ends

And if the moment is reduced by dM at one end only then there is increase in moment by $dM/2$ at the midspan as shown in Fig. 4.1.6

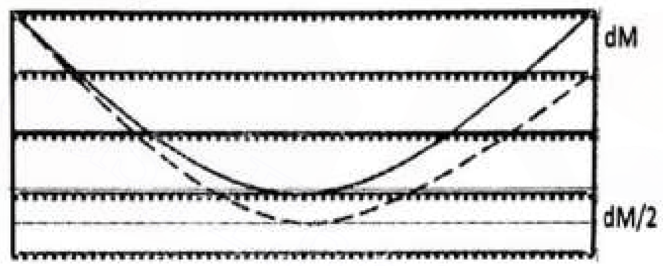


Fig. 4.1.6 Redistribution of Moments at One End

When the redistribution of moment is dM_A at one end and dM_B at the other end, the increase in span moment is $(dM_A/2 + dM_B/2)$

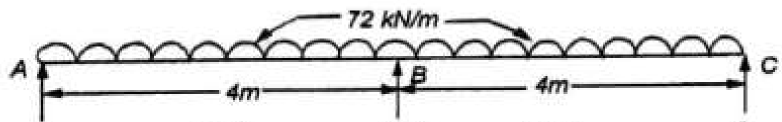
For example cosider a two-span continuous beam shown below

Span AB :

$M_B = w_u L^2/8 = 72 \times 4^2/8 = 144\ kN.m.$

$R_A = 72 \times 4/2 - 144/8 = 108\ kN.$

$x_{max} = 108/72 = 1.5m$ from $A.$, $M_{max} = 108 \times 1.5 / 2 = 81 kN.m.$



Final moments	0		81.0	− 144	81.0		0
dM = 30% @ B	-	-		43.2	-	-	
Increase in span moment = $dM/2$	-		21.6	-	21.6	-	
Final moment	0		102.6	− 100.8	102.6		0

It may be mentioned that normally the span moments obtained by approximate method are on higher side.

It is emphasized that maximum value of dM i.e. percentage reduction in moment is 30%. This does not mean we should always reduce the moment by 30%. It can be less than 30% to prevent excessive demand on the ductility of a structural member. About 15% to 20% moment redistribution may be taken reasonable limit. This gives some additional margin of safety.

The advantages of redistribution of moments have been given in Sect.3.1.3 .

4.2 DOUBLY REINFORCED RECTANGULAR SECTION

A singly reinforced balanced section can resist maximum moment $M_{ur.max}$ to avoid sudden failure. But if moment resisting capacity is required to be increased then the compression force is increased by providing steel in compression zone and additional steel in tension zone. Thus, **when the section is reinforced both in tension and compression zone it is called Doubly Reinforced Section.**

The doubly reinforced section is provided in the following situations:

(1) Moment resisting capacity of a singly reinforced section is inadequate.

(2) In the beam-floor system the beam is subjected to tension at the bottom in the mid-span egion while tension occurs in support section Fig. 4.2.1

In floor-beam system the slab is provided over the beam. In the case of continuous beam, the beam deflects with concavity upwards producing tension at the bottom face (Fig.4.2.1 Sect-AA), while it bends with convexity upwards developing tension at the top face (Fig.4.2.1 Sect-BB). The maximum moment at support is much greater then the span moment. The mid-section of the beam acts as a T-section providing more compressive force and capable of resisting more moment (Fig.4.2.1 Sect-AA). If the section provided is kept the same then the singly reinforced section is incapable of resisting more moment. In such a case the resisting capacity of the beam is increased by providing compression and additional tension reinforcement as shown in (Fig-4.2.1 Sect. BB) making the section doubly reinforced.

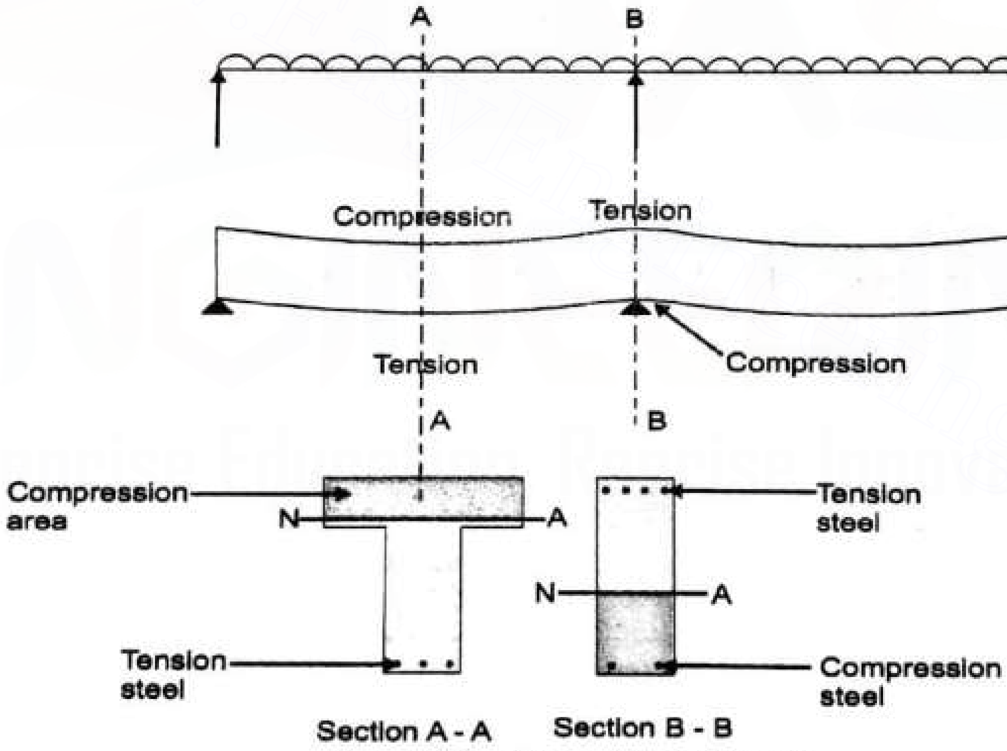


Fig. 4.2.1 Beam - Floor System

3) Beam section is restricted due to functional requirements.

In a big assembly hall or marriage hall or cinema hall, the columns are required to be provided at the ends only to satisfy the functional requirements. The span of the beam in such a case becomes large and it is to be designed as singly reinforced section, subjected to large moment, it will require much more depth. But when the depth of the beam is large the view of public gets obstructed and it also creates the psychological effect on people they may feel lack of safety. To obviate the difficulty the beam is reinforced on tension and compression side so that the depth is restricted and the beam can carry the required load and also satisfy the functional requirements.

4.2.1 Properties of Doubly Reinforced Section

A doubly reinforced section shown in Fig. 4.2.2a is considered to be composed of a singly reinforced *Sect.-I* resisting part moment M_{u1}, and balanced moment M_{u2} (= M_u - M_{u1}) resisted by *Sect - II*

The properties of the section are worked out as under.

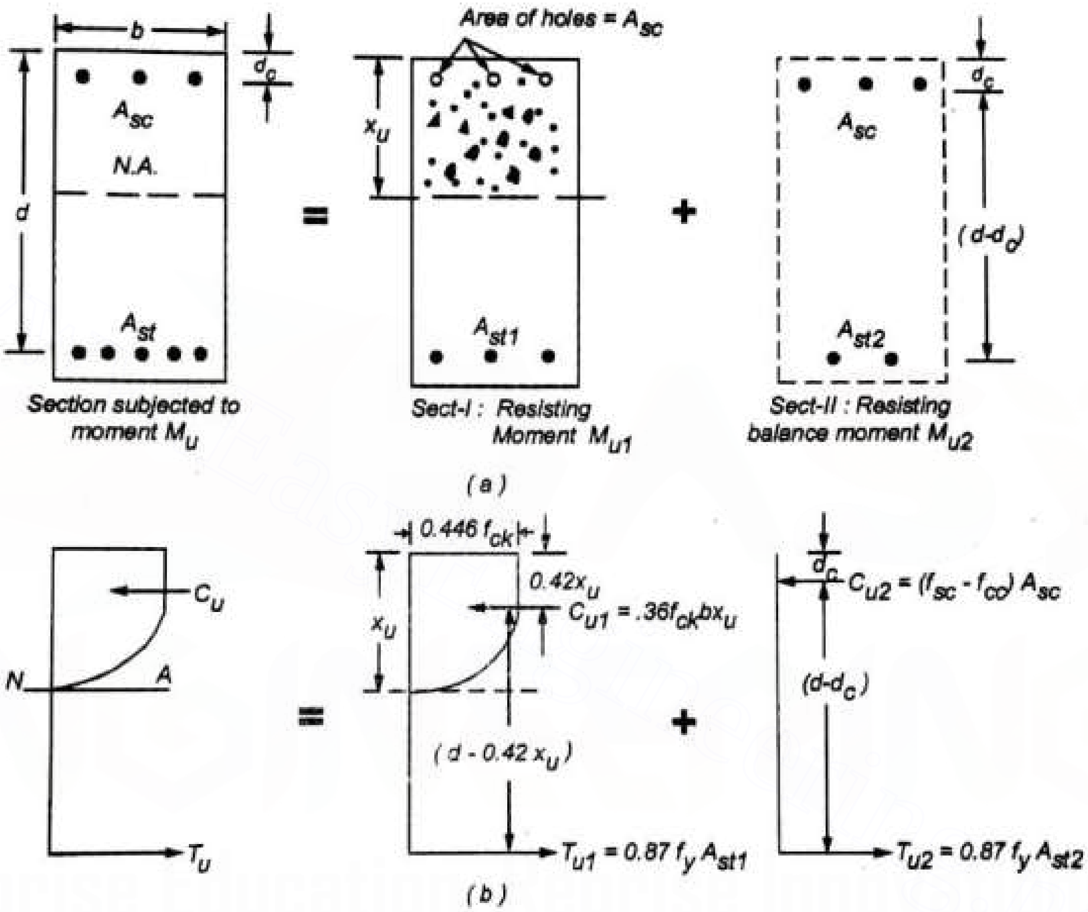


Fig. 4.2.2a Doubly Reinforced Section

(a) Depth of Neutral Axis

Equating total compression in concrete and compression steel (A_{sc}) with total tension in tension steel (A_{st}) we get,

$$(0.36 f_{ck}\, b\, x_u - f_{cc}\, A_{sc}) + f_{sc}\, A_{sc} = 0.87\, f_y\, A_{st}$$

$$\therefore\ x_u = \frac{0.87 f_y A_{st} - (f_{sc} - f_{cc})\ A_{sc}}{0.36\ f_{ck}\ b} \quad \text{... ...(4.2.1)}$$

where , f_{cc} = stress in concrete in compression steel = 0.446 $f_{ck} \cong 0.45 f_{ck}$
f_{sc} = stress in compression steel
b = width of the beam

4.2.2 Moment of Resistance

The moment of resistance of a doubly reinforced section is obtained by taking moments of C_{u1} and C_{u2} about the centroid of tension steel *(Fig. 4.2.1).*

$$M_{ur} = M_u = M_{u1} + M_{u2} = 0.36 f_{ck}\, b\, x_u\, (d - 0.42\, x_u) + (f_{sc} - f_{cc})\,(d - d_c) \qquad \text{... ...(4.2.2a)}$$

$$\cong 0.36 f_{ck}\, b\, x_u\, (d - 0.42\, x_u) + f_{sc}\,(d - d_c) \qquad \text{... ...(4.2.2b)}$$

where, d = effective depth, and d_c = effective cover to compression steel.

4.2.3 Area of Tension and Compression Steel for *Design Problems*

For *design problems* the singly reinforced *Sect -.1 (Fig 4.2.2b)* is kept balanced to make full utilization of resistance of concrete and the remaining moment ($M_{u2} = M_u - M_{ur.max}$) is resisted $(C_{u2}-T_{u2})$ couple *acting at lever arm distance (d -d$_c$) as shown in Fig. 4.2.2b and required area of steel is provided.*

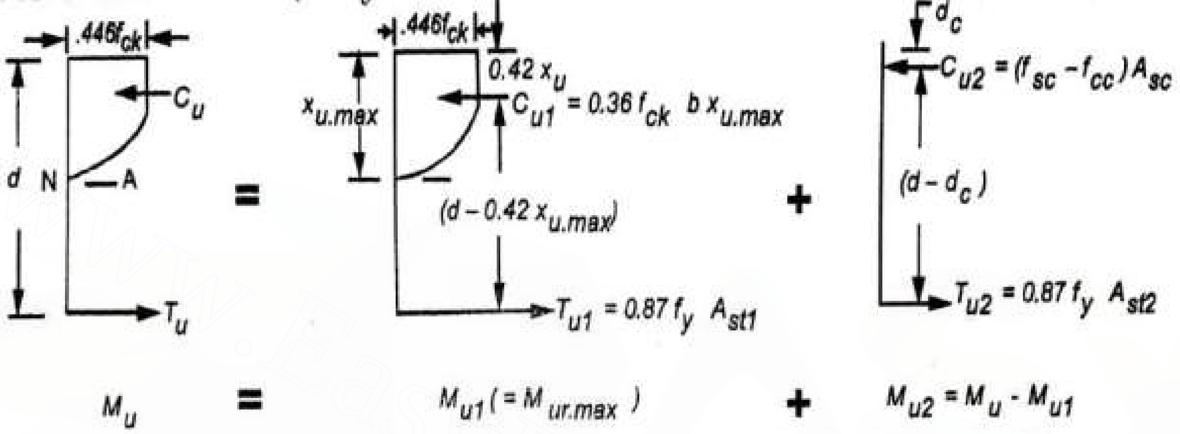


Fig. 4.2.2b Stress Distribution in Doubly Reinforced Section for Design Problems

For balanced section $x_u = x_{u.max}$, $M_{u1} = M_{ur.max}$, $= R_{u.max}\, bd^2$

and $A_{st1} = A_{st.max} = p_{t.max}\, bd$

$$A_{st1} = A_{st.max} = \frac{M_{ur.max}}{0.87 f_y\,(d - 0.42\, x_{u.max})} \qquad \text{... ...(4.2.3a)}$$

$$A_{st2} = \frac{M_{u2}}{0.87 f_y\,(d - d_c)} \qquad \text{... ...(4.2.3b)}$$

Total area of tension steel = $A_{st} = A_{st1} + A_{st2}$

$$A_{sc} = \frac{0.87 f_y\, A_{st2}}{(f_{sc} - f_{cc})} \cong \frac{0.87\, f_y\, A_{st2}}{f_{sc}} \qquad \text{... ...(4.2.3c)}$$

Alternatively, A_{sc} can be obtained from the relation,

$$M_{u2} = C_{u2}\,(d - d_c) = (f_{sc} - f_{cc})\, A_{sc}\,(d - d_c)$$

$$A_{sc} = \frac{M_{u2}}{(f_{sc} - f_{cc}) \times (d - d_c)} \cong \frac{M_{u2}}{f_{sc}\,(d - d_c)} \qquad \text{... ...(4.2.3d)}$$

4.2.4 Stress in Compression Steel

The stress in compression steel f_{sc} depends on the strain ε_{sc} at the level of compression steel and is obtained from the appropriate stress - strain curve.

For mild steel Fe 250, $f_{sc} = E_s \times \varepsilon_{sc} = (2 \times 10^5) \times 0.0035\,(1 - d_c/x_{u.max}) = 700(1 - d_c/x_{umax}) > 0.87 f_y$ (4.2.4a)

For HYSD bars,

From *Fig. 4.2.3*, $\varepsilon_{sc} = 0.0035\left(1 - \frac{d_c}{x_{u.max}}\right) = 0.0035\left(1 - \frac{d_c/d}{k_{u.max}}\right)$ ... ...(4.2.4 b)

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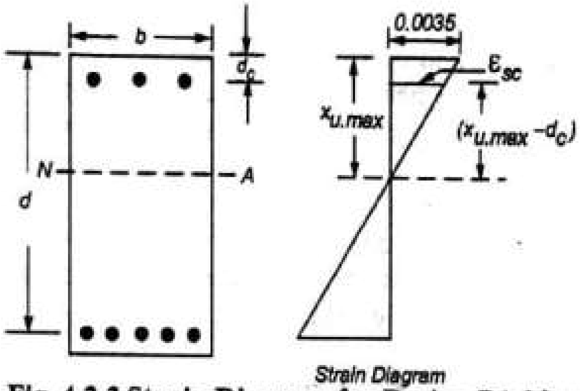


Fig. 4.2.3 Strain Diagram for Design Problems

Values of f_{sc} are obtained from *Table 4.2.1* corresponding to ε_{sc}.

Table 4.2.1 : Stress-Strain Relation for High Yield Strength Steels

	Fe 415		Fe 500	
Stress level	*Total Strain*	*Stress N/mm²*	*Total Strain*	*Stress N/mm²*
0.80 f_{yd}	0.00144	288.7	0.00174	347.8
0.85 f_{yd}	0.00163	306.7	0.00195	369.6
0.90 f_{yd}	0.00192	324.8	0.00226	391.3
0.95 f_{yd}	0.00241	342.8	0.00277	413.0
0.975 f_{yd}	0.00276	351.8	0.00312	423.9
1.00 f_{yd}	0.00380	360.9	0.00417	434.8

Table 4.2.2 gives values of f_{sc} directly for different values of d_c/d corresponding to k_u while *Table 4.2.3* directly gives values of $0.87 f_y/(f_{sc} - f_{cc})$ for different values of d_c/d which when multiplied by A_{st2} gives A_{sc}.

Table 4.2.2 : Values of f_{sc} for High Grade Steels in N/mm^2

	Fe 415				Fe 500			
	d_c/d				d_c/d			
k_u	0.05	0.10	0.15	0.20	0.05	0.10	0.15	0.20
0.30	353	340	314	233	417	393	349	233
0.35	354	345	328	294	420	401	373	300
0.40	354	348	334	314	422	407	386	349
0.45	355	351	340	325	423	411	394	369
0.46	–	–	–	–	**424**	**412**	**395**	**371**
0.48	**355**	**352**	**342**	**329**	–	–	–	–

Table 4.2.3 Values of $(0.87f_y)/(f_{sc} - f_{cc})$ for Different Ratios of d_c/d for HYSD Bars

Concrete mix	Fe415 and (k_u = 0.48) d_c/d				Fe500 and (k_u = 0.46) d_c/d			
	0.05	0.10	0.15	0.20	0.05	0.10	0.15	0.20
M20	1.043	1.053	1.084	1.128	1.048	1.079	1.126	1.201
M25	1.050	1.059	1.091	1.136	1.054	1.085	1.133	1.209

4.3 FLANGED SECTION

Even though details have been given in *Sect-3* it is repeated to emphasize that in the case of flanged section part of the slab acts along with the beam in resisting compressive forces provided *slab lies in the compression zone with respect to bending of the beam* and both slab and beam are cast together and tied properly with each other.

4.3.1 Effective Flange width *(clause 23.1.2)*

In absence of more accurate determination, the effective flange width may be taken as following, but in no case greater than the breadth of web plus half the clear diatance to adjacent beams on either side i.e actual width (B) as specified by Code (clause 23.1.2).

For T-eam $b_f = \frac{L_o}{6} + b_w + 6\,D_f \leq$ *Actual width (B)*

For L-beam $b_f = \frac{L_o}{12} + b_w + 3\,D_f \leq$ *Actual width (B)*

The effective width b_f of the flange is written in the general form as :

$$b_f = k\left(\frac{L_o}{6} + 6D_f\right) + b_w < \text{c/c distance between beams} \quad \text{... ... (4.3.1)}$$

for Isolated beams :

$$b_f = k\left(\frac{L_o}{L_o/b + 4}\right) + b_w \leq \text{actual width} \quad \text{... ... (4.3.2)}$$

where, k = 1 for T - beam , k = 1/2 for L - beam , k = 0 for rectangular beam.

b_w = width of web ,

D_f = depth of flange = thickness of slab ,

L_o = distance between points of zero bending moment

for continuous beam L_o may be taken equal to 0.7 x effective span.

4.3.2 Properties of Flanged Section

Case - I Neutral axis lying in the flange *i.e.* $x_u \leq D_f$ (See *Fig. 4.3.1*)

(a)Depth of Neutral Axis

$$\text{Depth of Neutral axis, } x_u = \frac{0.87\, f_y\, A_{st}}{0.36\, f_{ck}\, b_f} \not> D_f \quad \text{... ... (4.3.3)}$$

(b) Moment of Resistance M_{ur}

$$M_u = M_{ur} = 0.36\, f_{ck}\, b_f\, x_u\, (d - 0.42\, x_u) \quad \text{... ... (4.3.4a)}$$

(c)Area of Steel

$$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6\, M_u}{f_{ck}\, b_f\, d^2}}\right] \times b_f \times d \quad \text{... ...(4.3.5)}$$

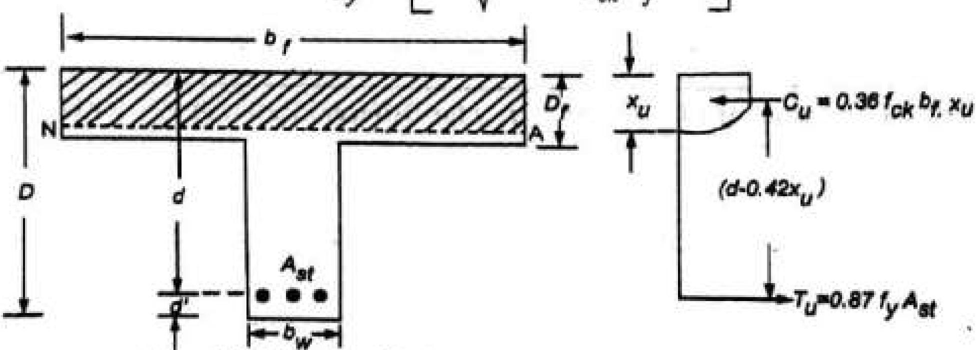


Fig. 4.3.1 Neutral Axis Lying Inside the Flange

Case - II Neutral axis lying inside the web (*i.e.* $x_u > D_f$) :

In this case, there are two possibilities.

Case - IIa Depth of flange is greater than the depth of rectangular part of the stress block

i.e. $D_f > (3/7)\ x_u$ *or* $x_u < 7D_f / 3$.

In this case, the depth of the neutral axis may be obtained from the relation,

$$0.36 f_{ck}\ b_w\ x_u + 0.45\ f_{ck}\ (b_f - b_w)\ y_f = 0.87 f_y A_{st} \qquad \text{... ... (4.3.6a)}$$

where, $y_f = 0.15x_u + 0.65\ D_f$ not greater than D_f ... ...(4.3.6b)

Case - IIb Depth of flange is less than the depth of rectangular part of the stress block

i.e. $D_f \leqslant (3/7)\ x_u$ *or* $x_u \geqslant 7D_f / 3$.

In this case, the depth of the neutral axis is obtained using *(Eq.4.3.6)* only, except that y_f will be replaced by D_f

Moment of Resistance (M_{ur})

$$M_{ur} = M_{uw} + M_{uf}$$

$$= 0.36\ f_{ck}\, b_w\, x_u\,(d - 0.42x_u) + 0.45 f_{ck}\,(b_f - b_w)\, y_f\,(d - y_f/2) \qquad \text{... ...(4.3.7)}$$

For $x_u \geqslant 7D_f/3$, replace y_f by D_f in the above equation.

(c) *Area of Steel* (A_{st})

In design, we are required to obtain the area of steel for a given or assumed section to resist a given design moment M_u. In this case, it is necessary to know in advance whether the neutral axis would lies in flange *or* in web for the given moment. To know this, given M_u is compared with M_{ur1} of the section for $x_u = D_f$ given by :

For $x_u = D_f$, $(M_{ur1}) = 0.36 f_{ck}\, b_f\ D_f\,(d - 0.42\ D_f)$ ... ...(4.3.8)

If $M_u \leqslant M_{ur1}$, then $x_u \leqslant D_f$ and A_{st} may be obtained by using *Eq. 4.3.5*

If $M_u > M_{ur1}$, then $x_u > D_f$ and then x_u will first be obtained by using *Eq. 4.3.7.*

A_{st} can then be obtained by using *Eq. 4.3.6.*

Comments : *In most of the cases, for normal loads and normal spans of beams, and when the depth of the beam is taken between L/12 to L/16, the neutral axis lies inside the flange only. Case - II is of rare occurrence, therefore, detailed equations have not been given.*

4.4 SHEAR

4.4.1 Cracking in Beam

Initially it is necessary to know the nature of cracking due to bending and shear. Consider a simply supported beam. Depending on the ratio of bending moment and shear force three different regions can be identified in the beam as shown in Fig.4.4.1

In Region -1 In the midspan of a simply supported beam, the bending stresses are maximum and shear force is zero and the tensile stresses acting horizontally (shown in Fig. 4.4.1a) are maximum at the bottom due to which the crack occurs in the vertical direction at the bottom. crack occurs in the diagonal direction as shown in Fig. 4.1.1c

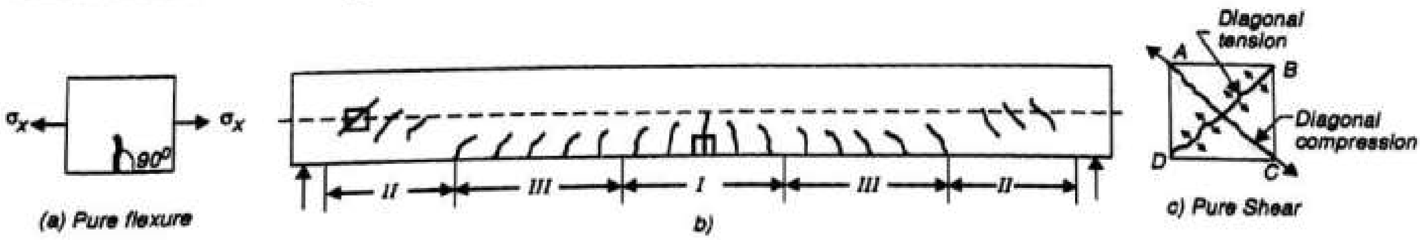


Fig. 4.4.1 Cracking in Beam

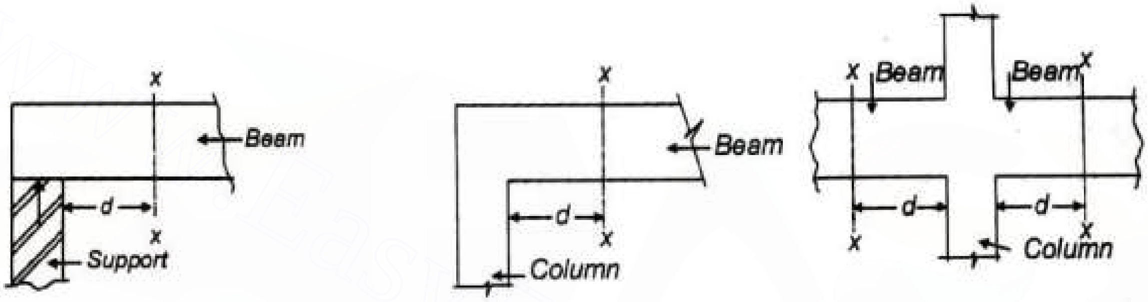
In Region - 2 Near the support the shear force is maximum while bending moment is zero the crack occurs in the diagonal direction as shown in Fig. 4.1.1c.

In Region - 3 This region is near the quarter span where the bending moment is still considerable and shear force is also significant. The cracking will be the combination of region 1 and region -2. The crack emerges at the tension face in the vertical direction due to flexure and gradually tend to bend in the diagonal direction towards the neutral axis as the shear stress goes on increasing and bending moment decreasing.

4.4.2 Critical Section for Shear *(clause 22.6.2)*

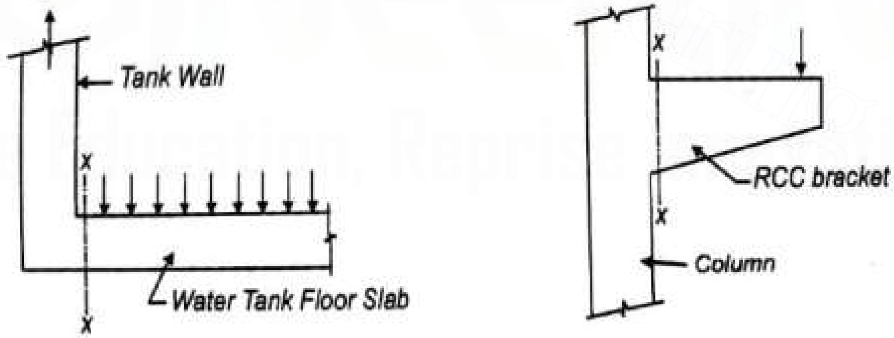
When the support offers compressive reaction in the end region, the diagonal crack gets displaced away from the face of the support and hence the sections located at a distance less than '*d*' from the face of support may be designed for the same shear as that computed at distance *d* Fig. 4.4.2(a).

Comments: In practice, many-a-times, shear is taken at its maximum value of end shear , instead of finding it at critical section '*d*' to reduce the computational efforts, which is on the safer side.



(a) CriticaL Section at a distance '*d*' from the Face of Support

While, when the support reaction introduces tension in the end region, a diagonal crack is likely to develop from the face and hence the design shear is taken equal to the shear at the face of the support. Fig. 4.4.2b



(b) Critical Section at the Face of Support

Fig. 4.4.2 Critical Section for Shear

4.4.3 Design Shear Force (V_{uD})

Let the maximum shear force at the end of a beam carrying a uniformly distributed load of intensity w_u be $V_{u.max}$

Design shear force for support offering compression reaction is given by :

$$V_{uD} = V_{u.max} - w_u (b_s/2 + d) \qquad \text{... ... (4.4.1a)}$$

Design shear force for support offering tension reaction

$$V_{uD} = V_{u.max} - w_u b_s / 2 \qquad \text{... ... (4.4.1b)}$$

where, b_s = the Breadth of Support

4.4.4 Shear Strength of Section in Diagonal Compression ($V_{uc.max}$) *(clause 40.2.3)*

$$V_{uc.max} = \tau_{uc.max}\ bd \qquad \text{... ... (4.4.2)}$$

where , $\tau_{uc.max}$ depends upon the grade of concrete.

$\tau_{uc.max}$ = 2.8 N/mm^2 for concrete grade M20 and $\tau_{uc.max}$ = 3.1 N/mm^2 for concrete grade M25

If shear at a section exceeds $V_{uc.max}$, the section is inadequate and should be revised either by changing *b* or *d*. For solid slabs the nominal shear stresses shall not exceed half the appropriate values given above.

4.4.5 Shear Resistance of R.C. Member (V_{uc}) with Main Steel but without Shear Reinforcement *Fig.4.4.3 (clause 40.2.1)*

When a beam is provided with main steel (*i.e.* without shear reinforcement), the shear is jointly resisted by shear resisted by concrete in compression, by aggregate interlock and the dowel action of main tension steel.

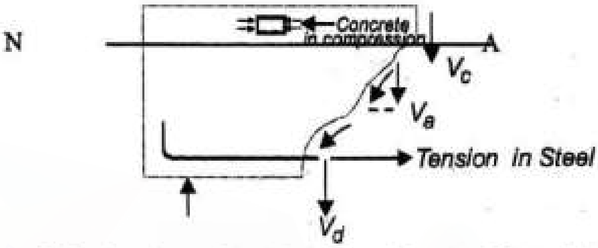


Fig. 4.4.3 Shear Failure Mechanism for Beam without Shear Reinforcement.

Since the shear resistance due to aggregate interlock is effective only after the development of diagonal crack, it is ignored by the Code and kept as reserve strength. Shear resisted by dowel action and concrete in compression which is function of grade of concrete and the percentage of tension steel.

It is given by the following relation :

$$V_{uc} = \tau_{uc}\ bd \qquad \text{... ... (4.4.3a)}$$

V_{uc} = shear resistance of R.C. member without shear reinforcement (but with main reinforcement) is many times loosely termed as shear resistance of concrete.

The values of τ_{uc} for concrete grades M20 and M25 are given in *Table 4.4.1*

Table 4.4.1 Design Shear Strength of Concrete τ_{uc} in N/mm^2

$\frac{100 A_{st}}{bd}$	Concrete Grade		
	M15	M20	M25
≤0.15	0.28	0.28	0.29
0.25	0.35	0.36	0.36
0.50	0.46	0.48	0.49
0.75	0.54	0.56	0.57
1.00	0.60	0.62	0.64
1.25	0.64	0.67	0.70
1.50	0.68	0.72	0.74
1.75	0.71	0.75	0.78
2.00	0.71	0.79	0.82
2.25	0.71	0.81	0.85
2.50	0.71	0.82	0.88
2.75	0.71	0.82	0.90
3.00 and above	0.71	0.82	0.92

Note : The term A_{st} is the area of longitudinal tension reinforcement which continues at least one effective depth beyond the section being considered except at supports where full area of tension steel may be used, provided bars are anchored properly according to code requirements.

For slab, $V_{uc} = \tau_{uc}\, b\, d \times k$, where k to be obtained from *Table 4.4.2*

The multiplying factor k accounts for increase in resistance of the slab due to membrane action as its thickness is very small in relation to its width. It is obtained from *Table 4.4.2.*

Table 4.4.2 Multiplying factor k to obtain design shear in slab							
Overall Depth of slab in *mm*	≥ 300	275	250	225	200	175	≤ 150
Values of k	1.00	1.05	1.10	1.15	1.20	1.25	1.30

4.4.6 Shear Resistance of Shear Reinforcement (V_{us}) (clause 40.4)

Types of Shear Reinforcement :

(i) Bent - up Bars, (ii) Vertical Stirrups,

(iii) Inclined Stirrups *or* Series of Bent - up Bars.

Shear Resistance of Bent up Bars at a section :

$$V_{usb} = 0.87 f_y\, A_{sb}\, \sin\alpha = 0.87 f_y\, A_{sb} \times 0.707 \quad \text{for } \alpha = 45^0 \qquad \dots\dots(4.4.4)$$

where, A_{sb} = area of bent up bar/s,

α = inclination of bent - up bar/s with the axis of the member.

Shear Resistance of vertical stirrups with area of vertical legs A_{sv} ,

$$V_{usv} = \frac{0.87 f_y A_{sv}\, d}{s} \quad or \quad s = \frac{0.87 f_y A_{sv}\cdot d}{V_{usv}} \qquad \dots\dots(4.4.5)$$

where, s = spacing of stirrups.

Total Shear Resistance of Shear Reinforcement for combination of bent up bars and stirrups :

$$V_{us} = V_{usb} + V_{usv}, \text{ but } V_{usb} \not> 0.5\, V_{us} \qquad \dots(4.4.6)$$

Minimum Stirrups : Minimum shear reinforcement in the form of stirrups is given by :

$$\frac{A_{sv}}{bs} > \frac{0.4}{0.87 f_y} \quad or \quad s \leqslant \frac{0.87 f_y}{0.4\, b} A_{sv} \qquad \dots\dots(4.4.7)$$

Shear Resistance of Minimum Stirrups :

$$V_{usv.min} = 0.4bd \qquad \dots\dots(4.4.8)$$

Shear Resistance of a R.C. member with Minimum Stirrups :

$$V_{ur.min} = V_{uc} + V_{usv.min} = V_{uc} + 0.4bd \qquad \dots\dots(4.4.9)$$

Maximum spacing : $s \not< 0.75d$ *or* 300*mm* whichever is less. *(clause 26.5.1.5)*

Minimum Spacing : $s < 75$ *mm* for ease of concreting.

The steps for design of shear reinforcement have been given in *Sect. 6.3.4 Step No. 9*

4.4.7 Shear Design in Case of Bar Curtailment *(clause 26.2.3.2)*

No bar in tension region shall be curtailed unless any one of the following conditions is satisfied.

(i) Shear resistance at the section is not less than 1.5 x shear at the section.

(ii) Additional stirrups with area equal to area of minimum stirrups are provided for a distance 0.75d beyond the cutoff point with resultant spacing not exceeding $d/(8\beta)$
where, β is the ratio of area of curtailed bars to area prior to curtailment.

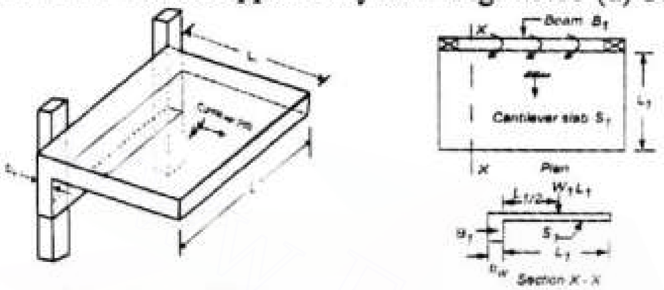
(iii) The continuing bars provide double the area required for flexure at the point of cutoff and the shear at the section does not exceed 3/4th of the shear resistance at the section.

4.5 Torsion

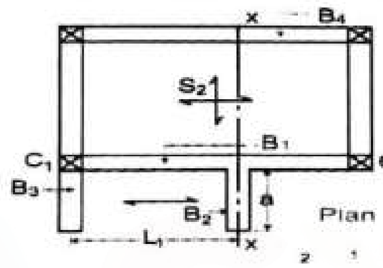
Torsion occurs either due to eccentric load lying in the plane contained by the cross-section, or due to rotational compatibility between interconnected members.
Therefore , torsion is classified into two categories.
(a) Equilibrium torsion (b) Compatibility torsion

4.5.1 Equilibrium Torsion.

It is the torsion induced to maintain equilibrium in the structure. In statically determinate structures only equilibrium torsion exists. Equilibrium torsion must be transferred to supports by torsional resistance of members and cannot be ignored. The neglect of the same would result in violation of equilibrium condition leading to disastrous collapse. Some of the examples of equilibrium torsion are (a) Cantilever slab inducing torsion in supporting beam Fig. 4.5.1a (b) Cantilever beam induces torsion in supporting beam Fig. 4.5.1b (c)Cantilever shell supported by beam Fig. 4.5.1c (d) Out of plane load inducing torsion in support.Fig. 4.5.1d.



(a) Cantilever slab inducing torsion in supporting beam



(b) Cantilever beam B_2 inducing torsion inbeam B_1

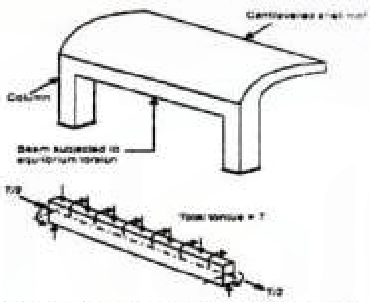
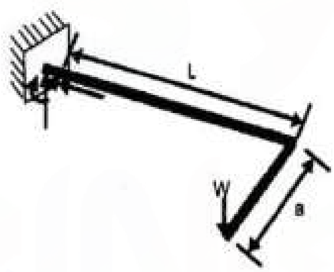


Fig.4.5.1 Equilibrium Torsion

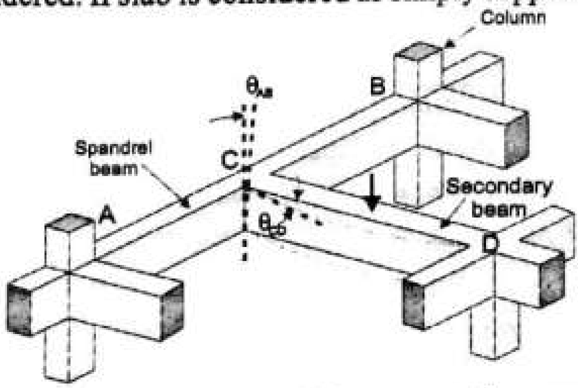


(c) Cantilever shell inducing torsion in beam

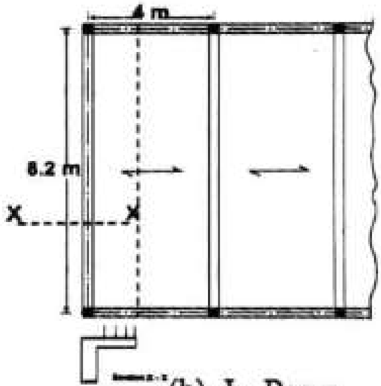
(d) Out of plane load inducing torsion at support.

4.5.2 Compatibility torsion

It is the torsion induced in the member due to compatibility of rotations at the joint of interconnected members. Some of the examples are (a) A spandrel beam rigidly connected to cross beam Fig. 4.5.2a. (b) As shown in Fig.4.5.2b torsion in L-beam shall be taken into account if stiffness of slab and beam are considered. If slab is considered as simply supported torsion gets eliminated due to release of end restraint.



(a) A spandrel beam rigidly connected to cross beam



(b) L- Beam

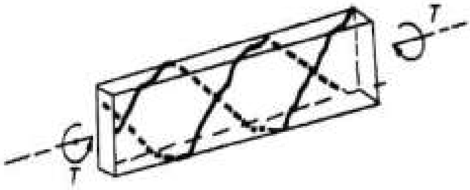
Fig. 4.5.2 Compatibility Torsion

In addition to this interconnected bridge girders, grids in horizontal plane, ring beams are some of the examples of structural members subjected to compatibility torsion.

Torsional cracking.

The torsion produces shear stress, which in turn produces diagonal tension. Because of torsion the beam fails in diagonal tension forming spiral cracks around the beam. *Fig4.5.3* shows the spiral cracking of a plain concrete beam subjected to pure torsion

Fig. 4.5.3 Spiral Cracks due to Torsion



Torsional Reinforcement

Thus, Torsion induced to maintain equilibrium in a structure (called equilibrium torsion) shall be designed for the torsional moment acting on the member. It is converted into equivalent bending moment and equivalent shear which is added to actual bending moment and actual shear acting at the section.. The member is designed to total bending moment and total shear.

4.5.3 Equivalent Bending Moment (M_{ue}) *(clause 41.4.2)*

$$M_{ue1} = M_u + M_t \quad \text{where,} \quad M_t = T \times \frac{(1 + D/b)}{1.7} \qquad \text{......(4.5.1)}$$

T_u = Torsional moment at the section,

M_u = Actual ultimate moment at the section.

When $M_t < M_u$, steel for total tension due to $(M_u + M_t)$ should be provided on bending tension side only

When $M_t > M_u$, steel will be provided on compression face also to resist a B.M. given by :

$$M_{ue2} = M_t - M_u \qquad \text{......(4.5.2)}$$

4.5.4 Equivalent Shear (V_{ue}) *(clause 41.3.1)*

$$V_{ue} = V_u + 1.6\, T_u / b \qquad \text{...(4.5.3)}$$

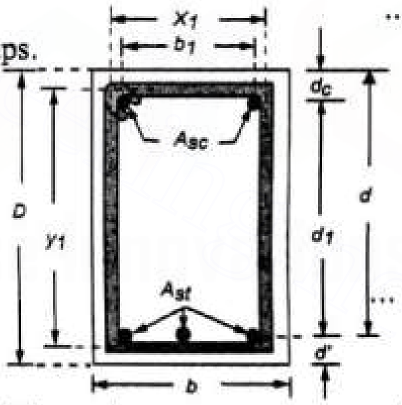
The shear reinforcement consists of only closed vertical stirrups.

It will be designed as follows :

Calculate $V_{ur.min} = V_{uc} + V_{usv.min}$

where, $V_{usv.min} = 0.4\, bd$

If $V_{ue} > V_{ur.min}$ design the shear reinforcement using formula,



$$s = \frac{0.87 f_y\, A_{sv} . d}{V_{us}} \qquad \text{... (4.5.5)}$$

where, $V_{us} = \left(\frac{T_u}{b_1} + \frac{V_u}{2.5}\right) \times \left(\frac{d}{d_1}\right)$ or $(V_{ue} - V_{uc})$ whichever is greater. ...... (4.5.5a)

where b_1 = horizontal distance between the centres of outermost corner bars,

d_1 = vertical distance between the centres of outermost corner bars.

4.5.5 Spacing of Stirrups *(clause 26.5.1.7a)*

$s \nless x_1$ or $(x_1 + y_1)/4$ or 300*mm* or 0.75*d* whichever is the less. ......(4.5.6)

where, x_1 = horizontal distance between centres of vertical legs of stirrups,

y_1 = vertical distance between centres of horizontal legs of stirrups.

4.5.6 Side Face Steel *(clause 26.5.1.7b and clause 26.5.1.3)*

When the cross-sectional dimension of the member exceeds 450*mm* (750*mm* if member is not subjected to torsion), additional longitudinal bars will be provided along the two side faces of web with total area equal to 0.1% of the web area and shall be equally distributed on two faces at a spacing between these longitudinal bars not exceeding b_w or 300*mm* whichever is less.

4.6 BOND

4.6.1 Definition

Bond is defined as interfacial shear acting over the contact surfaces of the bar which prevents relative movement between the two constituent materials namely concrete and steel.

Bond is necessary for transfer of strains and hence forces from concrete to steel and vice - versa so that the two materials act together as one composite material. Bond is due to chemical adhesion (gripping of concrete to bar on setting), mechanical friction and bearing on projections (lugs *or* ribs) on bars as in the case of deformed bars. In absence of bond, the force transfer can be made by mechanical anchorage at the end.

4.6.2 Bond Strength *(clause 26.2.1.1)*

The value of design bond stress for plain bars in tension prescribed by IS Code are given in *Table. 4.6.1*

Table 4.6.1 Design Bond Stress in Limit State method for Plain Bars in Tension

Grade of Concrete	M20	M25	M30	M35	M40
Design bond stress τ_{bd} in N/mm^2	1.2	1.4	1.5	1.7	1.9

Notes : 1. For deformed bars, these values will be increased by 60%.
2. For bars in compression, the above values shall be increased by 25% for all load levels.

4.6.3 Development Length *(clause 26.2.1)*

It is defined as that length (L_d) of bar required to develop a design stress (σ_s) in the bar at the prescribed rate of average bond strength τ_{bd} and is given by :

$$L_d = \{\sigma_s / (4\,\tau_{bd})\}\ \phi = k\phi \qquad \text{... ...(4.6.1)}$$

where, $k = \sigma_s/(4\tau_{bd})$ is known as development length factor.

For developing full strength in the bar, $\sigma_s = 0.87 f_y$ $\therefore k = [(0.87 f_y/(4\tau_{bd})]$ ... ...(4.6.2)

For round bars under tension, the values of development length factor k, are given in *Table 4.6.2.* for different grades of concrete and steel.

Table 4.6.2 : Development Length Factor $k = L_d/\phi = 0.87 f_y / (4\tau_{bd})$ or $L_d = k\,\phi$

Concrete Grade	M20			M25		
Steel Grade	Fe250	Fe415	Fe500	Fe250	Fe415	Fe500
Bars in tension	46	47	57	39	41	49
Bars in compression	37	38	46	31	33	39

Notes : (1) Values are rounded - off on higher side.
(2) Plain round bars are assumed for grade Fe250, and deformed bars for grades Fe415 and Fe500.

4.6.4 Standard End Anchorages - Hooks and Bends *(clause 26.2.2.1)*

A bar gives an additional equivalent bond length of 4ϕ for every 45° bend, subject to a maximum of 16ϕ. In case of bars in compression, hooks and bends are ineffective and cannot be used as anchorage.

4.6.5 Check for Development Length

(a) *Members under Direct Force (Tension or Compression)*

Every bar shall extend a distance equal to development length L_d on each side of a critical section (*i.e.* the total length of the bar, under no circumstances shall be less than $2L_d$).

(b) *Members under Bending* *(clause 26.2.3.4)*

(i) Negative moment reinforcement

At least one-third of the total tension steel provided for negative moment at the support shall extend beyond the point of inflection for a distance not less than d *or* 12 ϕ *or* one-sixteenth of the clear span whichever is greater.

(ii) Positive moment steel (clause 26.2.3.3)

(a) At least one - third the positive moment reinforcement in simple members and one - fourth the positive moment reinforcement in continuous members shall extend along the same face of the member into the support, to a length of $L_d/3$

(b) At simple supports and at points of inflection, positive moment tension steel shall be limited to a diameter such that

$$L_d \not> \frac{M_1}{V} + L_o \qquad \text{... ...(4.6.3a)}$$

and for ends of reinforcement confined by a compressive reaction,

$$L_d \not> \frac{1.3\ M_1}{V} + L_o \qquad \text{... ...(4.6.3b)}$$

where, M_1 = Moment of resistance of beam at the section, assuming all bars stressed to $0.87 f_y$.

When 50 % bars are available at support, M_1 can be approximately taken equal to $M_{max}/2$,

$$or \qquad M_1 = M_u = 0.87 f_y A_{st}\, d \times \left[1 - \frac{f_y A_{st}}{f_{ck}\, bd}\right]$$

L_o = extension of bars beyond the centre line of support and the equivalent anchorage value of any hook. (h_a)

At a point of contraflexure L_o is limited to d *or* 12ϕ whichever is greater. where, ϕ is bar diameter.

Referring to *Fig. 4.6.1*,

Using 90° bend for HYSD bars, $L_o = b_s/2 - x_1 + 3\phi$ ... ...(4.6.4a)

Using 180° bend for Fe250, $L_o = b_s/2 - x_1 + 13\phi$ ... ... (4.6.4.b)

For details see *Sect. 9.4.2* Step No. 10 of Ref [4.4]

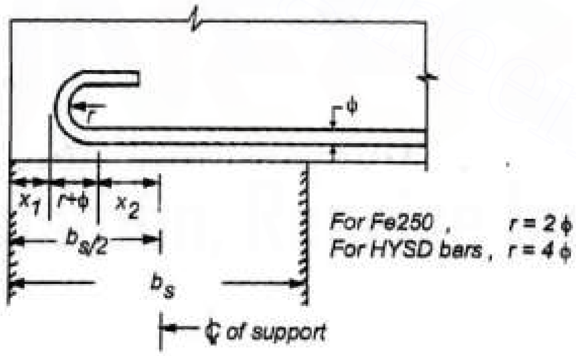


Fig. 4.6.1 Details of Anchorage

4.6.6 Curtailment of Bars *(clause 26.2.3)*

Reinforcement, which is no longer required to resist flexure beyond certain point may be curtailed subject to its extension beyond that point by a distance 12 ϕ *or* '*d*', whichever is greater except at simple support *or* end of cantilever and subject to provision of additional shear reinforcement as explained in *Sect. 4.2.6.*

4.7 SERVICEABILITY (Deflection and Cracking)

4.7.1 Deflection

(a) ***Allowable Deflections*** *(clause 23.2)*

The Code prescribes the following allowable deflections.

Total deflection $\not>$ Span/250

Deflection after the erection of walls, application of finishes $\not>$ Span/350 *or* 20*mm* whichever is less.

These requirements are said to have been met with, if actual *L/d* ratio is less than allowable *L/d* ratios given below.

(b) *Allowable L/d Ratio* (r_a) *(clause 23.2.1)*

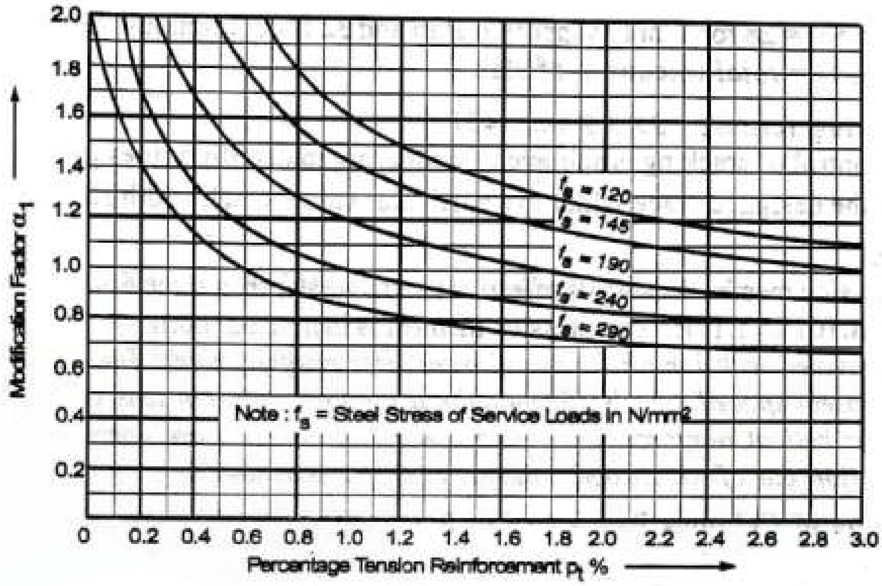
Allowable L/d ratio $= r_a = \alpha_1\,\alpha_2\,\alpha_3\,\alpha_4 \times r_b$ or $d = L\,/\,(\alpha_1\,\alpha_2\,\alpha_3\,\alpha_4 \times \text{Basic } L/d)$ ... ...(4.7.1a)

where, r_b = Basic L/d ratio = 7 for cantilever, = 20 for simply supported, = 26 for continuous.

α_1 = modification factor corresponding to percentage of tension steel p_t and f_s

f_s = steel stress of service load in $N/mm^2 = 0.58\, f_y \dfrac{(A_{st})_{reqd.}}{(A_{st})_{prov.}}$ ... ...(4.7.1b)

α_1 is obtained as per *Fig. 4.7.1*



$f_s = 0.58\, f_y \dfrac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}} = 0.58\, f_y \dfrac{A_{st\,(reqd.)}}{A_{st\,(prov.)}}$

Fig. 4.7.1 Modification Factor for Tension Reinforcement

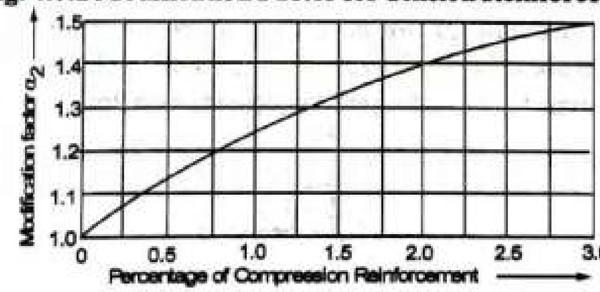


Fig. 4.7.2 Modification Factor α_2 for Compression Reinforcement

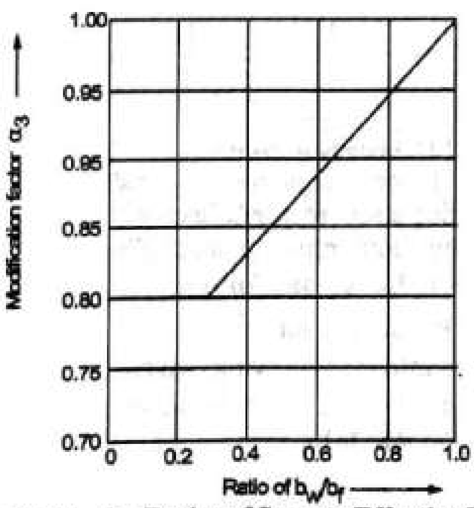


Fig. 4.7.3 Reduction Factor α_3, for Ratios of Span to Effective Depth for Flanged Beams

α_2 = modification factor for percentage of compression steel steel p_c as given by *Fig. 4.7.2.*
α_3 = modification factor for b_w / b_f to be obtained from *Fig. 4.7.3*
α_4 = modification factor for span > 10 *m* = 10/span.

In case of flanged beams, p_t will be based on 100 $A_{st}/(b_f.d)$ and A_{st} shall be at mid-span for beams supported on two supports and at fixed end in case of cantilevers.

In case of slabs, $\alpha_2 = 1$ (as $p_c = 0$), α_3 and α_4 do not apply.

4.7.1b For two - way slabs having span $L_x \leq 3.5\ m$ and $LL \leq 3\ kN/m^2$ and HYSD bars of Fe415 or Fe500, allowable *L/D* ratio r_a = 28 for simply supported slab and 32 for continuous slab.(see clause 24.1 Note - 2)

where, *D* = *total* thickness of slab

4.7.2 Cracking *(clauses 35.3.2 and 43)*

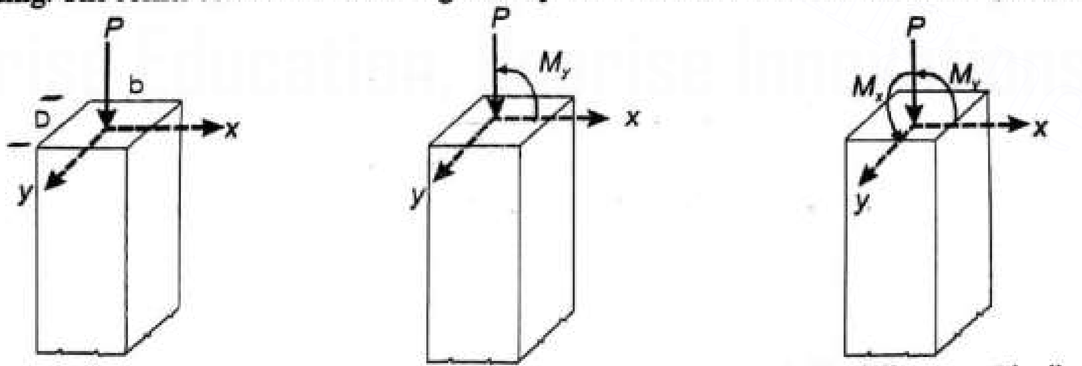
Normally control of cracking is achieved by adhering to detailing rules given in *Chap. 5.* Where it is required to limit the designed crack width to a particular value, crack width shall be calculated. [4.5]

4.8 COLUMN

A compression member having its effective length greater than three times its least lateral dimension is called a **Column.** (Cl.25.1.1) If a compression member is inclined or horizontal it is called **strut.** *The column is provided with longitudinal and transverse reinforcement. The transverse steel is in the form of ties or closely spaced spirals. Longitudinal reinforcement consists of main bars tied with transverse links or helical reinforcement. Tied and spiral columns are normally used in reinforced concrete construction out of which tied columns are very common.*

4.8.1 Classification of Columns Based on Loading

The column is subjected to axial load ,uniaxial bending and biaxial bending as shown in Fig.4.8.1. The column shown in Fig.4.8.1a carry the load along its centroidal axis is termed as axially loaded column. Such ideal column rarely occurs in practice. The columns in buildings are normally subjected to axial compression and bending. If a column carries axial compression and bending moment about either *x-axis* or *y-axis* it is termed as column subjected to axial load and uniaxial bending as shown in Fig.4.8.1b. *The end columns in a buildings are normally subjected to bending about either x or y-axes.* While if it is subjected to moment about both the axes in addition to axial load it is termed as column subjected to biaxial bending. The corner columns in a building are subjected to axial load and biaxial bending (Fig.4.8.1c)



(a) Concentrically loaded column
(b) Uniaxially eccentrically loaded column(about y-axis)
© Biaxially eccentrically loaded column

M_x is cosidered as moment acting about major *'x'* axis of bending, dividing depth *'D'* of the column while, M_y is cosidered as moment acting about minor *'y'* axis of bending, dividing width *'b'* of the column

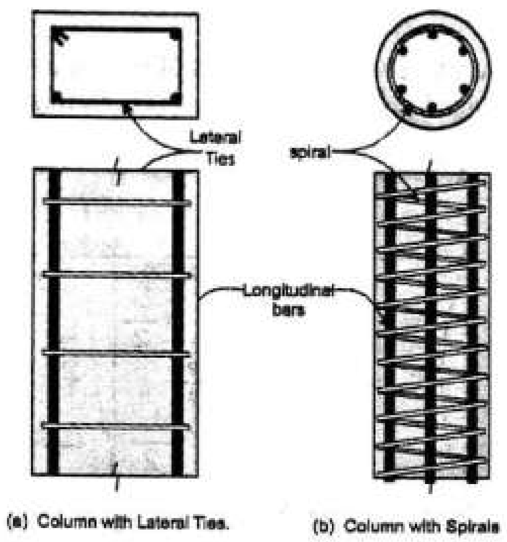
Fig. 4.8.1 Loads on Column

4.8.2 Classification of Column Based on Reinforcement

Concrete columns are mainly classified into the following two types:

(a) Column with Lateral Ties: *In this case the main reinforcing longitudinal bars are enclosed within closely spaced lateral ties (Fig.4.8.2a)*

(b) Column with Spirals: *In this case the main reinforcing longitudinal bars are enclosed within closely spaced and continuously wound spiral reinforcement (Fig.4.8.2b).*



4.8.2 Lateral Reinforcement in Column

4.8.3 Basic Assumptions *(clause 39)*

Limit state of collapse is said to have been reached when the maximum strain in concrete reaches the following values :

ε_{max} = 0.002 for pure axial compression

ε_{max} = 0.0035 for pure bending and for a cracked section under bending and axial compression (*i.e. when* part of the section is in tension)

For uncracked section (*i.e.* no tension on the section) the maximum compressive strain at the highly compressed extreme fibre in concrete under combined bending and axial compression is given by :

$\varepsilon_{max} = 0.0035 - 0.75 \times \varepsilon_{min}$ at highly compressed extreme fibre.

and ε_{min} = minimum compressive strain at least compressed fibre.

4.8.4 Unsupported Length (*L*) *(clause 25.1.3)*

The unsupported length of a compression member is defined as the clear distance between the end restraints. For a beam-slab floor construction, it is equal to floor to floor height minus the total depth of the shallower beam framing into the column at top.

In beam slab construction, it is the clear distance between the floor and the underside of the slab framing into the column in each direction as shown in Fig. 4.8.3

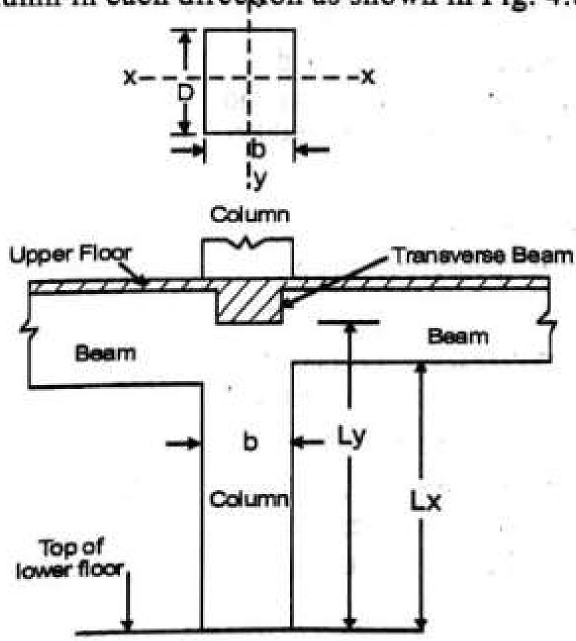


Fig. 4.8.3 Unsupported Length of Column

4.8.5 Effective Length (L_{eff}) *(clause 25.2)*

Effective length is the length of the column between the points of contraflexures of a buckled column. *It is related to end conditions. The theoretical values of effective length along with recommended by Code have been shown in Fig.4.8.4*

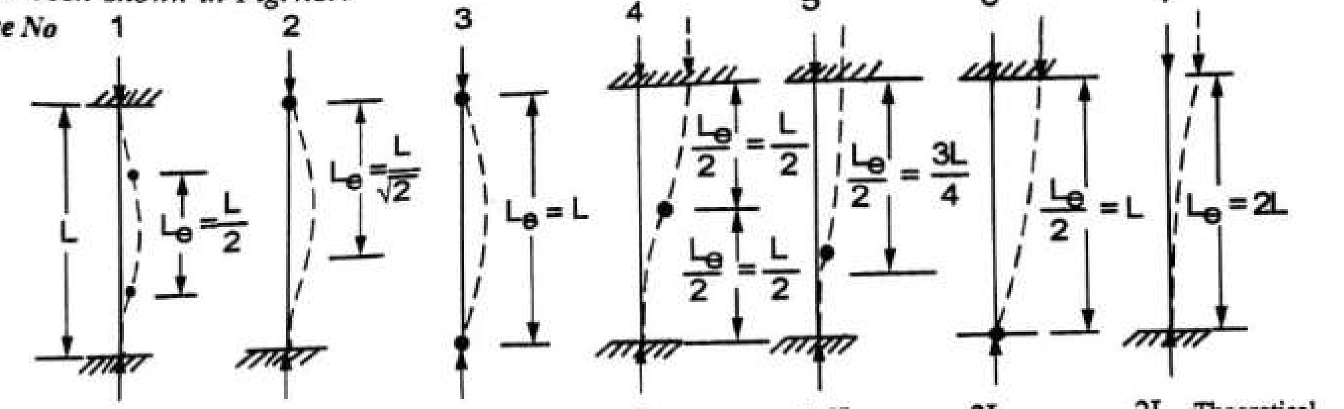


Fig. 4.8.4 Effective Length -- Theoretical and as Recommended by Code

Modified values of effective length in terms of unsupported length given by the Code are given in Table 12.1

Table 4.8.1 Effective Length of Compression Members

Case No.	End condition	Effective Length Theoretical	Effective Length Recommended
1	Effectively held in position and restrained against rotation at both ends	0.5 *L*	0.65 *L*
2	Effectively held in position at both ends and restrained against rotation at only one end	0.7 *L*	0.80 *L*
3	Effectively held in position but not restrained against rotation at both ends	1.0 *L*	1.00 *L*
4	Effectively held in position and restrained against rotation at one end, and at the other end restrained against rotation but not held in position	1.0 *L*	1.20 *L*
5	Effectively held in position and restrained against rotation at one end, and at the other end partially restrained against rotation but not held in position	–	1.50 *L*
6	Effectively held in position at one end but not restrained against rotation and at the other end restrained against rotation but not held in position	2.0 *L*	2.00 *L*
7	Effectively held in position and restrained against rotation at one end but neither held in position nor restrained against rotation at the other end	2.0 *L*	2.00 *L*
	where, *L* = unsupported length of the column.		

(ii) Columns in Frames :

The effective length of a column in a rigid plane frame is obtained by using the ratio L_{eff}/L obtained from *Fig.4.8.4* for columns not free to sway case 1 to 3 and for columns which are free to sway case 4 to 7

The ratio L_{eff}/L depends upon the rotation release factors β_1 and β_2 at top and bottom of column respectively where,

$\beta = \Sigma k_c / (\Sigma k_c + \Sigma k_b)$. ... ...(4.8.1)

$k_c = I/L$ of column and $k_b = I/L$ of beam

I = Moment of inertia of the member,

L = Length of the member between centres of the joints at two ends.

For braced frames[4.6]*,Fig. 4.8.1* $k_b = (1/2) \times I/L$

$$\beta = \frac{\Sigma I_c / L_c}{\Sigma I_c / L_c + \Sigma 0.5\, I_b / L_b} \quad \text{for braced frame} \qquad \text{... ...(4.8.1a)}$$

For unbraced frames[4.6], *Fig. 4.8.2* , $k_b = (3/2) \times (I/L)$

$$\therefore \beta = \frac{\Sigma I_c / L_c}{\Sigma I_c / L_c + \Sigma 1.5\, I_b / L_b} \qquad \text{... ...(4.8.1b)}$$

The value of I/L is based on the assumption that both the ends of the column are held in position as well as restrained against rotation (i.e. fully fixed). For a column with the other end position fixed but rotation free (*i.e.*hinged), $k_c = (0.75)I/L$ has two value β_1 at top and β_2 at bottom of column. For determining β_1 and β_2 the limited substitute frame (*i.e.* substitute frame *III* for columns given in *Sect. 3.2.2*) should be used.

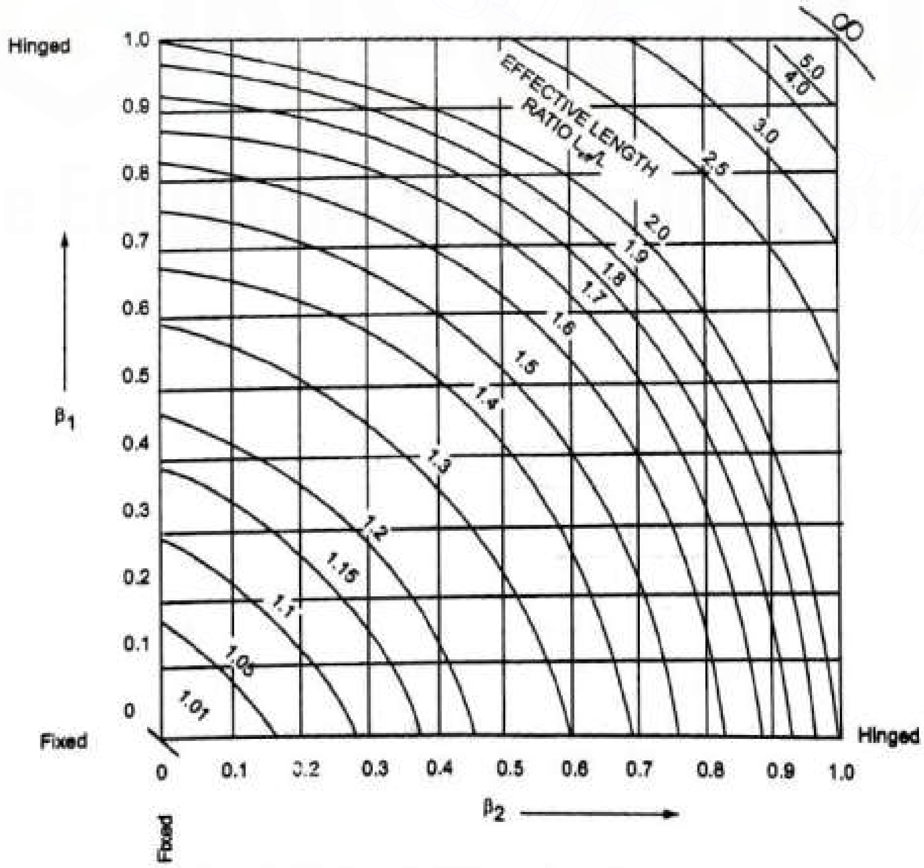


Fig. 4.8.5 Effective Length Ratios of Column in a Frame without Sidesway

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To determine whether a column is a non - sway or sway column, stability index Q may be computed as given below :

$$Q = \frac{\Sigma P_u}{h_s} \times \frac{\Delta_u}{H_u} \qquad \text{... ... (4.8.2)}$$

where, ΣP_u = sum of axial loads on all columns in a storey,

Δ_u = elastically computed first order lateral deflection ,

H_u = total lateral force acting within the storey,

h_s = height of the storey.

In an unbraced frame the storey drift per unit storey shear or lateral flexibility measure of the storey may be taken as [4.7]

$$\frac{\Delta_u}{H_u} = h_s^2 \left(\frac{1}{12\, E_c\, \Sigma I_c/h_s} + \frac{1}{12\, E_c\, \Sigma I_b/L_b} \right) \qquad \text{... ... (4.8.2.a)}$$

where, ΣI_c = sum of the moment of inertia of all columns in a storey in the plane under consideration.

$\Sigma I_b/L_b$ = sum of ratios of moment of inertia to span of all members in the storey and in the plane under considerations.

$E_c = 5000\sqrt{f_{ck}}$ = modulus of elasticity of concrete.

If the column is rectangular the moment of inertia about orthogonal axes will be :

$I_{cx} = 1/12\, bD^3$ and $I_{cy} = 1/12\, Db^3$

If bracing elements such as infill walls or shear walls are provided lateral flexibility will increase and Δ_u / H_u will get reduced.

If $Q \leq 0.04$, the column in the frame may be taken as non - sway otherwise as a sway column

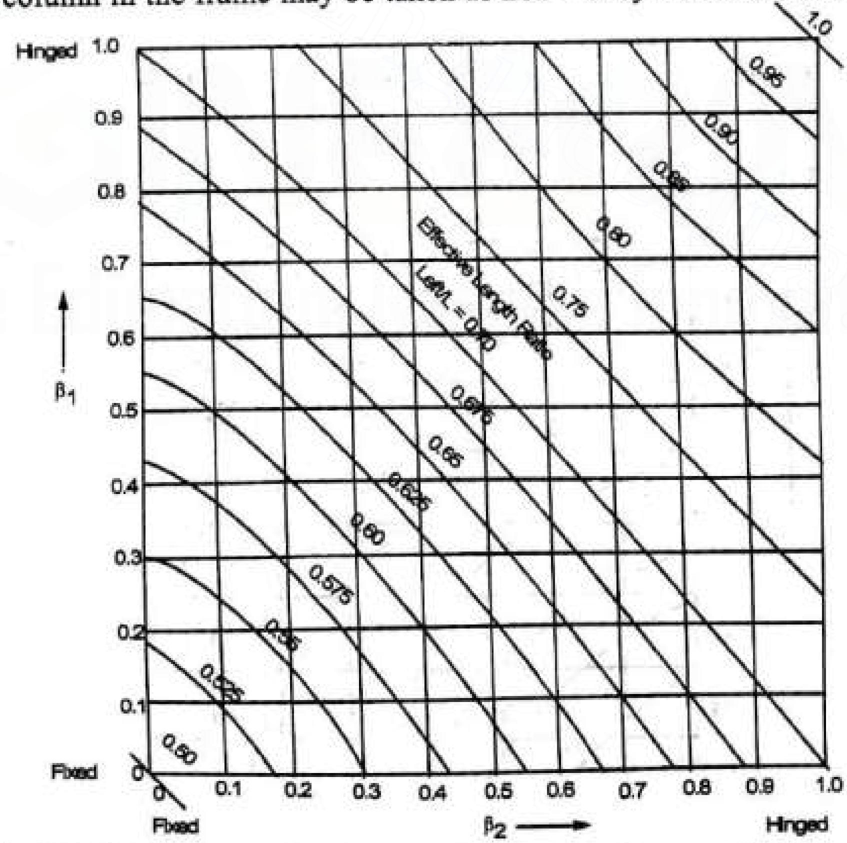

Fig. 4.8.6 Effective Length Ratios for a Column in a Frame *with Sidesway*

4.8.6 Slender Column *(clause 25.1.2)*

Slenderness ratio of the column is defined as the ratio of the effective length of column to its lateral dimension perpendicular to the axis of bending i.e. L_{eff} / h, *(h can be either b or D).*

A column is said to be short when the slenderness ratio is less than 12; else the column is slender or a long column.

If $L_{eff} / h < 12$, column is short otherwise long or slender
where, h = either b or D *i.e.* dimension perpendicular to the axis of bending.
i.e for rectangular section b x D of column to be short,
$L_{effx} / D < 12$, for bending about x-axis *i.e* an axis perpendicular to D
and $L_{effy} / b < 12$, for bending about y-axis *i.e.* an axis perpendicular to b.

4.8.7 Minimum Eccentricity (e_{min}) *(clause 25.4)*

Minimum eccentricity for bending about major axis of bending x-x bisecting the depth *(D)* of the column is given by :

$$e_{minx} = \frac{L_x}{500} + \frac{D}{30} < 20\ mm \qquad \text{...... (4.8.5a)}$$

Minimum eccentricity for bending about minor axis of bending y-y bisecting the width *(b)* of the column is given by :

$$e_{miny} = \frac{L_y}{500} + \frac{b}{30} < 20\ mm \qquad \text{...... (4.8.5b)}$$

Any one of the above two minimum eccentricities shall be taken appropriate to the axis of buckling. Both of them will not be considered to act simultaneously. When buckling is considered about x-axis, e_{minx} will be taken while for buckling about y-axis, e_{miny} will only be taken. When minimum eccentricity requirements control, the provisions are intended to be applied to bending about only one axis at a time and not as a case of biaxial bending.

4.8.8 Axially Loaded Columns *(clause 39.3)*

The ultimate load carrying capacity of an axially loaded column (*i.e.* the ultimate strength of a R.C. member in axial compression) is obtained from the following relations :

(a) *Columns with Lateral Ties*

(i) Ideal axial strength (with zero eccentricity)

$$P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc} \qquad \text{...... (4.8.6a)}$$

$$or \quad P_{uz} = 0.45 f_{ck} A_g + (0.75 f_y - 0.45 f_{ck}) A_{sc}$$

(ii) When minimum eccentricity, $e_{min} \ngtr 0.05$ x lateral dimension, h (*i.e.* $e_{min} \ngtr h/20$)

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc} \qquad \text{......(4.8.7a)}$$

$$or \quad P_u = 0.4 f_{ck} A_g + (0.67 f_y - 0.4 f_{ck}) A_{sc} \qquad \text{...... (4.8.7b)}$$

$$or \quad P_u = [0.4 f_{ck} + (0.67 f_y - 0.4 f_{ck}) \times p_c] \times A_g \qquad \text{...... (4.8.7c)}$$

where, $p_c = A_{sc} / A_g$

(iii) For columns with size < 400mm and e_{min} = 20mm (*i.e.* e_{min} > .05h)

$$P_u = \lambda (0.4 f_{ck} A_c + 0.67 f_y A_{sc}) \qquad \text{......(4.8.8a)}$$

$$or \quad P_u = \lambda [0.4 f_{ck} A_g + (0.67 f_y - 0.4 f_{ck}) A_{sc}] \qquad \text{...... (4.8.8b)}$$

$$or \quad P_u = \lambda [0.4 f_{ck} + (0.67 f_y - 0.4 f_{ck}) p_c] A_g$$

where, $p_c = A_{sc} / A_g$ ......(4.8.8c)

where, A_c = Area of Concrete in compression,
A_{sc} = Area of Steel in Compression,

Values of reduction factor λ are as under :

Concrete	Grade of Steel	Width of column (b) in mm. 200	230	250	300	> 400
		Reduction Factor λ				
M 20	Fe 415	0.80	0.90	0.92	0.96	1.00

(b) *Columns with Helical Ties* *(Clause 39.4)*

P_u = 1.05 x axial strength of columns with lateral ties; provided

$$s \ngtr \frac{\pi D_k A_{sh} f_y}{0.36\,(A_g - A_k)\, f_{ck}} \qquad \dots\dots (4.8.9)$$

where, s = Spacing of helical ties

A_g = gross cross - sectional area,

A_k = Area of concrete core = $\frac{\pi}{4} D^2{}_k - A_{sc}$

D_k = Diameter of concrete core measured to outside of helical steel = D – 2 x clear cover

A_{sh} = Area of helical steel

4.8.9 Eccentrically Loaded Columns - Uniaxial Bending

(a) Neutral Axis lying Outside the Section ($x_u > D$) :

$$P_u = P_{uc} + \sum_{i=1}^{n} P_{usi} = C_1 f_{ck} bD + \sum_{i=1}^{n} A_{si}\,(f_{si} - f_{ci}) \qquad \dots\dots (4.8.10)$$

$$M_u = C_1 f_{ck} bD^2\,(0.5 - C_2) + \sum_{i=1}^{n} A_{si}\,(f_{si} - f_{ci})\, x_i \qquad \dots\dots (4.8.11)$$

where, $C_1 = 0.446\,(1 - C_3/6)$, ... (4.8.10a)

$C_2 = (0.5 - C_3/7)\,/\,(1.0 - C_3/6)$, ... (4.8.10b)

$$C_3 = \frac{8}{7}\left[\frac{4}{7k-3}\right]^2 \qquad \dots\dots (4.8.10c)$$

$k_u = x_u/D$,

i = serial number of *ith* row of reinforcement, i varies from 1 to n

n = total number of rows of reinforcement.

A_{si} = area of steel in *ith* row,

f_{si} = stress in steel in *ith* row, (compression +ve and tension -ve),

f_{ci} = stress in concrete at level of *ith* row corresponding to ε_i,

ε_i = Strain at level of *ith* row

$$= \frac{0.002 \times (x_u - D/2 + x_i)}{(x_u - 3D/7)} \qquad \dots\dots (4.8.10d)$$

x_i = distance of *ith* row from the centroid of the section, positive towards highly compressed edge and negative towards the least compressed edge,

$f_{ci} = 0.446 f_{ck}$ for $\varepsilon_i \geq 0.002$, ... (4.8.11a)

$f_{ci} = 446\,\varepsilon_i\,(1 - 250\,\varepsilon_i)\, f_{ck}$ for $\varepsilon_i < 0.002$, ... (4.8.11b)

For Fe250, $f_{si} = \varepsilon_i E_s$ but $\ngtr 0.87 f_y$,

For Fe415, $f_{si} = \varepsilon_i E_s$ when $\varepsilon_i \leq 0.00144$, and f_{si} shall be obtained from *Table 2.14.2* when $\varepsilon_i > 0.00144$.

(b) Neutral Axis lying Inside the Section ($x_u \leq D$)

Eq.(4.8.10), (4.8.11) above hold good in this case also but the values of C_1, C_2 and ε_i will be taken as under:

$$C_1 = 0.36k_u, \text{ and } C_2 = 0.416k_u \qquad \text{... ... (4.8.10e)}$$

and $$\varepsilon_i = 0.0035\ (x_u - D/2 + x_i) / x_u \qquad \text{... ... (4.8.10f)}$$

Note : *These equations are used for the analysis or design of eccentrically loaded column. However, design or analysis of a column subjected to combined axial load and uniaxial bending becomes very lengthy using trial and error method. The practical alternative is to use $P_u - M_u$ interaction diagrams.*

(c) $P_u - M_u$ Interaction Diagram

A diagram showing allowable ultimate axial load (P_u) for different values of ultimate moment M_u, is known as $P_u - M_u$ interaction diagram.

It helps to determine one quantity when the other is given.

The $P_u - M_u$ interaction diagrams are very useful when they are drawn in non - dimensional form. This is achieved by expressing axial load P_u in non-dimensional form as $P_u/(f_{ck}bD)$ and moment M_u as $M_u/(f_{ck}\, bD^2)$. Such interaction curves are given in *Appendix - G* for different ratios of p/f_{ck} ranging from 0 to 0.26, distribution of reinforcement on two parallel faces *or* on all faces of rectangular section and different values of ratio of effective cover to the total depth of the section. The steps for design of column using equations and charts have been given in *Sect.6.4.9*

4.8.10 Columns under Axial Compression and Biaxial Bending *(clause 39.6)*

The safety of a column subjected to axial compression and biaxial bending can be verified using the following interaction formula.

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \leq 1 \qquad \text{...(4.8.12)}$$

where, M_{ux} = Given bending moment about x- axis bisecting depth of the column,

M_{uy} = Given bending moment about y- axis bisecting width of the column,

M_{ux1} = uniaxial moment of resistance about x-axis for given P_u when $M_{uy} = 0$

M_{uy1} = uniaxial moment of resistance about y-axis for given P_u when $M_{ux} = 0$

α_n is related to the ratio of P_u/P_{uz} and its variation is shown in *Fig.4.8.7*

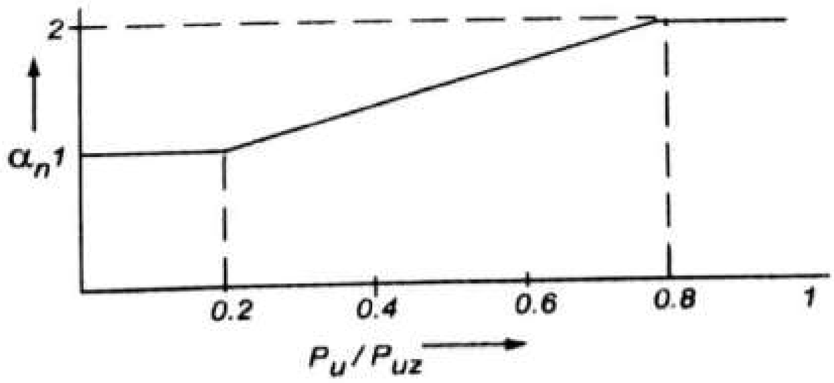

Fig.4.8.7 Variation of α_n with P_u / P_{uz}

4.8.11 Slender Column - Total Moment

In the case of slender columns *i.e.* columns having L_{eff} / Lateral dimension > 12, buckling effect gives rise to additional moments which are required to be taken in additional to the initial moments.

(a) Additional moments due to slenderness

$$M_{ax} = \frac{P_u D}{2000}\left(\frac{L_{effx}}{D}\right)^2 \times k \qquad \text{...... (4.8.13a)}$$

$$M_{ay} = \frac{P_u b}{2000}\left(\frac{L_{effy}}{b}\right)^2 \times k \qquad \text{...... (4.8.13b)}$$

$$k = \frac{(P_{uz} - P_u)}{(P_{uz} - P_{ub})} < 1 \qquad \text{......(4.8.13c)}$$

P_{ub} = axial load corresponding to the condition of maximum compressive strain of 0.0035 in concrete and tensile strain of 0.002 in the outer most layer of tension steel.

or $P_{ub} = (k_1 + k_2\, p/f_{ck}) \times f_{ck}\, bD$ (4.8.13d)

where, p = percentage of steel

k_1, k_2 to be obtained from *Table 4.8.1*

Table 4.8.1 Values of k_1, k_2 for Calculations of P_{ub} for Slender Columns

(a) Values of k_1

Section	d_c/D 0.05	0.10	0.15	0.20
Rectangular Section : $P_{ub} = (k_1 + k_2 p / f_{ck}) f_{ck} bD$	0.219	0.207	0.196	0.184
Circular Section : $P_{ub} = (k_1 + k_2 p / f_{ck}) f_{ck} D^2$	0.172	0.160	0.149	0.138

(b) Values of k_2

Section	f_y in N/mm²	d_c/D 0.05	0.10	0.15	0.20
Rectangular Section - Equal reinforcement on the opposite sides	250	-0.045	-0.045	-0.045	-0.045
	415	0.096	0.082	0.046	-0.022
	500	0.213	0.173	0.104	-0.001
Rectangular Section - Equal reinforcement on the all sides	250	0.215	0.146	0.061	-0.011
	415	0.424	0.328	0.203	0.028
	500	0.545	0.425	0.256	0.040
Circular Section	250	0.193	0.148	0.077	-0.020
	415	0.410	0.323	0.201	0.036
	500	0.543	0.443	0.291	0.056

***Note :** k_1 is a dimensionless coefficient while 100 k_2 gives stress (N/mm²)*

(b) *Initial Moments :*

(i) *For braced column :*

Single curvature, $M_i = 0.6\ M_{u2} + 0.4\ M_{u1}$ but $\nless 0.4\ M_{u2}$ ... ...(4.8.14a)

Double curvature, $M_i = 0.6 M_{u2} - 0.4 M_{u1}$ but $\nless 0.4 M_{u2}$ ... ...(4.8.14b)

where M_{u1} = Smaller end moment, and M_{u2} = Larger end moment.

Note : *If lateral stability to the structure as a whole is provided by walls or bracing then the column may be considered to be a braced column.*

(ii) *For unbraced column :*
For unbraced column the addition moment is added to the end moment or minimum moment whichever is large.

(c) *Total Design Moment :*

M_{uT} = Larger of (Initial Moment *or* end moment) + Additional Moment

$= (M_i + M_a) \nless M_{u2}$ ... ...(4.8.15)

4.9 FOOTING

The design of footing for axial load has been given in *Sect. 6.5*.

For design of eccentric footing and combined footing see Ref.[4.8]

4.10 References

4.1 Shah, V. L. and Karve, S. R., "Limit State Theory and Design of R.C.",Structures Publications, Pune 411009, Seventh Edition 2014,Chapter - 4, Sect. 4.13.

4.2 Kong, F.K., Evens, R. N., " Reinforced and Prestressed Concrete", ELBS, New Delhi, 1987

4.3 Allen, A.H., "Reinforced Concrete Design to BS : 110", E and F.N., Span, 1988

4.4 Shah, V. L. and Karve, S. R., "Limit State Theory and Design of R.C.", Structures Publications, Pune,411009 , Seventh Edition 2014,Chapter - 9, Sect. 9.4.2.

4.5 Shah, V. L. and Karve, S. R., "Limit State Theory and Design of R.C.",Structures Publications, Pune 411009, Seventh Edition 2014 , Chapter - 8, Sect. 8.8.

4.6 SP-24, " Explanatory hand book on IS Code of practice for plain and reinforced concrete", BIS, New Delhi, 1983, *pp* 152

4.7 Pillai, S. U. and Devdas Menon, "Reinforced Concrete Design", Tata-McGraw Hill, 1998

4.8 Shah, V. L. and Karve, S. R., "Limit State Theory and Design of R.C.", Structures Publications, Pune, 411009 , Seventh Edition 2014, Chapter - 12 , Sect. 12.8 and Sect. 12.9..

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CHAPTER - 5

STRUCTURAL BEHAVIOUR OF R.C. ELEMENTS AND DETAILING OF REINFORCEMENT

5.1 GENERAL

In R.C. framed structure the structural elements consist of slab, beam, column and footing. Slab and beam are primarily flexural members while column is mainly subjected to axial compression. The flexural members (*viz.* slab and beam) simply supported at ends and carrying a transverse load bend with concavity upwards producing tension at the bottom. *(Fig. 5.1.1a)*. With the introduction of one or more supports it becomes an indeterminate continuous beam with deflected curve passing over the support with convexity upwards producing tension at top *(Fig. 5.1.1b)*. The effect of continuity is to reduce the sagging *or* positive moment and induce (*or* develop) hogging *or* negative moment at the support. In this process the curvature changes sign from sagging near mid-span to hogging near support. Thus, at some point near the support the curvature becomes zero, where the bending moment is zero. The point where curvature changes sign or becomes zero is called *point of contraflexure* (*i.e contra* means *opposite* and *flexure* means *curvature*) *or* point of inflection. The effect of continuity is to throw more load at the penultimate support (*i.e.* support next to end support) and relieve the simply supported end.

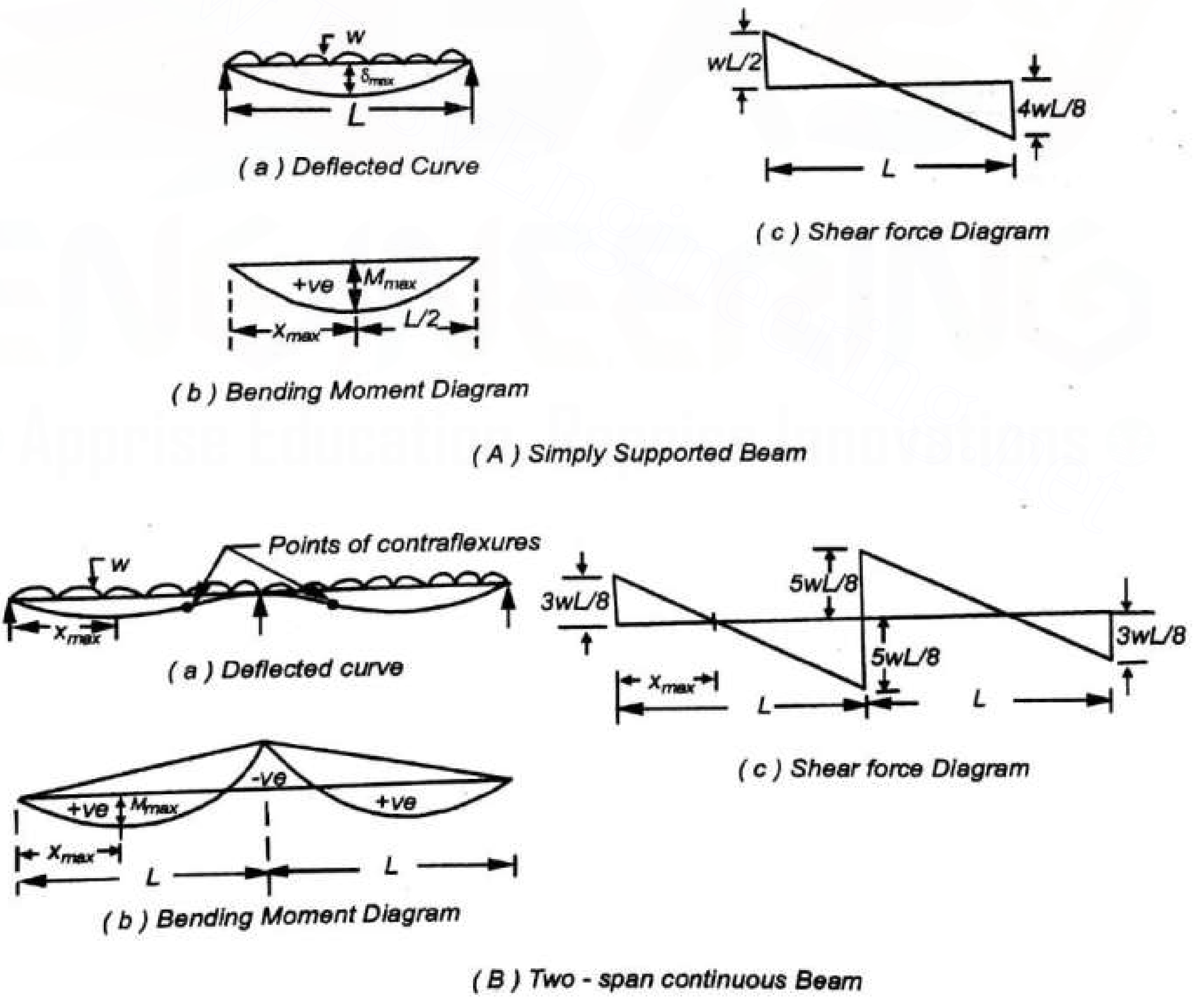

Fig. 5.1.1

When exact analysis is not carried out, for beams of uniform section and slabs supporting substantially uniformly distributed loads over *three or more spans* which do not differ by more than *15% of the longest,* the bending moments and shear forces can be obtained using coefficients given in *Table 5.1.1*

Table 5.1.1 Bending moment and Shear force coefficients for continuous beam/slab with three or more equal spans.

(a) Bending Moment Coefficients :

		End Support		Penultimate Support		Interior Supports →		
DL	α_d	0	+1/12	– 1/10	+1/16	– 1/12	+1/16	-1/12
LL	α_L	0	+1/10	– 1/9	+1/12	– 1/9	+1/12	-1/9

(b) Shear Force Coefficients :

	End Support	Penultimate Support (left)	Penultimate Support (right)	Interior Support (left)	Interior Support (right)	Interior Support
DL	0.4	0.6	0.55	0.5	0.5	0.5
LL	0.45	0.6	0.6	0.6	0.6	0.6

Notes : *(1) DL = Dead load , LL = Live load or imposed load not fixed.*
(2) For obtaining the bending moment, the BM coefficients shall be multiplied by the total design load and span
(3) For obtaining the shear force, the shear force coefficients shall be multiplied by the total design load.
(4) These coefficients are applicable for three or more spans which do not differ by more than 15% of the longest span. In other cases exact analysis should be made.
(5) At supports where two unequal spans meet or where the spans are not equally loaded, the average of the two values for the negative moment at the support may by taken for design.
(6) When coefficients given in the above Table are used for calculation of bending moment redistribution of moments shall not be permitted.

5.2 SLAB

Slabs are broadly categorized mainly into two types namely :

(1) One - way slab *(2) Two - way slab.*

5.2.1 One - way slab and Two - way slab

When a slab is supported only on two opposite parallel edges, it spans only in the direction perpendicular to two supporting edges. It bends in one direction and the deflected surface is primarily of single curvature. The main reinforcement is only provided in the direction of the span to resist one way bending. Such a slab is known as a *one - way slab or* slab spanning in one direction. *Fig. 5.2.1a.*

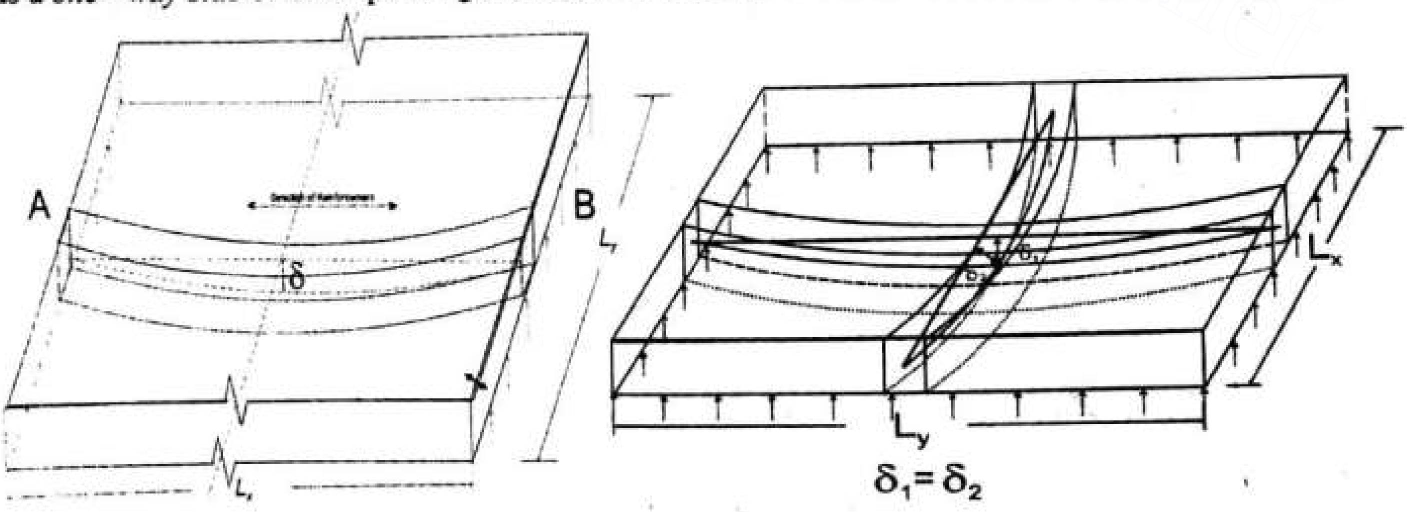

Fig. 5.2.1a. **Fig.5.2.1b**

Fig.5.2.1 Spanning of Slab

A slab supported on four parallel or non - parallel edge supports, which may either be walls or beams, bends in more than one direction. When the supports are orthogonal, the slab spans in two directions at right angles to supporting edges. The deflected surface is of double curvature with deflection in mid-span remaining the same. The load is carried in both directions to the four supporting edges, hence the slab is called a *two - way slab* or slab spanning in two directions. *(Fig. 5.2.1b).* In this slab, main reinforcement runs across both the spans to resist two - way bending. The two - way bending action is not only a function of non - parallel support conditions but also on the ratio of long span L_y to short span L_x. In the case of a rectangular slab supported on all four sides, the two – way bending is predominant only when L_y / L_x is less than 2. When L_y / L_x exceeds 2*, practically entire slab excepting a small portion near short edges spans only across short span and is, therefore, *designed as one-way slab.*

A rectangular slab with ratio $L_y / L_x < 2$ and supported on all sides can also be made to behave predominantly as one way slab (i.e. by making it to bend mainly along the short span) by providing main steel for full one way bending (i.e. to transfer full load across the short span only). However, this solution though favoured in practice to a large extent because of simplicity of design, it may not be economical and further it leads to cracking at top along the shorter edges because of differential deflections between the slab and supporting beam or wall. This is partially prevented by bending 50 % of the bottom distribution steel in long span direction to top of short supports.

5.2.2 Design Requirements

The slab is made to satisfy both deflection and strength requirements. Since the depth of the slab is small it is generally *governed by deflection criteria* rather than the flexural strength of slab. Normally, deflection requirements are satisfied by adhering to effective span to depth ratio. Generally solid slabs are safe against shear and therefore, shear check is generally skipped.

5.2.3 Effective span for slab *or* Beam *(clause 22.2)*

Initially the effective span is computed depending on the supporting conditions.

(a) For simply supported slab *or* beam which is not built integrally with its supports, and for continuous *slab or beam* having breadth of support less than 1/12 of clear span.
Effective span = *L* = (c/c distance between supports *or* clear span + effective depth) whichever is less.

(b) For continuous slab *or* beam having breadth of support greater than 1/12 of clear span *or* 600 *mm* whichever is less, the effective span shall be taken as under :

(i) For end span with one end fixed and the other continuous *or* for intermediate spans :
Effective span = *L* = clear span between supports.

(ii) For end span with one end simply supported and the other continuous,
Effective span = *L* = (clear span + ½ effective depth of slab / beam *or* clear span + half the width of discontinuous support) whichever is less.

(c) For cantilevers :

(i) Effective span = *L* = Length of a cantilever to the face of support + half the effective depth.

(ii) Cantilever at the end of continuous beam :
Effective span = *L* = Length of cantilever to the centre of support.

(d) Continuous Frames :
Effective span = *L* = distance between centers of supports.

In practice the centre to centre distance between the supports is taken as an effective span for simplicity and on the safer side.

5.2.4 Detailing of Reinforcement for Slabs

The steel required to resist the design moment is called *Main reinforcement*, while the steel required to distribute the load properly in the direction in which design moment is taken as zero is called *Distribution steel.* In the case of one-way slab normally main steel is provided along the short span and distribution steel is provided along the long span.

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(i) Minimum Reinforcement and Distribution Steel : (clause 26.5.2.1)

(a) For mild steel (Fe250)

The reinforcement in either direction shall not be less than 0.15% of the total cross - sectional area

$\therefore A_{st.min} = 0.15 \times 1000 \times D/100 = 1.5\,D$ where , D = total depth of slab in *mm*. ... ... (5.2.1a)

(b) For HYSD bars (*i.e.* Fe415 and Fe500)

The reinforcement in either direction shall not be less than 0.12% of total cross - sectional area

$\therefore A_{st.min} = 0.12 \times 1000 \times D/100 = 1.2\,D$ where , D = total depth of slab in *mm* ... ... (5.2.1b)

(ii) Maximum Diameter (clause 26.5.2.2)

Maximum diameter shall not exceed $D/8$ where , D is total thickness of slab in *mm*.

(iii) Spacing of Longitudinal Bar (clause 26.3.3b)

The maximum spacing of longitudinal bars shall not exceed the following values :

Main steel , $s_{max} \leqslant$ 3 d *or* 300 *mm* whichever is smaller ... ... (5.2.2a)

Distribution steel , $s_{max} \leqslant$ 5 d *or* 450 *mm* whichever is smaller ... ... (5.2.2b)

where , d = effective depth of slab

From practical considerations minimum spacing $s_{min} \nless 75$ *mm or* preferably $s_{min} \nless 100$ *mm* ...(5.2.3)

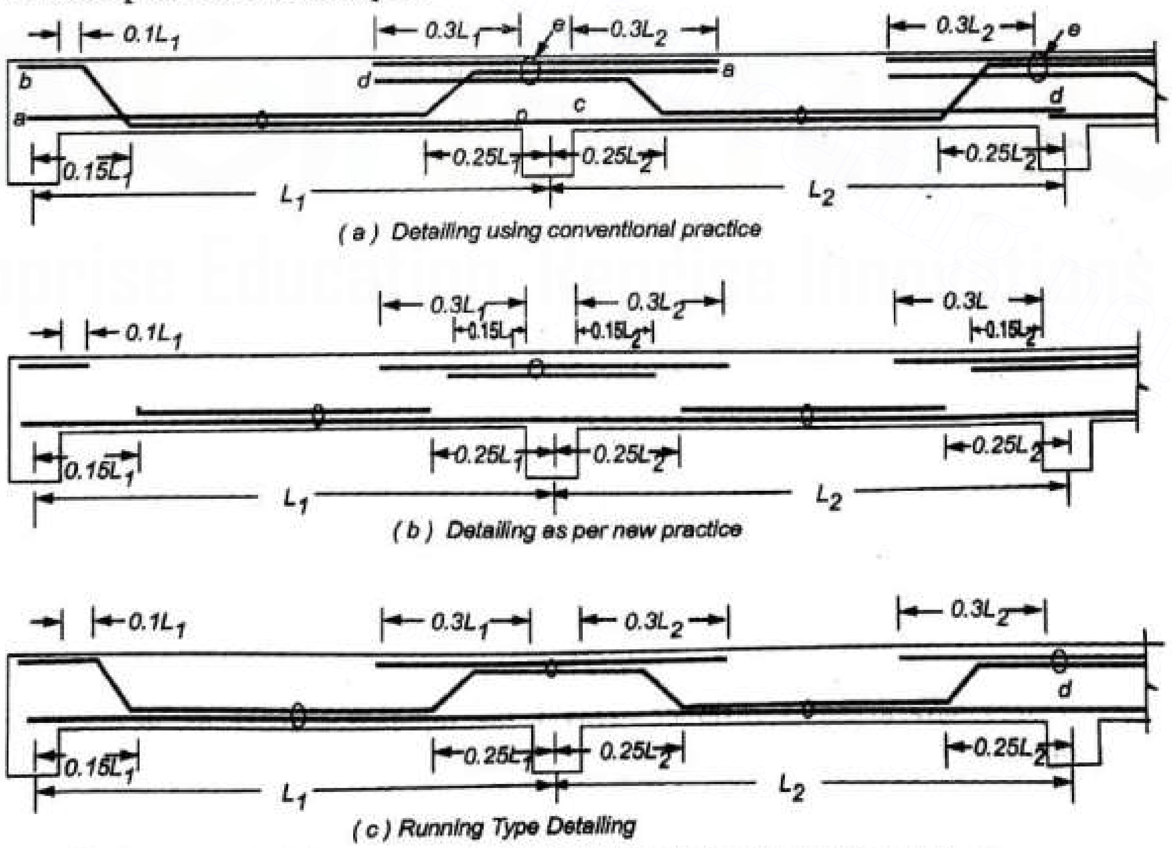
(iv) Cover :

The requirements of Nominal cover based on exposure conditions are given in *Table C-1*.

5.2.5 Different Methods of Detailing for Continuous Slab[5.1]

(i) Detailing using Conventional Practice

Conventional detailing using alternate bent up bar and providing extra steel at support is shown in *Fig.5.2.2a*.Since the diameter of bars required are normally less than 16 *mm* the fabrication of bent up bars is much simpler.



Note : 0 *All bars may be in the same layer. For clarity they have been shown in different layers*

Fig. 5.2.2 Different Methods of Detailing

(ii) Detailing using Soldier's Practice

In this type the required diameter-spacing combination of bars is provided at top face over the support to resist negative moment and independent diameter-spacing *(ϕ - s)* combination of bars is provided at bottom face of mid-span region to resist positive bending moment at shown in *Fig. 5.2.2b*

Even though this case cranking of bars is totally eliminated chairs are required to be provided for supporting top bars during concreting.

(iii) Running Type Detailing

In this case each bar runs from one end of support to the other end of support as a continuous bar. The alternate bars are bent up at the supports, and the remaining half number of bars run as continuous bars at the bottom *(Fig. 5.2.2c)*. In this case the bottom bars are designed for maximum positive moment at mid-span of end span and extra bars are provided at top of support to meet requirements of steel to resist negative moment.

Since the $\phi - s$ combination of bars is decided based on maximum moment at mid-span of end span, this reinforcement will be in excess of requirements at mid - span of interior spans. Therefore, the quantity of steel required will be more than the other cases.

(iv) Very Conservative Detailing

In this case the diameter - spacing of bars is calculated for the maximum moment at penultimate support and the same combination is provided at all mid - spans and top of interior supports.

This type of detailing is very uneconomical. The only advantage is that the fabrication and placement is much simpler and very less supervision is required.

5.3 BEAM

5.3.1 Behaviour of Beam

As mentioned earlier beams are also flexural members and their behaviour is similar to that of slabs. But the bending moments and shearing stresses are much greater than those of slabs. Therefore, the *depth* of the beam is *governed by bending moment criteria* while deflection criteria normally gets satisfied.

A beam simply supported at its ends carrying a uniformly distributed load bends with concavity upwards as shown in *Fig.5.1.1A*. It is subjected to maximum sagging *or* positive bending moment at its mid - span and zero at its supports.

Consider a two span continuous beam simply supported at its ends and carrying a uniformly distributed load as shown in *Fig.5.1.1B*. The bending moment is zero at its simply supported ends. The sagging moment goes on increasing and is maximum not at the mid-span but at a small distance towards the simply supported end *(See Eq .2.6.1)*. After this it goes on reducing becomes zero and the curvature starts changing from sagging to hogging (*i.e.* convexity upwards) reaching maximum value at the support. The point where the curvature changes from sagging to hogging is called *point of contraflexure*. From this point the bending moment reverses and maximum negative *or* hogging moment producing tension at top and compression at bottom occurs at the support. As mentioned earlier the effect of continuity is to throw more load at penultimate support (*i.e. support* next to end support) and relieve the load at simply supported end.

In general practice the beam and the slab are cast together with main steel of slab going inside the beam section, and beam bars and stirrups extend into the slab. In such a case part of the slab (= b_f) acts along with the beam in resisting compressive forces *provided slab lies in the compression zone with respect to bending of the beam*. The resulting section is called *a flange section*.

When the slab, and hence the flange occurs on both sides of the beam, as in the case of an intermediate floor beam, the resulting section resembles a *T* - shape and hence called "*Tee - beam*". On the other hand if the slab is only on one side of the beam (*eg*. end beam of the floor), it appears like an inverted '*L*' and hence the name "*Ell - beam*". In slab - beam construction, a continuous slab is designed assuming it to be simply supported over the end beams. In such a case, torsion gets eliminated due to release of end restraints. Hence end beams of the slab - floor system can be designed as *L* - beam for flexure and shear only.

At the *support* of a continuous beam, in slab-beam floor system, the *slab lies* in the *tension zone* and hence cannot assist the beam in resisting compression and hence the beam should be designed as a *rectangular section* only either singly reinforced (for $M_u < M_{ur.max}$) *or* doubly reinforced (for $M_u > M_{ur.max}$).

As mentioned in *Sect. 4.3.2* at mid-span the following three cases arise depending on the depth of the flange D_f in relation to the depth of neutral axis (x_u)

Case - I : $x_u \leq D_f$,
Case - II a : $x_u > D_f$ but $3x_u/7 < D_f$ and
Case - II b : $x_u > D_f$ but $3x_u/7 \geq D_f$

Even though the equations for *Case - IIa and Case - IIb* are too much involved these conditions generally do not occur in practice. This is because the minimum thickness of the slab is 100 *mm* and depth of the beam is between *L/10* to *L/16*.

The occurrence of *Case - I* is checked by obtaining moments of resistance of the flanged section having $x_u = D_f$ using *Eq.4.3.8. When* $M_u < (M_{ur1})$ *for* $x_u = D_f$, the neutral axis lies inside the flange the beam should be designed as a flanged section and the area of steel can be obtained replacing b by b_f as per *Eq. 4.3.5*. It may be noted that it is always economical to design a flange section instead of rectangular one. The required quantity of steel works out to be less by about 8% to 20% than that required for a rectangular section depending on the position of N.A.

It may further be noted that at mid-span, when $M_u > M_{ur.max}$ then beam *must* be designed as flanged section only and not as a rectangular section, because in such a case rectangular section becomes a doubly reinforced section. [see Sect.8.16 Explanatory Note (3)]

5.3.2 Calculation of Loads on Beam

The load on the beam comprises of :

(1) End shear from slab ,
(2) Load due to wall carried by the beam ,
(3) Self weight of the beam ,
(4) Concentrated load transferred by secondary beam resting on main beam, if any.

(1) Load from slab

The load on the beam from slab = sum of shears from adjacent slabs assuming slab to be simply supported x continuity factor (c)

The continuity factor or shear coefficient is obtained from *Table 5.1.1b*.

The transfer of load from slab depends on the type of slab *viz*. whether it is a one-way slab *or* a cantilever slab *or* a two-way slab.

(a) Load on beam supporting One-way slab and cantilever slab.

(i) For beam supporting one-way slab

In the case of one-way slab the load is primarily transferred to the beam in the direction (mainly along short direction) along which the main reinforcement is provided.

Fig. 5.3.1 shows the rectangular area of the slab transferring the load to the intermediate and end beams.

Load transferred from slab = $w\,(L_{x1}/2 + L_{x2}/2)$ kN/m = wL_x kN/m (if $L_{x1} = L_{x2}$) ... ...(5.3.1)

For beams supporting continuous slab, continuity factors given in *Table 5.1.1b* are applied for accurate analysis.

Load on beam = Sum of shears from adjacent slabs (assuming slabs as simply supported x continuity factor.

In the process of transfer of load some small part of the load will be transferred to short beam *B3* *(See Sect. 1.3.3)*. This load may be considered to be triangular load with central ordinate of $L_x/4$ (see *Fig. 5.3.1c*) giving equivalent UDL on beam = $w_{eq} = wL_x/6$ ... ...*Eq 5.3.1a*

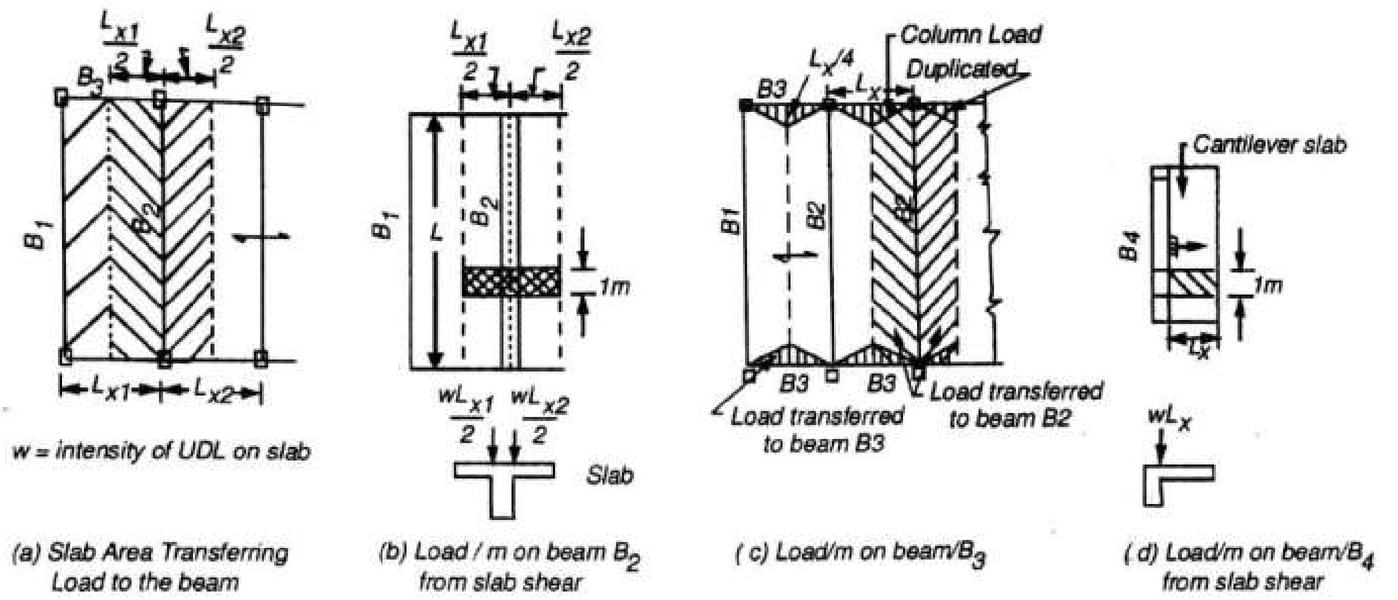


Fig. 5.3.1 Load Distribution in One-way Slab

The load on the long beam $B2$ is, however, taken equal to wL_x. The equivalent load $wL_x/6$ shall not be taken on column to avoid its duplication. (Duplicated load shown hatched in *Fig. 5.3.1c*)

(ii) For beam supporting cantilever slab (Fig 5.3.1c)

Load on beam/m = $w \times L_x$ ... ... (5.3.2)

In addition to this the beam is subjected to torsion of $wL_x^2/2$

For obtaining ultimate load on beam multiply w by load factor 1.5

(b) Load on beam supporting two - way slab

In the case of beams supporting solid slabs spanning in two orthogonal directions carrying uniformly distributed load, the load distribution is *trapezoidal* on long beams and *triangular* on short beams with base angle of 45°as shown in *Fig. 5.3.2.*

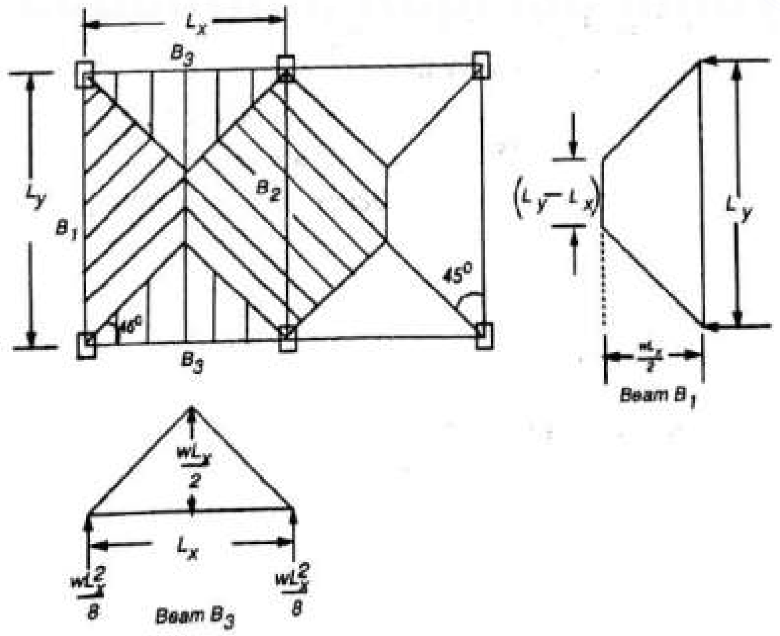


Fig. 5.3.2 Load Distribution in Two-way Slab

These trapezoidal and triangular loads for *two-way slab* are converted into *equivalent UDL* by using equivalence factors given by :

Short span beam (triangular load):

Equivalent *UDL* for bending moment $= w_{eqb} = \dfrac{w\,L_x}{3} = \dfrac{w\,L_x}{2} \times \dfrac{2}{3} = k_1 \times \dfrac{w\,L_x}{2}$ (5.3.3)

where , k_1 = equivalent factor for BM = 2/3

Equivalent *UDL* for shear $w_{eqs} = \dfrac{w\,L_x}{4} = \dfrac{w\,L_x}{2} \times \dfrac{1}{2} = k_2 \times \dfrac{w\,L_x}{2}$ ... ... (5.3.4)

where, k_2 = equivalent factor for shear= 1/2

Long span beam (trapezoidal load) :

$$w_{eqb} = \frac{w\,L_x}{2}\left(1 - \frac{1}{3\beta^2}\right) = k_1 \times \frac{w\,L_x}{2} \quad , \quad \text{where, } k_1 = \left(1 - \frac{1}{3\beta^2}\right) \qquad \text{... ...(5.3.5)}$$

$$w_{eqs} = \frac{w\,L_x}{2}\left(1 - \frac{1}{2\beta}\right) = k_2 \times \frac{w\,L_x}{2} \quad , \quad \text{where, } k_2 = \left(1 - \frac{1}{2\beta}\right) \qquad \text{... ...(5.3.6)}$$

where, w = intensity of loading on slab

$w\,L_x/2$ = rate of loading

If the slab is continuous over the beams continuity factors given in Table 5.1.1b. are applied for accurate analysis.

For obtaining ultimate load multiply the equivalent UDL by load factor of 1.5

(2) Wall load on beam

Load on beam due to wall = $\gamma \times t_w \times H_w$ *kN/m* ... ...(5.3.7)

where, γ = unit weight of masonry in kN/m^3,

t_w = thickness of wall in *m* ,

H_w = height of wall in *m*

The plinth beam normally carries wall load only.

(3) Self weight of beam

In slab-beam system the load due to slab is already taken into account therefore, the self weight of the beam will be due to its *rib portion* only.

Self weight of beam w_s = 25 x Depth of beam below slab × width of rib

$\therefore\ w_s = 25\,(D - D_f)\,b_w$ *kN/m* ... ...(5.3.8)

where , D = total depth of the beam, D_f = Depth of slab

b_w = width of rib.

(4) Load from secondary beam

Load on the main beam = concentrated load due to end reaction of secondary beam.

5.3.3 Detailing of Reinforcement for Beam

(i) Minimum tension reinforcement (clause 26.5.1.1a)

$\dfrac{A_{st}}{bd} \nless \dfrac{0.85}{f_y}$ or $p_t\% \nless \dfrac{85}{f_y}\%$ ...(5.3.9)

for Fe 250	Fe 415	Fe500
0.34%	0.2%	0.17%

(ii) Maximum reinforcement (clause 26.5.1.1b)

Maximum percentage of tension steel (p_t%) or compression steel (p_c%) ≯ 4%

$\therefore p_t\%$ or $p_c\% >$ 4% ... ...(5.3.10)

where , $p_t = 100\, A_{st}/(bD)$ and $p_c = 100\, A_{sc}/(bD)$

(iii) Side face reinforcement (clause 26.5.1.3)

Side face reinforcement when depth of beam exceeds 750*mm or* 450*mm* for beam subjected to torsion ≮ 0.1% of web area (*i.e.* $0.1 \times D \times b_w / 100$)

This should be distributed equally on both faces at a spacing not exceeding 300 *mm or* web thickness b_w whichever is less.

(iv) Minimum shear Reinforcement (clause 26.5.1.6)

$$\frac{A_{sv}}{bs} \geq \frac{0.4}{0.87 f_y} \quad or \quad s \leq \frac{0.87 f_y A_{sv}}{0.4b} \quad \text{... ...(5.3.11)}$$

Spacing of stirrups s shall not exceed $0.75d$ *or* 300 *mm* whichever is less.

where, s = stirrup spacing ,

A_{sv} = total cross - sectional area of stirrup legs (*i.e.* vertical legs) effective in shear,

b = width of beam *(b)* or breadth of web (b_w) of flanged beam ,

f_y = characteristic strength of stirrup reinforcement ≯ 415 N/mm^2.

(v) Transverse reinforcement for shear

$$s = 0.87 f_y A_{sv}\, d / V_{us} \quad \text{... ... (Eq. 4.5.5)}$$

For beams subjected to Torsion ·

$$V_{us} = \left(\frac{T_u}{b_1} + \frac{V_u}{2.5}\right) \times \frac{d}{d_1} \quad or \quad (V_{ue} - V_{uc}) \text{ whichever is greater} \quad \text{... ..(Eq. 4.5.5a)}$$

where, T_u = torsional moment , V_u = ultimate shear force ,

b_1 = c/c distance between corner bars in the direction of width ,

d_1 = c/c distance between corner bars in the direction of depth ,

V_{ue} = equivalent shear = $V_u + 1.6\, T_u / b$, V_{uc} = shear carried by concrete.

s < [x_1 *or* $(x_1 + y_1)/4$ *or* $0.75d$ *or* 300 *mm*] whichever is less

where, x_1 and y_1 are shorter and longer dimensions of stirrups respectively. ... ... (5.3.13)

(vi) Spacing of Main Steel (clause 26.3.2)

(i) Horizontal clear distance :

The horizontal clear distance between two parallel main reinforcing bars *shall not be less than the greatest of the following :*

(a) diameter of bar, (b) The diameter of larger bar if the diameters are unequal,

(c) 5mm more than the nominal maximum size of aggregate i.e. 25 mm for maximum size of aggregate of 20 mm which is normally used.

(ii) Vertical clear distance : ... ... (5.3.14)

When there are two *or* more rows of bars, the bars shall be in vertical line, and the minimum clear distance between two rows shall be *greatest of the following* :

(a) 15 mm (b) 2/3 of nominal maximum size of aggregate (c) maximum bar diameter

(vii) Positive Moment Reinforcement *(clause 26.2.3.3)* :At least 1/3 of positive moment reinforcement in simple members and 1/4 the positive moment reinforcement shall extend along the same face of the member into the support, to a length equal to $L_d / 3$

(viii) Negative Moment Reinforcement *(clause 26.2.3.4)* : At least 1/3 of the total reinforcement at the support shall extend beyond the point of inflection for a distance $\nless$ (d or 12ϕ or clear span /16) whichever is greater.

5.4 COLUMN

5.4.1 Behaviour of Column

The columns in the structure are mainly subjected to axial compression (Fig.4.8.1a)., The columns carrying axial compression only are the internal columns with beams in all four directions *or* beams in one plane having same spans and same loading.

Some columns in the structure are subjected to bending due to rigid connections with the beam *or* due to eccentric loading (Fig.4.8.1b). The side columns on the external wall carrying beams in three mutually perpendicular directions *or* a single beam in one direction are subjected to axial compression and uniaxial bending.

The corner column in the structure are required to be designed for bi - axial bending and axial compression (Fig.4.8.1c) (for further details see *Sect. 6.4.3*)

Even if the beam is assumed to be simply supported at its ends, the column will be subjected to bending moment due to partial fixity. The moment on the column which may either be due to rigid connection with the beam *or* due to eccentric loading is called *Initial moment.*

The load carrying capacity of the column also depends on the slenderness ratio (L_{eff}/h) which is the ratio of effective length L_{eff} to the corresponding lateral dimension (h). If the slenderness ratio is greater than *or* equal to12 it is called a *long or slender* column (For details see *Sect 4.8.9*). The long column buckles prior to reaching its ultimate strength. The effect of slenderness is to cause additional moment M_a and consequently reduce the load carrying capacity of the column.

5.4.2 Loads on Column

(a) Exact Method *(Fig. 5.4.1a)*

As mentioned earlier the actual load on column can best be obtained by calculating first the loads on beams and their end shears. The total load acting on any column is the algebraic sum of the shear at the end of all beams meeting at the column, plus the axial load coming from the upper column and its self weight.

Thus, at any floor level, $P = V_1 + V_2 + V_3 + V_4 + P_a + P_{self}$ ... ... (5.4.1)

where, P_a = load coming from upper floors

V_1, V_2, V_3, V_4 = end shear of beams

P_{self} = self weight of column.

(b) Approximate Method :

This method may be used when design of footing is required to be given in a short time before the detailed design could be carried out.

For this following procedure may be followed :

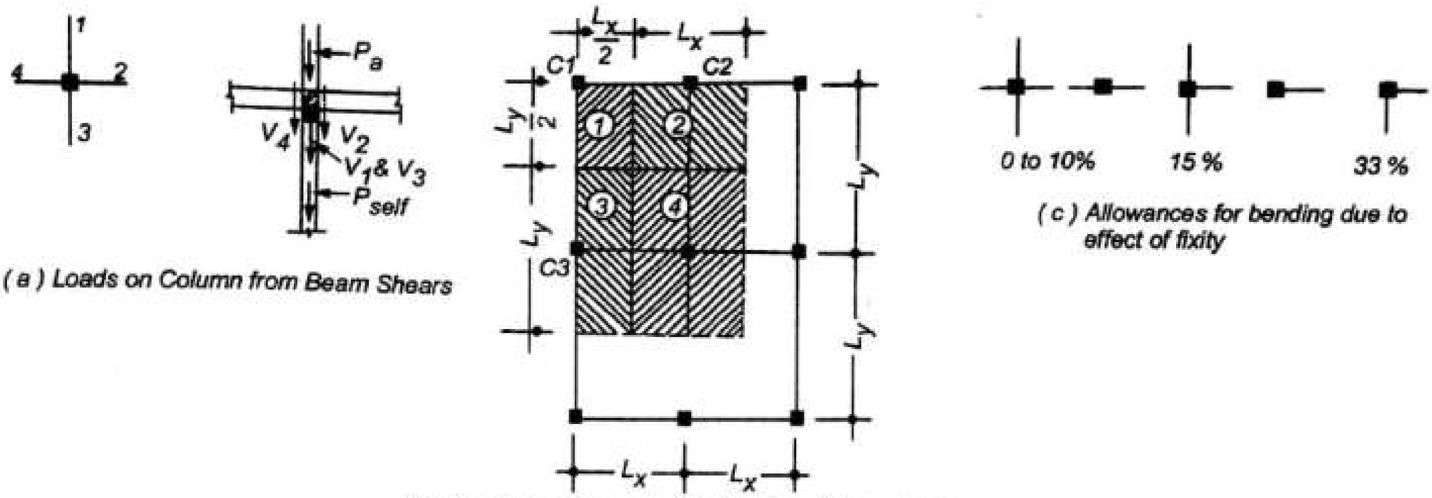
1) Mark the floor area (A_{col}) contributing the load to the column. The load transfer area (A_{col}) is the area contained between the intersecting perpendicular lines drawn from the midpoints of the lines joining the adjacent column *(see Fig. 5.4.1b).*

2) Calculate the depth of the slab and hence the intensity of floor load.

w_{us} = 1.5 (25 D + FF + LL) kN/m^2 where , D = Depth of slab in m

$\therefore$

Load transferred from slab to column = $P_{us} = w_{us} \times A_{col}$ kN ... ...(5.4.2a)



(a) Loads on Column from Beam Shears

(b) Loads on Column by Distribution of Floor Areas

(c) Allowances for bending due to effect of fixity

Fig. 5.4.1 Loads on Column

3) Calculate weight of the wall per unit length using *Eq.5.3.7*

$$w_{uw} = 1.5\ (\gamma \times t_w \times H_w)\ kN/m$$

Determine the length of walls within and on the outer boundary of the load transfer area. The wall on the common boundary of two column load areas will be equally divided to the respective columns.

∴

Wall load transferred to column at each floor level $P_{uw} = w_{uw} \times L_w$ ... ... (5.4.2b)

where , L_w = Sum of the lengths of wall within and on the load transferred area.

∴ Total load transferred to column at any floor is given by :

$$P_{u\ ,floor} = P_{us} + P_{uw} + P_a \qquad \text{... ... (5.4.2c)}$$

where, P_a = load on column from above.

Since the length of the wall is taken as centre to centre distance between the columns, the self weight of column is not added separately. Obtain equivalent axial load : for corner column multiply by 1.33, for side column by 1.15 and for interior column by 1.1.

When the secondary beam rests on main beam, the concentrated load transferred by secondary beam to main beam is not totally added to the nearest column but it is transferred to the columns supporting main beam in proportion to the distances of the point loads from the columns.

Remarks *:The above procedure of calculation of column loads on the basis of column load area may not work in plans having irregular layout of columns. This normally occurs in the case of residential buildings. In such cases loads on the beam are calculated assuming them to be simply supported over columns. The beam shears to be transferred to columns are then calculated using appropriate continuity factors. This is illustrated in Project - III.*

5.4.3 Detailing of Reinforcement for Column

(A) Longitudinal Reinforcement for Columns *(clause 26.5.3.1)*

(a) The cross - sectional area of longitudinal reinforcement :

$\nless$ 0.8% of gross cross - sectional area of concrete,

$\ngtr$ 6 % of gross cross-sectional area of concrete

preferably $\ngtr$ 4 %

However, to avoid congestion of bars, especially when the bars are to be lapped percentage of steel shall not exceed 4%.

In any column that has a cross - sectional area greater than that required to support the load the minimum percentage of steel shall be based on the area of concrete required to resist the direct stress and not on the actual area.

Comments :

The lower limit of 0.8% of steel has been kept on account of the following reasons :

1. *Shrinkage of concrete effects in redistribution of the load from concrete to steel reinforcement, therefore, unless a lower limit is placed on steel ratio even under service load, the steel may reach the yield stress.*
2. *The lower limit protects columns in structural frames against failure in tension when, for example, the surrounding floors near the column are unloaded above but heavily loaded below or when the structural frame is subjected to uneven settlement.*

(b) *Minimum number of longitudinal bars (clause 26.5.3.1)*

= 4 for rectangular columns

= 6 for circular columns

= No. of corners in polygonal or other shapes.

(c) *Diameter of longitudinal steel* ≮ 12 *mm*

(d) For longitudinal reinforcing bar *nominal cover* ≮ 40 *mm* or less than the diameter of such bar. For concrete grade M30 and above the nominal cover shall be as specified in *Table C-1*

(e) Spacing of longitudinal bars measured along the periphery of the column ≯ 300 *mm*

(f) In columns where longitudinal bars are offset at a splice (at the junction of two columns) the slope of the inclined portion of the bar with the axis of the column shall be 1 in 6 maximum, and the portion of the bar above and below the offsets shall be parallel to the axis of the column and provided with adequate lateral ties excepting in the sloping part as shown in *Fig. 5.5.5.*

Where column faces are offset 75 *mm* or more, the longitudinal bars in the column below should be terminated at the floor slab and separate dowels used as shown in *Fig. 5.5.6.*

B] Transverses Reinforcement *(clause 26.5.3.2)*

(a) General : A reinforcement concrete compression member shall have transverse or helical reinforcement so placed that every longitudinal bar nearest to the compression face has effective lateral support against buckling subject to provision of arrangement of bars given in *(Fig.5.4.2).* The effective lateral support is given by transverse reinforcement either in the form of circular rings capable of taking up circumferential tension *or* by polygonal links (lateral ties) with internal angle not exceeding 135° *(Fig. 5.4.2a).* The ends of such transverse reinforcement shall be properly anchored.

(b) Arrangement of Transverse Reinforcement

(i) If the longitudinal bars spaced at a distance not more than 75 *mm* on either side, transverse reinforcement need only to go round corner and alternate bars for providing effective lateral support *(Fig. 5.4.2b).* Any single longitudinal bar may remain untied transversely at right angles only if it is spaced at distance < 75 *mm* from the adjacent tied bar.

(ii) If the longitudinal bars spaced at a distance of not exceeding 48 times the diameter of the tie are effectively tied in two directions, additional longitudinal bars in between these bars need to be tied in one direction by open ties as shown in *Fig. 5.4.2c*

(iii) Where the longitudinal reinforcing bars in compression member are placed in more than one row, effective lateral support to the longitudinal bars in the inner rows may be assumed to have been provided if:

Transverse reinforcement is provided for the outermost row as started above and no bar of the inner row is closer to the nearest compression face than three times the diameter of the largest bar in the inner row *(Fig. 5.4.2d)*

(iv) Where the longitudinal bars in a compression member are grouped (not in contact) and each group adequately tied with transverse reinforcement in accordance with *Sect. 5.4.3(B),* the transverse reinforcement for the compression member as a whole may be provided on the assumption that each group is a single longitudinal bar for purpose of determining the pitch and diameter of the transverse reinforcement in accordance with *Sect. 5.4.3(B).*The diameter of such transverse reinforcement need not, however, exceed 20 *mm* (see *Fig. 5.4.2e.*)

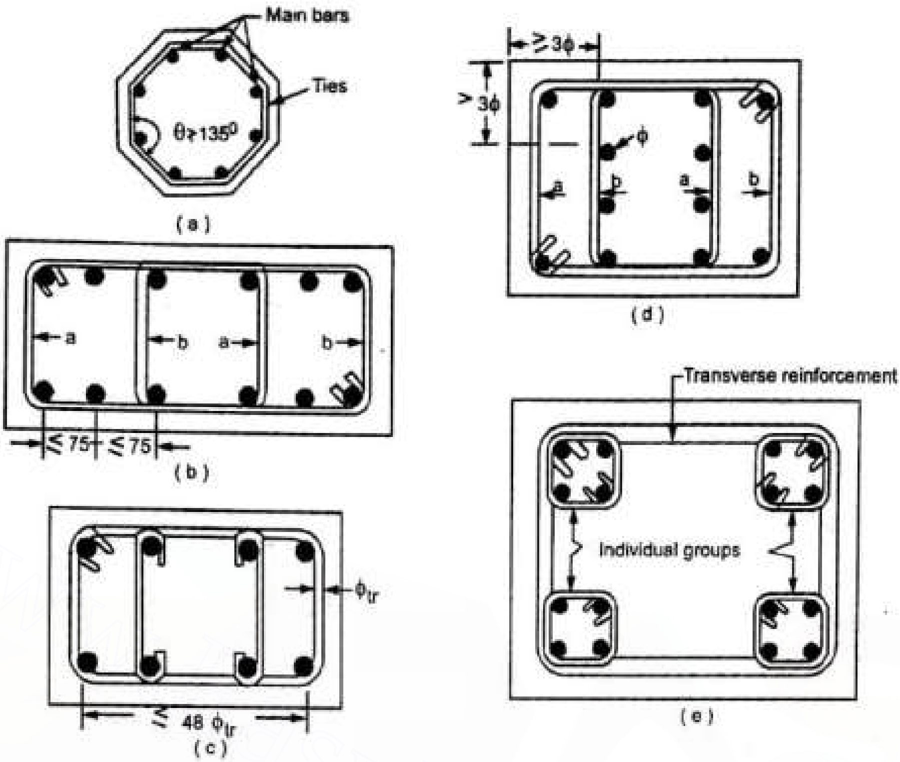


Fig. 5.4.2 Arrangement of Transverse Reinforcement

(c) Pitch and diameter of lateral ties (ϕ_{tr}) *(clause 26.5.3.2c)*

(i) Diameter : The diameter of the polygonal links or lateral ties shall be not less than *one-forth* of the diameter of the largest longitudinal bar, and in no case less than 5 *mm.*

$(\phi_{tr}) \nless (\phi / 4 \text{ or } 5\ mm)$ whichever is greater

(ii) Pitch (s) : The pitch of the lateral ties shall *not be more than the least of the following:*

- The least lateral dimension *(b)* of the compression member ;
- Sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied ;
- 300 *mm*

$s \ngtr (b \text{ or } 16\,\phi \text{ or } 300\ mm)$

(d) Diameter and Pitch of helical reinforcement *(clause 26.5.3.2d)*

(i) Diameter : The diameter of the helical reinforcement shall be not less than 5 *mm nor less than* one-fourth of the diameter of the largest longitudinal bar.

(ii) Pitch : Helical reinforcement shall be continuous going round the longitudinal bars with uniform spacing along the length of the column and its ends shall be properly anchored by providing one and a half extra turns of the spiral bar.

Where an increased load of 5% is allowed for, the pitch of the helical turns shall satisfy the following requirements :

- Pitch $\ngtr$ (75 *mm or* 1/6 x core diameter of column) whichever is less
  $\nless$ (25 *mm or* 3 x diameter of helical bar) whichever is greater

5.5 TYPICAL PROBLEMS IN DETAILING

Some typical problems in detailing occur especially at joints since joints are the weakest link in a structural system. The detailing of structural members requires special attention because there is little scope for rectification after construction. The detailing of typical structures have been shown in the following figures. For further details see Reference[5.2]

5.5.1 Detailing for Members Subjected to Directional Changes.

The directional changes in the opening corners induce resultant force acting outwards tending to split the concrete and hence require proper detailing as shown in *Fig. 5.5.1*

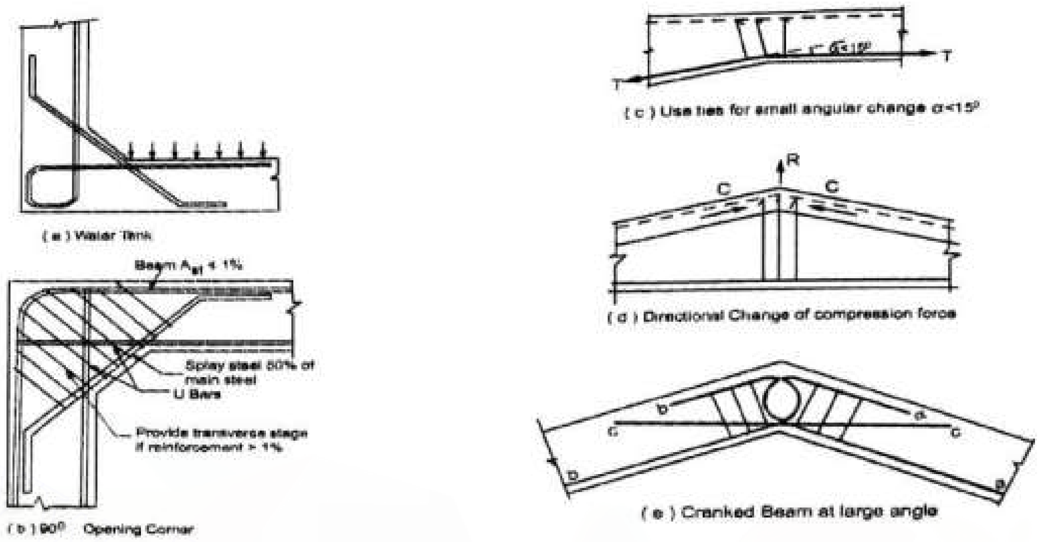


Fig. 5.5.1 Detailing for Members Subjected to Directional Changes.

5.5.2 Cantilever slab supported by Beam

Detailing of the cantilever slab supported at different levels of beam are given below.

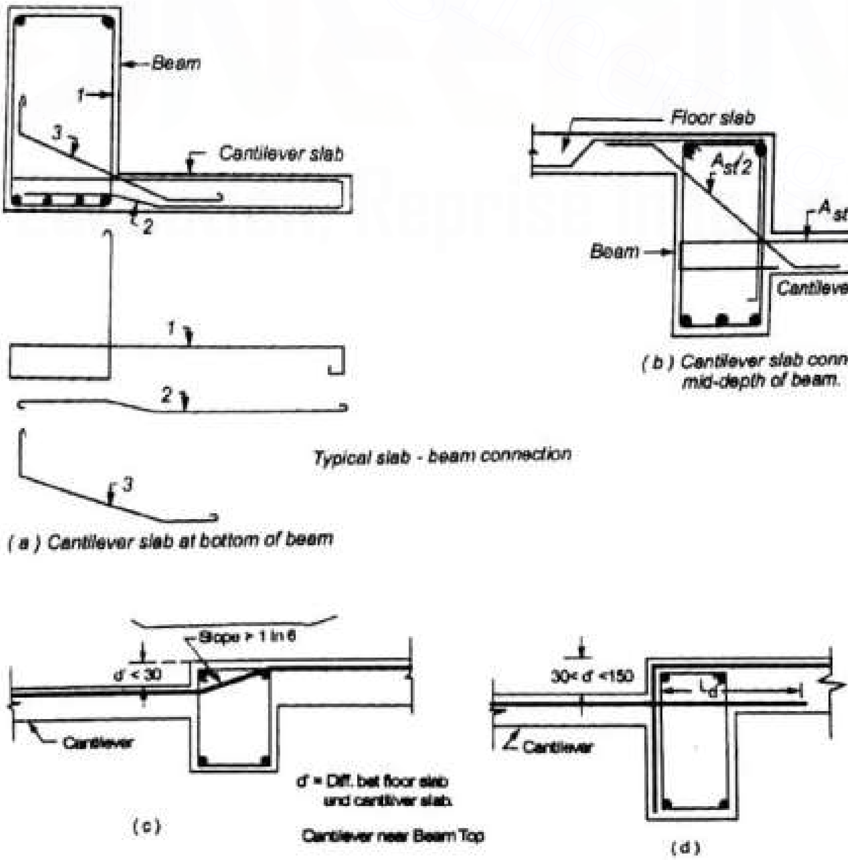


Fig. 5.5.2 Cantilever slab supported by Beam

5.5.3 Main beam Supporting Secondary Beam

The different problems that arise for supporting secondary beam with the main beam are as under :

(1) The depth of the main beam and supporting beams are unequal

(2) The depth of the main beam and supporting beams are equal

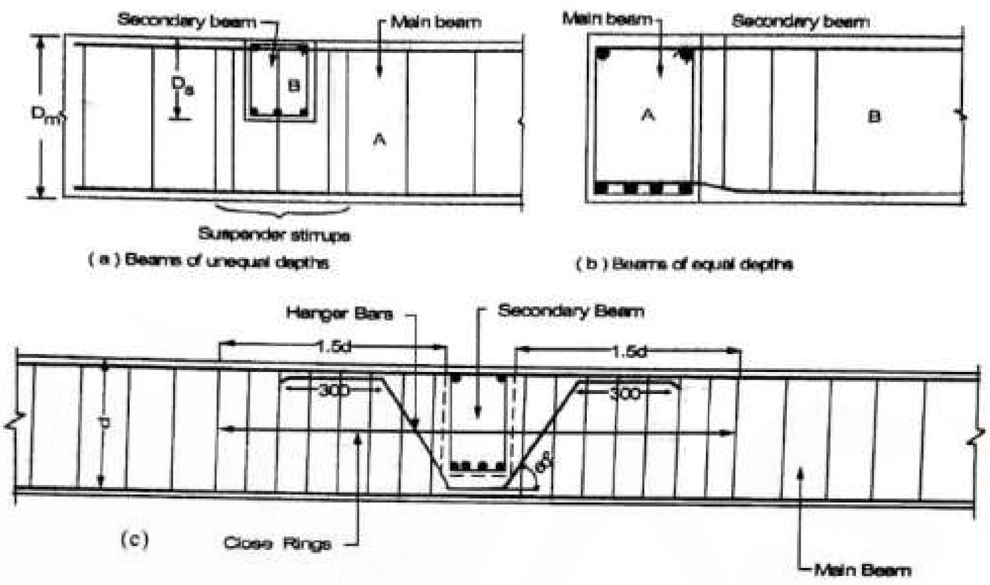


Fig. 5.5.3 Main Beam Supporting Secondary Beam

5.5.4 Anchoring of Shear Reinforcement of Inclined Bars

In beam the bars are bent up to resist shear. The first bent up bar shall be bent at a distance not exceeding $2d$ from the centre of support to obtain its contribution for resisting shear. The subsequent bar shall be bent at a distance not exceeding $(d/2 + d/2 \cot \alpha)$ i.e. d for $\alpha = 45°$. These bent up bars shall be considered to resist shear provided they are anchored for a distance of L_d from the mid-depth of the beam in compression zone. The details are shown in *Fig. 5.5.4*

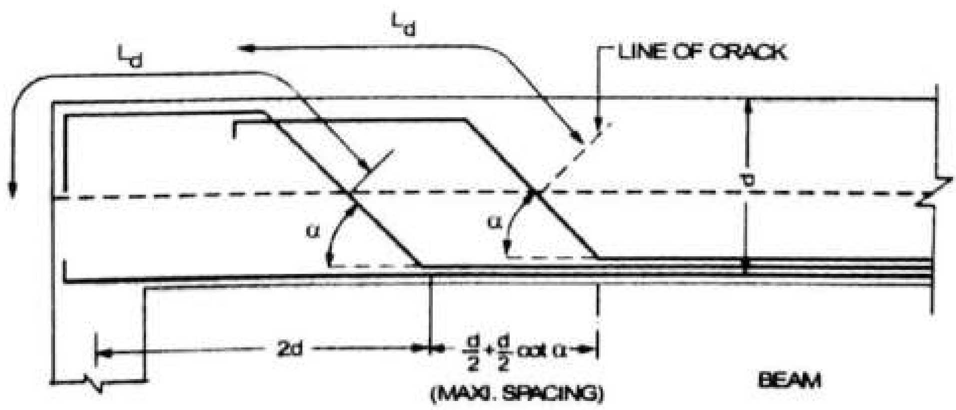


Fig. 5.5.4 Bent-up Bars in Beams

5.5.5 Splicing of Column

Since the length the column is much more than the length of the bars they are required to be spliced. The different methods of splicing are shown in *Fig.5.5.5*

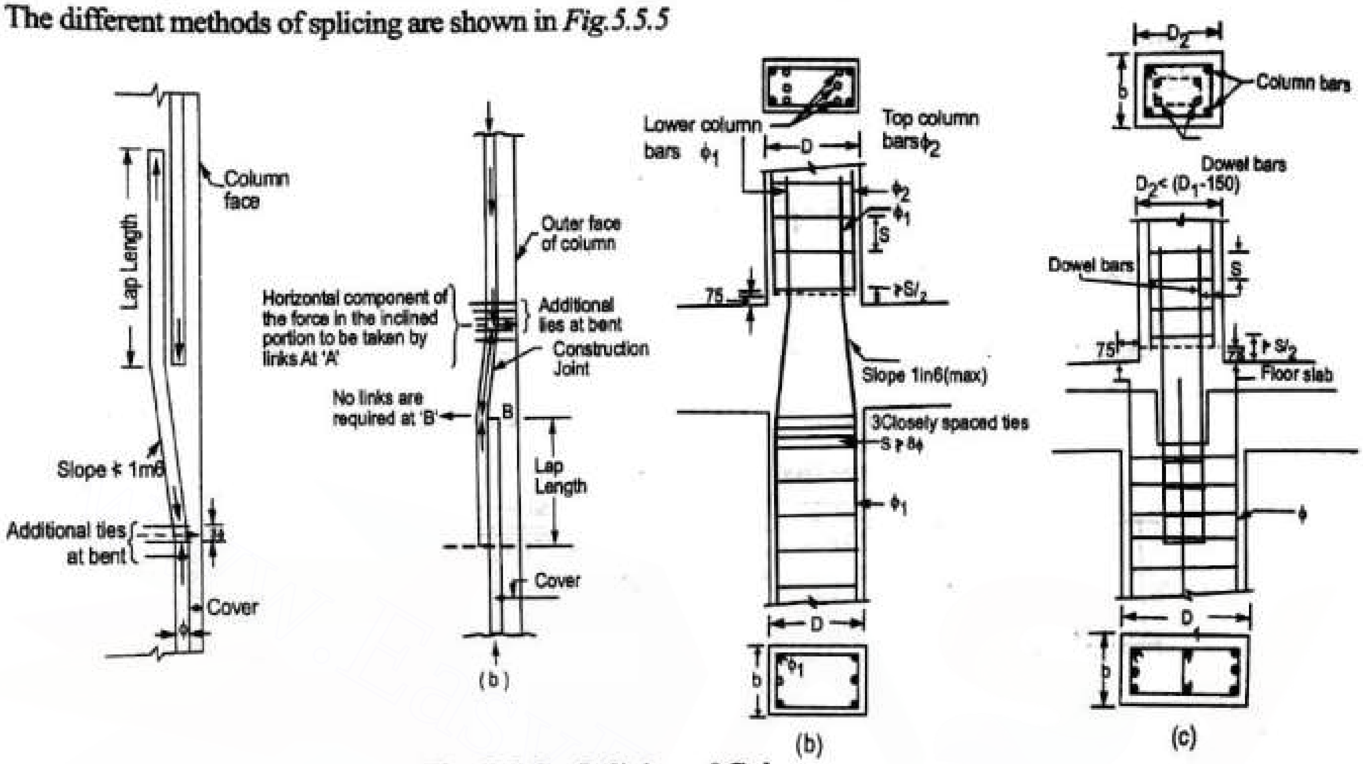


Fig. 5.5.5 Splicing of Columns

5.5.6 Beam - Column Connection

Depending on whether the beam is framed with the column from one side the required detailing is shown in *Fig. 5.5.6* , *Fig. 5.5.8* and *Fig. 5.5.9*

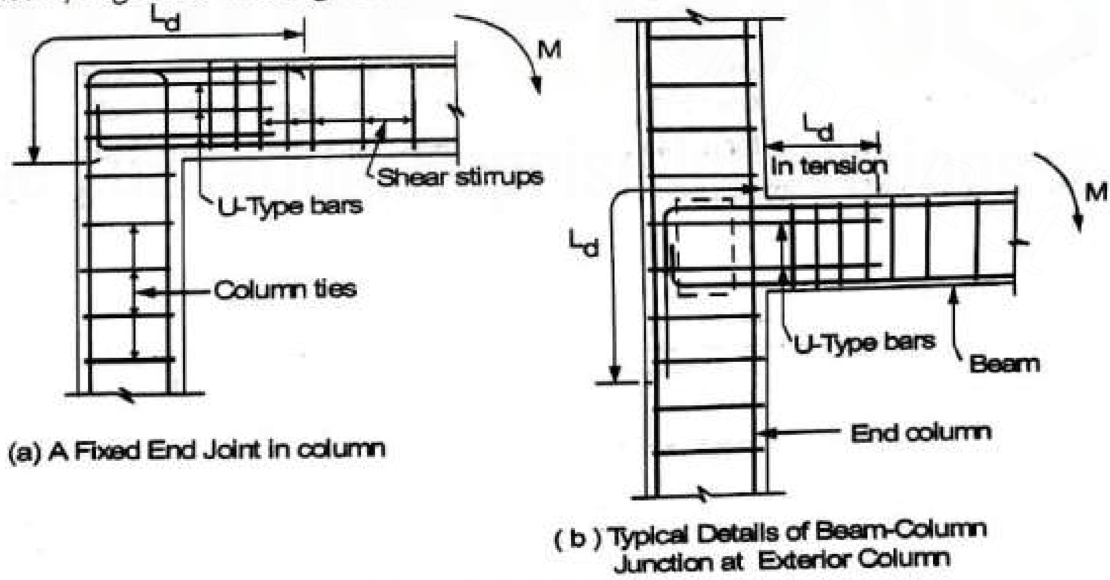


Fig.5.5.6 Beam-Column Connections from one aside

References :

5 .1 Venkatesh, N. , Sanchetee, H.C., "Design data book on concrete slabs", Tata - McGraw - Hill, New Delhi , 1994

5.2 Shah, V. L. and Karve, S.R., " Limit State theory and design of reinforced concrete", Structures Publications, Pune , 211009, Seventh Edition 2014, Chap - 8, Sect. 8.10.

CHAPTER - 6

DESIGN OF MEMBERS

The procedure for design of component members namely, slab, stair, beam, column, and column footing has been presented.

6.1 PRELIMINARIES

Before starting with the design, prepare a structural plan from the given architectural/building plan. For this, first of all, plan the structural frame according to the principles explained in *Chapter-1*. This involves determination of positions of columns, positions of beams, spanning of slabs, layout of stairs, and type of footing.

The structural plan will be drawn showing therein :

(i) positions of columns, beams, stairs, and spanning of slabs,
(ii) centre to centre dimensions between beams and columns to decide the span lengths of slabs and beams,
(iii) marking of slabs, beams, and columns using one of the marking schemes given in *Sect. 1.8 or* any other standardized/established method.

After the preparation of structural plan, the calculations will be done for unit loads as :

(i) unit loads on slabs of roof, floor, balconies, stairs, W.C. and bathrooms, lofts etc, (in *kN/m*.)
(ii) unit loads of walls (external, internal) per meter height, (in *kN/m*)
(iii) unit loads of parapet walls, grills, weather sheds etc. (in *kN/m*).

Once these preliminaries are over, design the frame components starting from slab, followed by stairs, beams, columns, and column footings provided sufficient time is available for doing the design prior to commencement of the construction work. However, if the work to be started *urgently*, it may be necessary to give sizes of footing and ground floor columns first. In such a case, the design will first be done of footings and columns by estimation of approximate equivalent axial load on columns as detailed in *Sect. 5.4.2* giving sufficient allowance for effect of continuity of slabs and beams, uniaxial/biaxial bending in columns due to fixity with beam; slenderness of column etc. wherever necessary. This approach, of course, should be avoided as far as possible.

In the text that follows both the procedures have been given for the design of members.

6.2 DESIGN OF SLAB

6.2.1 General

(a) One-way Slab

As mentioned earlier, one-way slab supported on opposite edges *or* when $L_y / L_x > 2$, predominantly bends in one direction across the span and acts like a wide beam of unit width.

If a continuous slab/beam loaded by *UDL* has equal spans *or* if the spans do not differ by more than 15% of the longest they are designed using IS : Code coefficients given in *Table. 5.1.1 (see Sect .7.2.3)*. For accurate analysis a continuous slab carrying ultimate load is analyzed using elastic method (*e.g.* using moment distribution method see *Sect. 7.2.3 Design - II)* with redistribution of moments.

The *approximate method* is to categorise one-way slab into the following types depending upon the end conditions :

(1) Simply supported slab	:	End Condition No. 1 *(EC = 1)*,
(2) Slab simply supported at One end, and Continuous at the Other end.	:	End Condition No. 2 *(EC = 2)*,
(3) Slab Continuous at Both ends.	:	End Condition No. 3 *(EC = 3)*,
(4) Miscellaneous types.	:	End Condition No. 4 *(EC = 4)*.

(b) Two-way slab

A rectangular slab supported on four edges with ratio of long span to short span less than 2 ($L_y/L_x < 2$) deflects in the form of a dish. It transfers the transverse load to its supporting edges by bending in both directions.

Two-way slabs are categorised into following two-types depending on the supporting conditions as follows:

(1) Slabs with Corners Restrained (*i.e.* corners not free to lift), and
(2) Slabs with Corners not Restrained (*i.e.* corners free to lift).

After having categorised the slabs under each category they may be designed according to the procedure given in *Sect. 6.2.2* and *6.2.3*.

6.2.2. Design of One-Way Slab

Steps

1. Slab Mark : Write the slab mark *or* designation such as *S1*, *S2*, etc.

2. End Condition : For *approximate* analysis write the end condition No. according to the category of the slab as given in *Sect. 6.2.1.*

Span (L) : Depending upon end conditions determine the effective span (see *Sect 5.2.3*) of the slab.

In fact, since the depth of slab is not known in advance (as it is to be designed) and the width of support is normally greater than the effective depth of slab, *in practice* the *effective span is taken equal to centre to centre distance between the supports to be on the safer side.*

3. Trial Section :

$$\text{Effective depth required } d = \frac{\text{Effective span } L}{\text{Basic } L/d \text{ ratio} \times \alpha_1} \qquad \text{... ... (6.2.1)}$$

where, Basic *L/d* ratio = 7 for cantilever, 20 for simply supported and 26 for continuous

α_1 = depends upon p_t% and steel stress of service load steel and is to be obtained as per *Fig. 4.7.1*.

But steel stress at service load (f_s) cannot be predicted and hence the modification factor α_1 cannot be found. To avoid this problem use graph given in *Fig.4.7.1* for $f_s = 290\ N/mm^2$, $f_s = 240\ N/mm^2$ and $f_s = 145\ N/mm^2$ for steel grade Fe 500, Fe 415 and Fe 250 respectively corresponding to percentage of tension steel required [6.1, 6.2].

Initially, assume p_t = 0.5% to 0.9% for steel of grade Fe250
p_t = 0.25% to 0.45% for steel of grade Fe415
p_t = 0.2% to 0.35% for Fe 500.

Obtain the nominal cover from *Table C-1*, add half the diameter of main steel ϕ to give effective cover.

$\therefore$ Effective cover = d' = Nominal cover + $\phi/2$ ... ... *(Table C-1)*

$\therefore$ Total depth of slab = D = effective depth + effective cover = $d + d'$

This should be rounded in multiple of 10 *mm*.

4. Loads :

Calculate load in *kN/m* on one metre wide strip of slab.

Dead Load : Self weight = w_s = $25D$ where, D shall be in *metre*. ... ... (6.2.2)

Floor Finish FF = As per *Table A-3*

Total dead load = DL = $w_d = w_s + FF$

Imposed Load = LL = As per *Table A-2*.

Total working load w = $DL + LL$

Total ultimate load w_u = $1.5\,w$

When maximum and minimum loads are required for analysis, calculate them as given below :

Maximum design load = w_{max} = $1.5\,(DL + LL) = w_u$,

and Minimum design load = w_{min} = DL.

5. *Design Moments :*

Design moment $M_u = \alpha \times w_u L^2$

where, α is design moment coefficient for approximate method given in *Table 6.2.1*

For approximate analysis the design moment coefficients may be taken as under :

Table 6.2.1 Design Moment Coefficients for Approximate Analysis

End Condition No.	EC =1	EC = 2	EC = 3
Design Moment Coefficient	α = 1/8	α = 1/10	α =1/12

As mentioned earlier the exact analysis may be carried out *or* the design moment obtained using I.S. Code coefficients given in *Table. 5.1.1*

6. *Check for Concrete Depth from bending moment criteria :*

Calculate the maximum moment carrying capacity of a balanced section.

$M_{ur.max} = R_{u.max}\ bd^2$. For slab, b = 1 metre = 1000 *mm*.

Obtain , $R_{u.max}$ from *Table 4.1.1*

Calculate $d_{reqd} = \sqrt{M_u / (R_{u.max} \times 1000)}$

Check than $d_{reqd} < d_{prov.}$

Alternatively, it is sufficient to Check that $M_{ur.max} \geq M_u$ for adequacy of concrete depth for strength.

7. *Main Steel (A_{st}) :*

$$\text{Required} \quad A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1-\sqrt{1-\frac{4.6M_u}{f_{ck} \times 1000 \times d^2}}\right] \times 1000 \times d \nless A_{st.min} \qquad (6.2.3)$$

where, $A_{st.min} = p_{t.min}\ bD$

$p_{t.min}$ = 0.12% of gross section for Fe 415 and Fe 500 and 0.15% for Fe 250

Assume bar diameter (8 *mm* or10 *mm* for steel of grade Fe 415, and 10*mm or* 12*mm* for Fe 250).

Required spacing $s = 1000\ a_{st}/A_{st}$ where, a_{st} is area of one bar. ... ...(6.2.4a)

Maximum spacing $s_{max} \leq$ (3*d* *or* 300 *mm*) whichever is less *(Eq. 5.2.2a)*

From practical consideration minimum spacing $s \geq$ 75*mm* *or* preferably $\nless$ 100 *mm* *(Eq. 5.2.3)*

Round off the value to multiple of 10 *mm* or 25 *mm* on lower side as desired.

The spacing shall preferably be between 100 *mm* and 200 *mm*.

8. *Check for Deflection :*

Calculate required p_t% (maximum value at mid - span of continuous slab or simply supported slab)

If $(p_t)_{assumed} < (p_t)_{required}$

Then the check may be considered to be satisfied else detailed check should be carried out as given in the Code as under :

Calculate steel stress of service load = $f_s = 0.58 \times f_y \times (A_{st})_{reqd.} / (A_{st})_{prov.}$ ... ... *(Eq. 4.7.1b)*

Obtain modification factor, α_1, corresponding to $(p_t)_{prov.}$ and f_s from *Fig. 4.7.1*

Required depth $d = L/$ (basic L/d ratio x α_1) < effective depth provided. ... ... *(Eq. 6.2.1)*

9. *Distribution Steel*

Required $A_{st.min}$ = 1.2 D for HYSD bars , $A_{st.min}$ = 1.5 D for Fe250 where, D in *mm* *(Eq.5.2.1)*

Assume bar diameter (6 *mm* for steel grade Fe 250 and 8*mm* for Fe 415).

Required spacing , $s = 1000\ a_{st}/A_{st.min}$ to be rounded off on lower side in multiple of 10*mm* .. (6.2.4b) or 25 *mm* as desired.

Maximum spacing, s =< (5*d* *or* 450 *mm*) whichever is less. *(Eq. 5.2.2b)*

In practice, spacing is kept between 150 *mm* to 300 *mm*.

10. Check for shear :

(a) Calculate design (maximum) shear.

In case of slabs, design shear may be taken equal to maximum shear $V_{u.max}$ at support, and is given by :

$V_{u.max}$ = w_u x L x Shear coefficient as per *Table 5.1.1(b)*

= w_u x $L/2$ for simply supported slab

where, w_u = ultimate *UDL* on slab/unit width.

In other cases, the maximum shear may be calculated from principles of mechanics.

(b) Calculate shear resistance (V_{uc}) of slab : (clause. 40.2)

This may be obtained from relation, $V_{uc} = \tau_{uc}.bd \times k$ ($b = 1000$ *mm* in case of slabs) *(Eq.4.4.3a)*

τ_{uc} depends upon $p_t = 100\, A_{st1} / bd$

τ_{uc} obtained from *Table 4.4.1* and k from *Table 4.4.2*

where , A_{st1} = area of tension steel. It is the bottom steel at simply supported end and top steel at continuous end.

$A_{st1} = A_{st}/2$ if alternate bars from mid-span are bent to top at simple support.

Check that $V_{uc} > V_{u.max}$. If not , increase the depth.

This check for shear is mostly satisfied in all cases of slabs subjected to uniformly distributed load and therefore, many times omitted in design calculations.

It may be noted that when the check for shear is obtained, it is not necessary to provide minimum stirrups as they are required in the case of beams.

11. Check for Development Length

Required $L_d \ngtr 1.3\, M_1/V + L_o$ … …*(Eq. 4.6.3b)*

For slabs, alternate bars are bent at support $M_1 = M_{u.max}/2$

and $L_o = b_s/2 - x_1 + 3\phi$ for HYSD bars using 90^0 bend … …*(Eq. 4.6.4a)*

$L_o = b_s/2 - x_1 + 13\phi$ for mild steel using 180^0 bend … … *(Eq. 4.6.4b)*

where, x_1 = end clearance *(see Fig. 4.6.1)*

6.2.3 Design of Two - Way Slab (Corners Restrained)

Steps

1. *Slab Mark :* Write slab designation *eg. S2,S3 etc.*
2. *End Condition No :* Write End (Boundary) Condition No. according to *Table D -7*
3. *Spans :* Determine Short Span L_x , Long Span L_y , check that $L_y/L_x < 2$
4. *Trial Depth (D) :*

It will be decided by deflection criteria based on short span L_x and the total depth D.

The allowable *L/D* ratio for two - way slab with short *span up to 3.5m* and for *loading class up to 3kN/m²* is given as under *(clause 24.1)*

Table 6.2.2 Allowable *L/D* Ratio for span ≤ 3.5*m* and loading class ≤ 3 *kN/m²*

End Condition	*L/D* Ratio	
	Grade of Steel	
	Fe 250	Fe 415 or Fe500
Simply supported slabs	35	28
Continuous slabs	40	32

(ii) If $L_x > 3.5\ m$ or live load > 3 kN/m^2, allowable L/d ratio will be the same as that for one - way slab. This may be obtained by assuming p_t % between 0.2% to 0.3%and proceeding as per *Step - 3* of *Sect. 6.2.2*

5. *Loads :* Calculate load for one metre width strip of slab. $w_u = 1.5\ (25D + FF + LL)\ kN/m$

6. *Design Moments :*

Obtain the bending moments by using the relation $M_u = \alpha\, w_u\, L_x^2$.

Values of coefficients, α_x for L_x and α_y for L_y may be obtained from *Table D - 7* for given boundary condition No. and the aspect ratio, L_y / L_x.

Comments : *It may be noted that :*

(i) *for long span also the bending moment is a function of* $w_u\, L_x^2$ *and* ***NOT*** $w_u\, L_y^2$,

(ii) *value of* α_y *is the same as that of* α_x *corresponding to aspect ratio = 1*

7. *Check for Concrete Depth from Bending Moment criteria :*

In the case of a two-way slab, effective depths for reinforcement in short span and long span at mid - span differ by a bar diameter since long span steel is placed above short span steel.

The effective depth d_o is for outer layer of short span steel and effective depth d_i is for inner layer of long span steel at *mid-span*. As far as support section is concerned, the effective depth is d_o only for both spans.

$d_o = D - (\text{nominal cover} + \phi/2)$, where , ϕ = diameter of bar

$d_i = d_o - \phi$ for mid-span long-span steel.

It is sufficient to check that $M_{ur.\,max} > M_{u.max}$, for adequacy of concrete depth from B.M. Criteria

8. *Main steel :*

Calculate area of steel required at four different locations using *Eq. 6.2.3*

Main steel calculated is provided only in the middle strips of width equal to 3/4th the slab width (at right angles to the span *i.e.* (3/4) L_y for short span steel and (3/4) L_x for long span steel). There will be no main steel parallel to the support in edge strip of width equal to 1/8th of slab width (*i.e.* $L_y/8$ for short span steel and $L_x/8$ for long span steel). In this edge strip, only distribution steel will be provided which is obtained as given in *Step - 9* of *Sect. 6.2.2*.Distribution steel shall also be provided for middle strip bars at top of supports.

9. *Check for deflection :*

If $L_x \leq 3.5\ m$ and $LL \leq 3\ kN/m^2$, check that $(L/D)_{provided} > (L/D)_{required}$ as per *Table 6.2.2*

For $L_x > 3.5\ m$ or $LL > 3\ kN/m^2$, the deflection check should be carried out as per *Step - 8 Sect. 6.2.2* of one-way slab taking reinforcement at middle of short span *i.e.* A_{stx}

10. *Torsion steel :*

At corners where slab is discontinuous over both the edges, $A_t = (3/4)\, A_{stx}$.

At corners where slab is discontinuous over only one edge, $A_t = (3/8)\, A_{stx}$.

At corners where slab is continuous over both the edges, $A_t = 0$.

The above area of torsion steel will be provided at corners over width equal to $L_x/5$ in each direction in each layer of bars provided orthogonally in two meshes - one at top and the other at bottom of slab.

In practice this may be achieved by providing the main steel in the edge strips also and continuing all the bars at bottom within a width $L_x/5$ each way and bending back the bars at top through 180° and continuing them through a distance $L_x/5$ at top.

11. *Check for shear :*

(a) Design (maximum) shear in two - way slab may be obtained using following relations.

At middle of short edge, $V_{u.max} = w_u\, L_x/3$ per unit width.

At middle of long edge, $V_{u.max} = w_u\, L_x\ [\beta/(2\beta + 1)]$ where, $\beta = L_y / L_x$.

Increase above value by 20% for shear at continuous edge and decrease the same by 10% at simply supported discontinuous edge for a slab simply supported at one edge and continuous over the other.

(b) Shear resistance and hence shear check is obtained in the same way as it is obtained for one-way slab *Step - 10b of Sect. 6.2.2.*

(c) Load carried by supporting beams of Two-way slab.

Long Edge : Trapezoidal load with ordinate $wL_x/2$.

Equivalent UD load for bending $w_{eqb} = \frac{wL_x}{2}\left[1 - \frac{1}{3\beta^2}\right]$ ... ...*(Eq. 5.3.5)*

Equivalent UD load for shear $w_{eqs} = \frac{wL_x}{2}\left[1 - \frac{1}{2\beta}\right]$ ... ...*(Eq. 5.3.6)*

Short Edge : Equivalent UD load for bending $w_{eqb} = wL_x/3$ *(Eq. 5.3.3)*

Equivalent UD load for shear $w_{eqs} = wL_x/4$ *(Eq. 5.3.4)*

In the case of slab simply supported at one end and continuous at the other reduce the load at simply supported end by 10% (*i.e.* take shear coefficient = 0.45) and increase the same by 20% at the continuous end (*i.e* take shear coefficient = 0.6) and 25% at continuous end of two span continuous beam.

12. *Check for Development Length :* It will be applied in the same way as it is applied for one -way slab.

Remarks : *The bending moment coefficients for rectangular slab simply supported on all four edges with corners free to lift have been given in Table D-2(a). In a frame structures the corners of the slab get restrained due to monolithic constructions. Therefore, the slabs of this type are of rare occurrence. The roof slab simply supported over brick masonry of a single storeyed building may be designed using coefficients given in Table D-2(b), such slab can be considered to have corners free to lift.*

6.2.4 Design of Stairs

Steps

1. *Data :* Staircase room size, floor to floor height (*H*), live load.
2. *Functional details :*

The guide lines for fixing dimensions of component parts of stairs have been given in *Sect. 1.3.4.* Based on the type of structure (whether residential *or* commercial) fix up the rise (*R*) and tread (*T*) of the stairs.

Number of risers = Floor to floor height / Rise = H/R

Number of risers for any flight $\leq$ 12

Based on these requirements decide the number of risers per flight.

Number of steps per flight = Number of risers – 1

Calculate, Going = Number of steps per flight x Tread

Depending on the size of stair - case room fix up the size of landing.

Compute the span L = Horizontal distance between the supports.

3. *Trial Depth* = Span/25 to Span/20. This is the thickness of waist slab.
4. *Loads* : w = self weight of slab + weight of steps + $FF + LL$

$= 25\,D\sec\phi + 25 \times R/2 + FF + LL$ where , D and R in meters,

$\sec\phi = \sqrt{T^2 + R^2}\,/\,T$

$w_u = 1.5\,w$

Remaining design steps are the same as those for one - way slab.

6.3 DESIGN OF BEAMS

6.3.1 General

In a building frame at every floor level, there can be large number of beams with different spans, end conditions, and loadings. It would not be practicable to design all beams serially from first to last. It is quite likely that some of the beams may have the same end conditions, spans, and/or loadings. Under such circumstances, it is always advisable to categorise them and group them to facilitate design, and reduce the computational efforts.

6.3.2 Categorization of Beams

The categorization of beams may be done on the basis of design which depends on the following factors :

(1) End Condition (EC = 1, 2, 3, 4), (2) Span,
(3) Load Type (UD load, Point load, triangular/trapezoidal load etc),
(4) Section Type (Rectangular / Flanged). (5) Load Magnitude.

Since categorization of beams would principally depend upon the end conditions of beam it is necessary, in the beginning, to take certain decisions *or* make suitable simplifying assumptions regarding the following :

(i) Whether the multispan continuous beams are to be analyzed and designed as a whole or as made up of independent beams with appropriate end conditions as explained in *Sect. 6.2.2 Step - 5*

(ii) What will be the end conditions of the beams ?

The decision would depend upon the following :

(1) Whether detailed calculations are required by the client (as in case of public buildings) for future/office record.

(2) Whether the client requires only the results in the form of schedules of members as in case of residential buildings constructed by private owners or builders.

(3) What is the accuracy required ? It depends upon the importance of the building and magnitude and repetitious nature of the work.

For example, If it is to be used for a big residential complex with large number of such units, then small excess of concrete and/or steel that may occur by using simplifying assumption in design of one unit can lead to appreciable increase in overall cost of materials in the entire big scheme.

The decision regarding the assumptions made for the end conditions of the beam materially affects the design procedure and design itself.

Bearing the above points in mind, the decision has to be taken very carefully whether to use the methods of structural analysis *or* simplifying assumptions and approximations . A beam may be assumed as simply supported at discontinuous end for simplicity on safer side, simultaneously taking care to provide steel at top at least equal to 1/3*rd* the midspan steel to account for partial fixity developed.

For *approximate* method, the beams may be categorised on the basis of end conditions as follows :

Category - I : Beam simply supported at both ends and carrying only uniformly distributed load.
- II : Beam simply supported at one end and continuous at the other end and carrying UD load.
- III : Beam continuous at both ends and carrying UD load only.
- IV : Miscellaneous beams such as overhanging beams, beams with any end condition but carrying unusual loading like UD load over part of the length of beam, continuous beams with abnormally unequal spans *etc.*

The beams under each category may further be divided into different groups on the basis of approximate equality of spans and loads. For beams with uniform cross section and having the same end conditions the equality of spans may be assumed when they do not differ more than 15% of the longest.

6.3.3. Beam Section

The cross sectional dimensions of the beam consists of fixing breadth and depth of the beam. The *breadth of the beam is generally kept equal to the thickness of the wall* to avoid offset inside the room. It shall also not exceed the width of the column for effective transfer of load from beam to column. The minimum width of beam shall be 200 *mm* to meet the requirements of fire resistance of 0.5 hours (see *Fig.1* of IS:456-2000).

The depth of the beam is taken between *L*/10 to *L*/16. The types of beam having different sections are kept minimum to facilitate reuse of form work. Even in some cases, specially in residential buildings, the depth of the beam is provided equal to the difference between the top of the floor and top of the door / window. The advantages are there is no need to provide lintel, the depth of the form work remains the same so that they can be reused and the top of the form work being at the same level there is considerable saving in labour.

6.3.4. Procedure for Design of Beam

Initially the beam is analyzed using one of the following methods :

(a) Exact Analysis : The beam is analyzed by rigorous linear elastic theory *(see Sect. 3.1.1)* to calculate the internal actions (Such as bending moment, shear force *etc.*) produced by ultimate load. Further redistribution of moment may be carried out to the maximum extend of 30% *(see Sect 3.1.3)*, if desired.

(b) Simplified Analysis :

(i) A simplified substitute frame analysis *(see Sect. 3.2.2)* can be used for determining the bending moments and shearing forces at any floor *or* roof level due to gravity loads. The redistribution of moment may be carried out, if desired.

Where side-sway considerations become critical due to unsymmetry in geometry *or* loading, rigorous analysis may be required.

(ii) A simplified analysis , for continuous beams of uniform cross-section supporting substantially uniform loads over three or more span which do not differ by more than 15% of the longest, may be carried out for obtaining bending moment and shear force using the coefficients given in *Table 5.1.1.* Where coefficients are used the redistribution of moments is not permitted.

(c) Approximate Method : In this case the continuous beam is further approximated by treating it to be made of independent single span beam loaded by a uniformly distributed load, and moments at supports are obtained using *Eq. 3.3.1* and *Eq. 3.3.2.*

The categorization of beam is made as detailed in *Sect. 6.3.2* by specifying End Condition No.

The steps that may be followed for design of beam are as under :

1. *Beam Mark* : Specify beam mark (*eg. B1 , B2 , etc*) as per selected scheme of marking.
2. *Span (L)* : In general, it may be taken as centre to centre distance between the supports, on the safer side. For exact value, refer to *Sect. 5.2.3*
3. *Section and Materials :* Assume grades of concrete and steel to be used and
   (a) breadth , $b \leq$ Breadth of Wall. *Common values* - 150, 200, 230, 250, 300, 350, 380, 400*mm.*
   (b) Depth , $D = L/10$ to $L/16$. *Common values* - 300, 380, 450, 530, 600, 680, 750, 840, 900*mm*
   (c) Effective cover : Assume effective cover d' between 40*mm* to 70*mm* depending on environmental condition a per *Table C-1.* The narrower the beam more is the effective cover required.

Decide whether the beam is acting as a flanged section *or* a rectangular section (See *Sect.3.3.3*) For flanged section it will be the weight due to rib portion only because the weight of the flange is already taken in to account in the design of slab.

(d) Effective depth = d = $(D - d')$ in *mm.* (e) Depth of flange = D_f = Depth of slab.

(f) Breadth of Support = b_s

5. *Loads :*

(a) Uniformly Distributed Load : (*w*) in *kN/m*

The load transferred from the slab per metre length of the beam will be either rectangular from one-way slab *or* trapezoidal / triangular from two-way slab as detailed in *Sect. 5.3.2.* Since the design of beam will be done in the tabular form it is necessary to decide the position of the slab with respect to the beam. The left *or* right side of the slab will be decided as we see the beams along its length from below and from right side of the plan.

(i) Slab from Left Side : The equivalent uniformly distributed load transferred from slab on left side of the beam is denoted as w_{s1}

(ii) Slab on the Right Side : The load transferred from the slab on right side is denoted as w_{s2}.

(iii) Masonry Wall : $w_w = \gamma\, t_w\, H_w$ where, t_w = thickness in *m*, H_w = height in *m*.,
γ = Unit weight of masonry ... ...*(Eq. 5.3.7)*

R.C. Wall : $w_w = 25 \times t \times H$, where, t = thickness in *m* (for grill take $t/2$ to account for openings)
H = height in *m*

(iv) Self weight : $w_s = 25 \times (D - D_f) \times b_w$ ... ...*(Eq. 5.3.8)*

(v) Total Working Load : $w = (w_{s1} + w_{s2}) + w_w + w_s$ for calculation of B.M., and S.F.

(vi) Design (Ultimate) Load : $w_u = 1.5w$ in *kN/m.*

(b) Point Loads : Give total No. of point loads = Number of secondary beams supported.

Give Data for each beam : Beam Shear W_i in *kN*, Distance (x_i) of W_i from the nearest support.

6. *Design moment :*

For flanged section calculate flange width b_f, either using *Eq.4.3.1or Eq. 4.3.2.* Check whether the neutral axis lies inside the flange or outside the flange.

$$\text{Calculate } (M_{ur})_{x_u = D_f} = 0.36 f_{ck}\, b_f D_f (d - 0.42\, D_f) \qquad \text{... ... (Eq. 4.3.8)}$$

If $M_{u.max} < (M_{ur})_{x_u = D_f}$, N.A lies inside the flange. , Where $M_{u.max}$ is maximum B.M.

This case normally governs in the case of slab- beam construction.

For continuous beam calculate maximum span moments and points of contraflexures using *Eq.2.6.1and 2.6.2*

7. *Main Steel :*

Depending on the type of beam calculate area of steel at mid - span,

For rectangular section :

$$\text{Required } A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck}\, bd^2}}\right] \times b \times d \qquad \text{... ... (Eq. 4.1.6a)}$$

For flanged section with $x_u < D_f$,

$$\text{Required } A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck}\, b_f d^2}}\right] \times b_f \times d \qquad \text{... ... (Eq. 4.3.5)}$$

The continuous beams at supports are generally required to be designed as a doubly reinforced section and rarely as a singly reinforced section.

The steps for design of a Doubly Reinforced section are as follows :

1. Calculate $M_{ur.max} = R_{u.max}\, bd^2$, $R_{u.max}$ to be obtained from *Table 4.1.1*

2. If $M_u < M_{ur.max}$, obtain A_{st} as per *Eq. 4.1.6a* else proceed to obtain areas of steel for a doubly reinforced section as under

$$\text{Required } A_{st1} = \frac{M_{ur.max}}{0.87 f_y\,(d - 0.42\, x_{u.max})} \qquad \text{... ... (Eq. 4.2.3a)}$$

or $A_{st1} = p_{t.max}\, bd / 100$, p_{tmax} to be obtained from *Table 4.1.1*

$$M_{u2} = M_u - M_{ur.max}$$

$$\text{Required } A_{st2} = \frac{M_{u2}}{0.87 f_y\,(d - d_c)} \qquad \text{... ...(Eq. 4.2.3b)}$$

Total area of tension steel = $A_{st} = A_{st1} + A_{st2}$

3. Calculate $$A_{sc} = \frac{0.87 f_y A_{st2}}{(f_{sc} - f_{cc})} \cong \frac{0.87 f_y A_{st2}}{f_{sc}} \qquad \text{... ...(Eq. 4.2.3c)}$$

for HYSD bars obtain f_{sc} corresponding $\varepsilon_{sc} = 0.0035\left(1 - \dfrac{d_c / d}{k_{u\,max}}\right)$ from *Table 4.2.1*

or from Table 4.2.2 corresponding to d_c/d or obtain A_{sc} directly from *Table 4.2.3*

8. *Detailing of Reinforcement.*

(a) Select number - diameter combination of bars using *Table H-2*

Required width b to accommodate N number of bars of diameter ϕ is :

Required $b = N\phi + (N + 1) \times 25 + 2 \times$ diameter of stirrups (ϕ_{st}) ... ...(6.3.1)

If assumed width < Required width, provide more number of rows and calculate the revised effective depth. Revise the area of steel if required.

Adhere to the detailing rules given in *Sect. 5.3.3*

(b) Decide

(i) No. of bottom straight bars at midspan.

(ii) No. of bent up bars if to be provided *or* curtailment of bars.

(iii) No. and diameter of straight bars at top as anchor bars. Normally anchor bars of 2 of 8, *or* 2 of 10, *or* 2 of 12 *mm* diameter are provided depending on the span of the beam

(iv) No. of extra bars required, if any, at top at left/right support. Advantage can be taken of the top anchor bars by continuing them over the supports.

9. *Design of Shear Reinforcement*

(a) Calculation of total uniformly distributed load on beam :

(i) In the case of beams carrying one-way slab the uniformly distributed loads w_u is the same as that calculated earlier for bending. Go to step *(b)*.

(ii) In case of beams carrying triangular/trapezoidal loads (supporting two - way slab), calculate equivalent uniformly distributed load for shear.

For trapezoidal load, $w_{u.eqs} = \frac{w_u L_x}{2}\left(1 - \frac{1}{2\beta}\right) = k_2 \times \frac{w_u L_x}{2}$, where, $k_2 = \left(1 - \frac{1}{2\beta}\right)$ *(Eq. 5.3.6)*

For triangular load on short edge beam carrying,

two - way slab, $w_{u.eqs} = w_u L_x / 4 = k_2 w_u L_x / 2$ where $k_2 = 1/2$ ... ...*(Eq. 5.3.4)*

one - way slab, $w_{u.eqs} = w_u L_x / 6$ where, w_u = load on slab. ... ...*(Eq.5.3.1a)*

Total load on beam $w_u = w_{u.eqs}$ from slabs on both sides + wall load + self weight

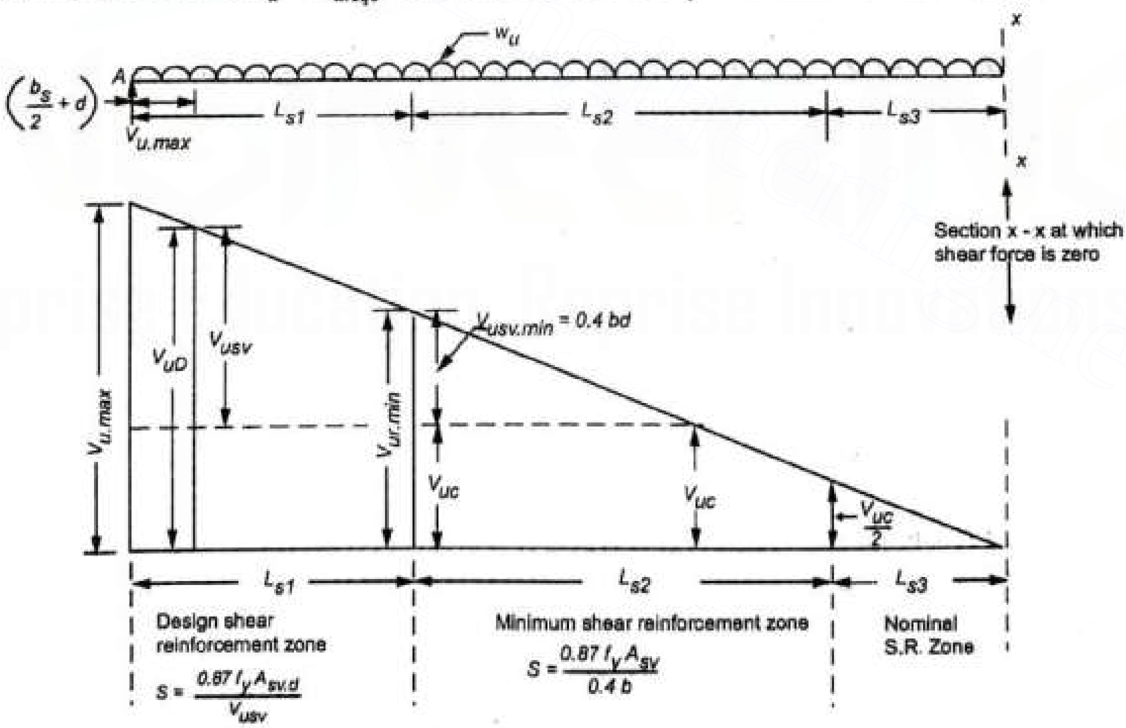


Fig. 6.3.1 Shear Force Diagram

Fig. 6.3.1 shows a beam simply supported at ends carrying *UDL* of intensity w_u

Let *x-x* be the section at which shear force is zero.

(b) Calculate maximum shear $V_{u.max}$ at the centre of support :

$$V_{u.max} = \alpha_d w_{uD} \times L + \alpha_L w_{uL} \times L$$

where, α_d and α_L = shear coefficients given in *Table 5.1.1b.*

(c) *Checking adequacy of concrete section :*

Compute maximum shear resistance of concrete in diagonal compression.

$V_{uc.max} = \tau_{uc.max}\ bd > V_{uD}$ else revise the section by increasing b or D

where, $\tau_{uc.max}$ = shear strength of concrete in diagonal compression,

= 2.8 N/mm^2 for M20 and 3.1 N/mm^2 for M25 concrete

where, Design shear $V_{uD} = V_{u.max} - w_u (b_s/2 + d)$ for support offering compressive reaction (Eq.4.4.1a)

$V_{uD} = V_{u.max} - w_u \times b_s/2$ for support offering tensile reaction ... ...(Eq.4.4.1b)

b_s = Breadth of Support.

(d) *Determination of Necessity of Design of Shear Reinforcement :*

(i) Compute shear resistance of concrete (V_{uc}) :

$$V_{uc} = \tau_{uc}.bd$$

where, τ_{uc} = shear strength of concrete given in *Table 4.4.1* corresponding to percentage (p_t) of tension steel at support and grade of concrete used *or* using *Eq.4.4.3.*

and $p_t = 100\ A_{stl}/bd$, where, A_{stl} = Area of tension steel at support.

(ii) Calculate the Shear Resistance of Minimum Stirrups ($V_{usv.min}$) :

$$V_{usv.min} = 0.4bd \qquad \text{... ...(Eq. 4.4.8)}$$

(iii) Therefore, Shear Resistance of Section with minimum Stirrups ($V_{ur.min}$) :

$$V_{ur.min} = V_{uc} + V_{usv.min} = V_{uc} + 0.4\ bd \qquad \text{... ...(Eq.4.4.9)}$$

(iv) If $V_{uD} < V_{ur.min}$ provide minimum stirrups at spacing given by :

$$s < \frac{0.87 f_y}{0.4b} A_{sv} \qquad \text{... ...(Eq. 4.4.7)}$$

where, A_{sv} = total area of vertical legs of stirrups,

b = width of the beam *or* breadth of web b_w, for flanged beams

s ≮ (0.75d *or* 300mm) whichever is less.

Note : *If $V_{u.max} < V_{ur.min}$, it is even not necessary to calculate V_{uD} but only minimum stirrups are required to be provided.*

If $V_{uD} > V_{ur.min}$, design the shear reinforcement else proceed to step (f)

(e) *Design of shear reinforcement :*

(i) Shear carried by shear reinforcement = $V_{us} = V_{uD} - V_{uc}$

(ii) If bent up bars are used, calculate shear carried by bent up bars

$$V_{usb} = 0.87 f_y A_{sb} \sin\alpha \not> 0.5\ V_{us} \qquad \text{... ...(Eq. 4.4.4)}$$

where, A_{sb} = area of bent up bar

α = angle between bent up bar and beam axis = 0.707 for α = 45°

Bend the bar preferably at a distance of 1.5d (≯ 2d) from the centre of support *(see Fig. 5.5.4)*

If bent up bars are not provided , $V_{usb} = 0$

(iii) Ultimate shear to be carried by stirrups,

$V_{usv} = (V_{us} - V_{usb}) \nless 0.5\ V_{us}$ or $V_{usv} = V_{us}$ when bent up bars are not provided.

Spacing s of design stirrups of area A_{sv} is given by :

$$s = \frac{0.87\ f_y\ A_{sv}.d}{V_{usv}} \qquad \text{... ... (Eq. 4.4.5)}$$

$s \leq$ (0.75 d *or* 300 mm) whichever is less.

(f) *Calculate zones of shear reinforcement (when $V_{usb} = 0$)*

(i) *Zone - I :* Zone of design shear reinforcement $L_{s1} = (V_{u.max} - V_{ur.min})/w_u$ (6.3.2a)

(ii) *Zone - II :* Zone of minimum shear reinforcement $L_{s2} = (V_{ur.min} - 0.5\,V_{uc})/w_u$ (6.3.2b)

If $V_{u.max} < V_{ur.min}$ *or* $V_{uD} < V_{ur.min}$, $L_{s1} = 0$, $L_{s2} = (V_{u.max} - 0.5V_{uc})/w_u$

(iii) *Zone - III :* Zone of nominal shear reinforcement $L_{s3} = 0.5\,V_{uc}/w_u$ (6.3.2c)

In zone L_{s3}, minimum diameter of stirrup of 6*mm* at maximum spacing $\ngtr$ (0.75*d or* 300*mm*) should be provided.

Comments :

(1) In practice only design shear reinforcement zone - I (if required) and minimum shear reinforcement Zone - II are provided taking Zone - III absent.

(2) Also many times maximum value of end shear $V_{u.max}$ is taken instead of V_{uD} to reduce computational efforts because zone of design shear L_{s1} is small no appreciable savings is obtained by using exact method.

9. *Check for Bond*

Bond is not very critical in beams. If required check for development length may be carried out at simple support and at point of contraflexure as detailed in *Sect. 4.6.5*

10. *Check for Deflection*

Check that $(d)_{provided} > \dfrac{\text{effective span } L}{\text{Basic } L/D \times \alpha_1 \times \alpha_2 \times \alpha_3 \times \alpha_4}$ ... ... *(Eq. 4.7.1a)*

In the case of beam deflection criteria is normally satisfied , because *L/d* < 16 and hence computations are skipped.

If required, actual deflection should be calculated and checked with permissible value *(See Ref [6.3])*.

Practical Note :

(1) Keep the types of beams and column sections minimum to enable one to reuse the same form work.

(2) It is economical to use less steel in column.

6.4 DESIGN OF COLUMNS

6.4.1 Introduction

The design of column necessitates determination of loads transferred from beams at different floor levels. Loads are transferred from slabs to beams and then to columns. Hence, slabs and beams are normally designed prior to the design of columns. This method called as *Exact method* which enables one to assess the loads on columns more accurately and thereby the design of column becomes realistic and economical.

However, in practice, many times situations arise which require the design of columns and footings to be given prior to the design of slabs and beams. In such a case, loads on columns and footings are required to be assessed using judgement based on past experience and using *approximate methods. The* loads on the columns can be determined approximately on the basis of floor area shared by each column as detailed in *Sect. 5.4.2*. These loads are normally calculated on the higher side so that they are not less than the actual loads transferred from slabs/beams. In such cases, the design of column is likely to be uneconomical.

In the sections that follow, the design procedure using both these approaches of column load calculations has been explained.

6.4.2 Design Procedure

Design of columns involves following steps:

(1) *Categorization of Columns :*

(a) *Category - I* : Internal Columns *or* Axially Loaded Columns.

(b) *Category - II* : Side Columns *or* Columns subjected to Axial Load and Uniaxial Bending

(c) *Category - III* : Corner Columns *or* Columns subjected to Axial Load and Biaxial Bending.

(2) *Computation of Loads on column,*

(3) *Calculation of Moments in Columns,*
(4) *Determination of Effective Length and Type of Column - Short or Long ,*
(5) *Grouping of Columns ,*
(6) *Design of Column Section.*

6.4.3 Categorization of Columns

Categorization of columns is extremely helpful because the procedure for design of column in each of the three categories is different.

The columns shall be first divided into the following three categories.

(I) *Category - I : Internal Columns or Axially Loaded Columns*

Internal columns carrying beams either in all four directions *or* only in opposite directions are predominantly subjected to axial compression because moments due to loads on beams on opposite sides balance each other. Judgement should be used to place a column under this category because if spans and/or loads on beams on opposite sides vary appreciably the beam moments on opposite sides may not balance each other and the column will be subjected to bending moment, and it will be required to be placed under the second category. Structurally, internal columns can be termed as *Axially Loaded columns.* Therefore, they require practically no or little allowance (say 5%) in axial load.

(II) *Category - II : Side columns or Columns subjected to Axial Compression and Uniaxial Bending*

Columns along the sides of a building, which carry beams either in three orthogonal directions or a single beam in one direction are subjected predominantly to axial load and uniaxial bending due to unbalanced moment transferred from a single beam on one side, while the moments from the other two beams in opposite directions balance each other provided their spans and loads on them are approximately equal. If such columns are to be designed as axially loaded columns using approximate method, the axial load is required to be increased to account for the effect of uniaxial bending in column. The load thus arrived is called *Equivalent axial load* for the purpose of design of column section.

(III) *Category - III : Corner Columns or Columns subjected to Axial compression and Biaxial Bending*

Corner columns *or* the columns which carry beams in two perpendicular directions are subjected to biaxial bending due to beams in orthogonal directions. They require large increase in axial load to account for the effect of biaxial bending for obtaining an *Equivalent axial load.*

6.4.4 Computation of Load on Column

(A) Exact Method : This method is used when the beam end shears are known prior to column design. The load on column at each floor level is given by

$$P_{u,floor} = V_1 + V_2 + V_3 + V_4 + P_a + P_{self} \quad \text{... ...(Eq. 5.4.1)}$$

where, V_1, V_2, V_3, V_4 = end shears of beams meeting at the column at the floor under consideration from all the four directions 1,2,3,4.*(see Fig. 5.4.1a)*

P_a = axial load coming from above.

P_{self} = self weight of the column at the floor under consideration.

(B) Approximate Method : This method is used when the design of footing is required to be given prior to design of slab and beam , and approximate sizes of column are required to be assumed.

$$P_{u,floor} = P_{us} + P_{uw} + P_a \quad \text{... ... (Eq. 5.4.2c)}$$

where, P_{us} = load transferred from slab to column at each floor level = $w_{us} \times A_{col}$ ... ... *(Eq. 5.4.2a)*

P_{uw} = wall load transferred to column at each floor level = $w_{uw} \times L_w$ ... ... *(Eq. 5.4.2b)*

P_a = load on column from above

For details see *Sect.5.4.2*

Above procedure of calculation of column loads does not work well when there are number of secondary beams. In such cases, approximate loads are required to be calculated on beams first and column load are obtained from beam shears. (see *Project - III*)

6.4.5 Calculation of Moments in Columns

This step may be omitted in preliminary design if column is designed by equivalent axial load approach in which case proceed to *Sect. 6.4.7* directly. The moments in column are obtained directly and exactly if the entire structural frame is analyzed using any method given in *Sect. 3.2.2* (For example, see ***Project II***). However, if the building cannot be divided into a number of frames due to peculiar positions of columns, as in some cases of residential buildings, *or* in building frames in which the connections are assumed to be simple (for example, a load bearing structure, *or* the building in ***Project - I***), the moments in columns at any floor level can be obtained by considering substitute column frame which consists of only the relevant column together with connected beams fixed at their far end. (see substitute frame - III ***Sect. 3.2.2c*** for column system)

The moment in the column can be calculated using the equation,

$$M_{col} = (k_c / \Sigma k) \times M_e \qquad \text{... ... (6.4.2)}$$

where, k_c = stiffness of column under consideration = I_c / L_c

Σk = sum of the stiffnesses of members meeting at the joint = $\Sigma k_c + \Sigma k_b / 2$

Stiffnesses of beams k_b shall be reduced to half to account for effect of members beyond the adjacent spans being ignored.

M_e = unbalanced fixed end moment at the joint.

= $w_u L^2/12$, if a single beam is rigidly connected to column on one side.

= $(w_{u1} L_1^2 / 12 - w_{u2} L_2^2/12)$, if two beams with unequal loads *or* unequal spans are rigidly connected on opposite sides of the column.

M_e = $w_u L^2 / 24$, if a single beam is simply connected to column.

= $(w_{u1} L_1^2 / 24 - w_{u2} L_2^2 / 24)$, if two beams with unequal loads *or* unequal spans are simply connected on opposite sides of the column, in which w_{u1}, w_{u2} are the loads and L_1, L_2 are lengths of beams on two sides.

The calculated moment in column shall not be less then $M_{u.min} = P_u \times e_{min}$,

where, e_{min} = minimum eccentricity given by *Eq. 4.8.5.*

When column above and below the floor level are of different sizes with their outer faces flush, the load from upper column becomes eccentric with respect to the lower column. However, it may be noted that the moment due to this eccentricity is opposite to the moment transferred by the beam to the column at that level. This in fact, results in reduction of the effective moment and hence the moment due to this eccentricity need not be considered. It needs consideration only when there is no floor beam in the plane of the offset.

6.4.6 Determination of Effective Length of Column and Type of Column

(a) Determination of Effective Length of Column : For computation of effective length of a column in a frame, first it is necessary to determine whether a column is a no sway column *or* a sway column. (see *Sect. 4.8.3*)

If stability index $Q \leq 0.04$, the column may be taken as a no sway else a sway column. ... ...*(Eq. 4.8.2)*

For normal usage when there are longitudinal and cross walls in both directions, the frame is assumed to be non-sway frame.

In absence of exact ratio of L_{eff}/L can be obtained from *Fig. 4.8.1* or *Fig. 4.8.2* for no sway column and sway column respectively corresponding to rotation release factor β. which is obtained from *Eq. 4.8.1.*

In residential building, it is difficult to divide the frame into number of plane frame due to its irregular layout.

In such a case, the effective length of the column may be approximately taken as follows :

(i) For top storey , $L_{eff} = L$

(ii) For intermediate storey , $L_{eff} = 0.8\,L$

(iii) For columns in bottom storey : When plinth beams are not provided $L_{eff} = 0.8\,L_f$

When plinth beams are provided $L_{eff} = 0.8\,L_w$

where, L = Floor to floor height – Depth of shallower floor beam framing into the column.
where, L_f = Distance between top of footing to the underside of the shallower beam at first floor level.
where, L_w = Distance between top of plinth beam to underside of the shallower beam at first floor level.

It may be noted that outer plinth beams are normally provided just below ground level and not at the plinth level so that peripheral walls can retain the plinth filling.
If there are no walls below first floor as in case of apartment buildings in cities where parking space is provided underneath, the entire structure above rests on the columns. In this case, there is a possibility for sway to occur and hence the effective length of the columns are taken equal to 1.2L to 2L depending on the end conditions. Here L is length of column from the soffit of shallower beam of first storey to the top of footing, in absence of plinth beam else from top plinth beam.

(b) Determination of Type of Column (Short or Long) :
If $L_{eff}/h < 12$, the column is short *else* it is slender *or* long. where, h = either b *or* D of the column
To begin with, it is necessary to decide whether the column will be short *or* long. This depends upon the slenderness ratio. Normally, effective lengths of columns are equal in two orthogonal planes and may be assumed to be the same. Thus, if $L_{eff\,x} = L_{eff\,y}$, the buckling under the action of axial load takes place about the weaker axis *i.e.y*-axis bisecting the width b of the column. Therefore, width b decides whether a column is short or long. For this, it is necessary to assume the width of column. Usual practical values of widths are 200*mm*,230*mm* and 300*mm*.Column having width equal to 230*mm* acts as a short column when its unsupported length does not exceed 2700*mm i.e.* when the floor to floor height does not exceed 3 metres assuming minimum depth of shallower beam to be 300 *mm*. The columns having width of 200*mm* are likely to be slender for floor to floor height greater than 2.7*m* (depth of beam has been assumed equal to 300*mm*) provided they are not braced at the lintel level.

6.4.7 Grouping of Columns

Once the load on each column and effective lengths are determined, the columns in the same category which have total loads on them not varying by more than 10 to 20% and having the same effective lengths may be grouped together. In such a case column carrying maximum load may only be designed in that group and the same section be adopted for all the columns in the that group. This saves the computational efforts considerably, and labour during the execution of work. This is of prime importance in practical design.

6.4.8 Design of Column Section

The design of section may be done by any of the following methods :

(A) Approximate Equivalent Axial Load Method.

(B) Exact (Theoretical) Method.

The exact method should be used in general. However, equivalent axial load approach may be used when a quick assessment of column section and footing details are required.

6.4.9 Approximate Equivalent Axial Load Method

In this approach, total equivalent axial load is obtained by adding calculated approximate axial loads, as detailed in *Sect. 5.4.2b*. Preliminary section is designed for this total equivalent axial load using the procedure for design of axially loaded columns explained in *Sect. 6.4.10(I)*. The section so obtained is later on checked by exact method for actual compression and bending moment .

I - Preliminary Design :

(a) Allowance for Moment : The calculated axial load on the column of each storey may be incremented by an allowance as detailed in *Sect. 5.4.2* to account for the effect of bending moment due to fixity between beams and column. This allowance is to be applied to the load of corresponding floor under consideration and *not* on the total load including the load from above.

(b) Allowance for Slenderness : If the column is slender an allowance is required to be provided for reduction in load carrying capacity due to slenderness effect. The allowance may be approximately taken corresponding to stress reduction coefficient C_r used in working stress method.

$$\% \text{ allowance} = (1/C_r - 1) \times 100 \qquad \text{where, } C_r = 1.25 - \frac{L_{eff}/b}{48}$$

The total axial load on column shall be further incremented by this percentage allowance value to account for the slenderness effect.

(c) Calculation of Total Equivalent Axial Design Load on column ($P_{u.eq}$) :

$P_{u.eq}$ = axial load on column obtained in *Sect. 5.4.2* + allowance for fixity + allowance for slenderness.

(d) Section Design : The section of the column is obtained using the procedure explained for design of axially loaded columns in *Sect. 6.4.10(I)*.
The minimum width of column shall be 200 *mm* to meet the fire resistance of 0.5 hour (see *Fig.1* IS:456-2000)

II - Check for moment in Column :

Calculate the moment carrying capacity of the designed section using either *Tables or* the interaction charts given in *Appendix - G*. From charts, the value of $M_u/(f_{ck}\, bD^2)$ can be obtained corresponding to calculated value of d'/D, p/f_{ck} and $P_u/(f_{ck}\, bD)$, from which value of moment resisting capacity M_{ur} can be obtained. If $M_{ur} \geq M_u$ acting on the section, then the section is safe. If not, the section is unsafe and hence revise the section.

For biaxial bending, check the safety of column as explained in *Sect. 6.4.10 (III)*. For slender columns, check the column as detailed in *Sect. 6.4.10(IV)*

6.4.10 Design of Columns Section - Exact Theoretical Method

Exact method of designing a column depends upon the type of column (*i.e.* whether the column is short or slender) and the type of loading (*i.e.* the category of column) whether the column is subjected to axial load only *or* subjected to combined axial load and uniaxial bending *or* combined axial load and biaxial bending. The procedure for design of each type and category of column is presented using design aids and not from first principles since design from first principles is extremely complicated, especially for columns subjected to combined bending and axial compression. Reader may refer to Authors' Text book[6.4] for the same.

(I) Axially Loaded Short Columns

In practice this is done by use of available ready made design tables. In absence of such design tables, the design is done by use of equations given in *Sect. 4.8*

(a) Practical Design by Use of Tables :

The design tables suitable for G + 3 buildings are given in *Appendix-G*. The appropriate depth and the number - diameter combination of bars is selected from tables given in *Appendix - H*.

Students *or* beginners can themselves prepare such tables for their design using the following formulae.

Load carried by concrete $\quad P_{uc} = 0.4\, f_{ck}\, bD$. Obtain P_{uc} for standard sizes.

Load carried by steel $\quad P_{us} = (0.67 f_y - 0.4 f_{ck})\, A_{sc}$.

Obtain values of P_{us} for different standard Number - Diameter Combinations of bars.

Load carried by the column $\quad P_u = \lambda\,(P_{uc} + P_{us})$ where values of λ depend on the width of column assumed and can be obtained from *Sect. 4.8.6a*. For $b = 200mm$, $\lambda = 0.8$ and for $b = 230mm$, $\lambda = 0.9$. The table giving load carrying capacity of the column for width of column equal to 200 *mm* and 230 *mm* for various number diameter combinations of bars have been given in *Appendix - G*.

(b) Theoretical Design by Use of Equations :

In absence of any such design aid, following procedure may be adopted.

(i) Assume percentage of steel (*p* between 1% to 3%). Higher percentage requires lesser area of concrete and vice - versa. (Common percentage used is between 1% to 2%).

For assumed percentage, calculate required gross area (A_g) using *Eq. 4.8.8b*.

Required $A_g = (P_u / \lambda)/[0.4 f_{ck} + (0.67 f_y - 0.4 f_{ck}) p_c]$ where , $p_c = A_{sc} / A_g$ ... ...(*Eq. 4.8.8c*)

(ii) From assumed width b , obtain depth $D = A_g /b$. Use practical dimension of : 230, 300, 380, 450, 530, 600, 750*mm etc. i.e.* in multiples of 50*mm or* in module of 3"

(iii) Calculate now A_{sc} for selected values of b and D using the formula :

$A_{sc} = (P_u / \lambda - 0.4 f_{ck}\, bD)/(0.67 f_y - 0.4 f_{ck})$. ... ...(*Eq. 4.8.8b*)

(iv) Select appropriate Number - Diameter combination of bars

(v) Assume diameter of lateral ties (ϕ_{tr} not less than 5*mm or* 1/4th the diameter (ϕ) of main bar whichever is greater). Normally, 6*mm* diameter ties are used for main bar diameter less than 25*mm*. Decide the pitch s of ties such that s is not greater than least of (16ϕ , 300*mm* and the least lateral dimension *i.e.* width b.)

II Short Columns Subjected to Axial Compression and Uniaxial Bending

The section has already been assumed earlier in calculation of bending moments. Determine the bending moments in columns, if not calculated earlier, as explained in *Sect. 6.4.5. Assume arrangement of bars. If the column is subjected to large bending moment M as compared to axial load P [say e/D = M/(PD) ≥ 0.5), assume bars to be equally placed on opposite faces like a doubly reinforced section. On the contrary, if P is large compared to bending moment M (i.e. e/D = M/(PD) < 0.5), assume bars to be uniformly placed all around the periphery.* The charts for bars equally distributed on all four sides have been prepared for a section with 20 bars equally distributed on all sides. These charts can be used without significant error for any number of bars *greater than* 8, provided the bars are equally distributed on the four sides. It may be noted that the second arrangement requires large area of steel than that required by the first arrangement. In case of ambiguity of deciding the arrangement, second one may be assumed on safer side.

Calculate A_{sc} from charts as detailed below :

(a) For bending about *x-axis* bisecting the depth of the column

1. Calculate $P_u / (f_{ck}\, bD)$ and $M_u / (f_{ck}\, bD^2)$
2. Calculate d'/D where , d' = effective cover
3. Select appropriate chart corresponding to d'/D , grade of steel and distribution of reinforcement . Obtain point of intersection of $P_u / (f_{ck} bD)$ and $M_u / (f_{ck} bD^2)$
4. Interpolate the value of p/f_{ck} where , $p = 100\, A_s / (bD)$
5. Calculate total area of steel required = $A_s = f_{ck} \times (pbD / 100)$

(b) For bending about *y-axis* bisecting the width of the column the chart to be referred to is having value of d'/b and use expression $M_u / (f_{ck} b^2 D)$. Rest of the procedure is the same as given above.

III Short Columns Subjected to Axial Compression and Biaxial Bending

(i) Assume steel percentage between 1% to 3%, and the Number - Diameter combination of bars for the same. Assume bars to be placed uniformly all around the periphery as this arrangement is better for biaxial bending. Calculate p/f_{ck} where, $p = 100\, A_s / (bD)$ and $P_u / (f_{ck} bD)$

(ii) Select appropriate chart corresponding to d'/D. Draw a horizontal line from $P_u / (f_{ck} bD)$ and continue it till it reaches a point corresponding to the value of p/f_{ck}. Drop a perpendicular on *x-axis* to give the value of $M_{ux1} / (f_{ck} bD^2)$. Calculate M_{ux1}

Repeat the process by selecting appropriate chart corresponding to d'/b and obtain the coefficient by dropping the perpendicular on *x-axis*. which gives $M_{uy1} / (f_{ck} b^2 D)$. Calculate M_{uy1}

(iii) Calculate P_{uz} using *Eq. 4.8.6* and hence ratio P_u / P_{uz}. From this ratio, obtain α_n from *Fig.4.8.3*.

(iv) Check that

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \le 1 \qquad \text{... ... (Eq. 4.8.12)}$$

If this equation is not satisfied, the section is unsafe. Increase the section and/or reinforcement and revise the calculations from *Step - (i)* above. If left hand side of the interaction equation is less than 0.8, the section is uneconomical. Reduce the reinforcement *or* reduce the section and repeat the procedure if desired. Continue with the trials until the section is safe and economical.

IV Slender Columns

(i) Calculate additional moment due to slenderness using *Eq. 4.8.13*. Obtain P_{uz} using *Eq. 4.8.6* and P_{ub} by using *Eq. 4.8.13d or* from first principles.

(ii) Calculate initial moments using *Eq. 4.8.14*.

(iii) Obtain total moment M_{uT} using *Eq. 4.8.15*. This is now the design moment for the column accompanied by given P_u.

(iv) Check the safety of column for combined effect of P_u and total moment M_{uT} using the procedure explained in *Sect. 6.4.10*.

(v) Revise the section if unsafe *or* uneconomical.

6.5 DESIGN OF ISOLATED FOOTING

The footing for an axially loading column of size $b \times D$ is designed as an inverted cantilever outstanding from column and loaded with uniform upward soil pressure. The various steps involved in the design are given below.

6.5.1 Proportioning of Base Size *(Fig. 6.5.1)*

Initially suitable footing dimensions are required to be selected to ensure that under serviceability conditions (*i.e. under working load*) the soil bearing pressure is not exceeded. The maximum load transferred to the soil is equal to axial load on column plus self weight of the footing. Since the size of the footing is unknown, its self weight is assumed to be equal to 10% of the axial load on the column.

If the axial load *(working)* on column is P then,

$$\text{Area of footing} = A_f = 1.1\ P/f_b = L_f \times B_f \qquad \text{... ... (6.5.1)}$$

where, L_f = Length of Footing,

B_f = Breadth of Footing,

f_b = safe Bearing capacity of soil.

Once the area of footing is known the size of footing gets fixed. The shape of the footing may be square *or* rectangular *or* circular. The size of the rectangular base is selected such that the cantilever projections of the footing from the faces of the column are equal. This gives approximately the same depth for bending about x and y-axes. The length *or* breadth of the footing based on equal projection is obtained as under :

Cantilever projection of footing for bending about x - axis $= C_x = (L_f - D)/2$

Cantilever projection of footing for bending about y - axis $= C_y = (B_f - b)/2$

For equal projections, $(L_f - D)/2 = (B_f - b)/2$ *or* $B_f = L_f - D + b$

Substituting the value of B_f in *Eq. 6.5.1* and solving the quadratic equation in L_f we get,

$$L_f = \frac{D-b}{2} + \sqrt{\left(\frac{D-b}{2}\right)^2 + A_f} \qquad \text{... ... (6.5.2a)}$$

Select the length of footing by rounding out the value of L_f.

Recalculate, $C_x = (L_f - D)/2$ and $C_y = (B_f - b)/2$ ... ... (6.5.2b)

where, breadth of the footing $= B_f = b + 2 \times C_x$

and L_f and B_f are the length and breadth of footing provided.

$$\text{For square footing}, \ L_f = B_f = \sqrt{A_f} \qquad \text{... ... (6.5.2c)}$$

$$\text{Area of footing provided} = A_f = L_f \times B_f$$

Upward factored soil reaction $= w_u = P_u / A_f$, where, P_u = Load factor × axial force $= 1.5 \times P$ (6.5.3)

Comments :

(1) In calculating the upward factored soil reaction the self weight of the footing is not considered because the dead load of the footing acts in the opposite direction of soil pressure and hence does not induce any moment or shear in the footing.

(2) The value of w_u will work out to be greater than the bearing capacity of the soil. But this is not unsafe because the comparison can be made with the upward working soil reaction which can be obtained by dividing w_u by the load factor of 1.5. Then it will be seen that the value of working soil reaction so obtained (w_u /1.5) will be less that the bearing capacity of the soil.

6.5.2 Depth of Footing from Bending Moment Considerations

The maximum bending moment is calculated at the face of the column *or* pedestal by passing through the section a vertical plane which extends completely across the footing and computing the moment of forces acting over the entire area of footing on one side of the said plane.

Bending moment at the column face parallel to *x* - axis is : $M_{ux} = w_u B_f C_x^2 / 2$ ...... (6.5.4a)

Bending moment at the column face parallel to *y* - axis is : $M_{uy} = w_u L_f C_y^2 / 2$ ...... (6.5.4b)

Required effective depth for bending about *x* – axis : $d_x = \sqrt{\dfrac{M_{ux}}{R_{u.max} \times b_1}}$ ...... (6.5.5a)

Required effective depth for bending alone *y* - axis : $d_y = \sqrt{\dfrac{M_{uy}}{R_{u.max} \times D_1}}$ ...... (6.5.5b)

where, $b_1 = b + 2e$ and $D_1 = D + 2e$,

b = width of column , D = depth of column ,

e = offset provided at the top of footing for seating column form work.

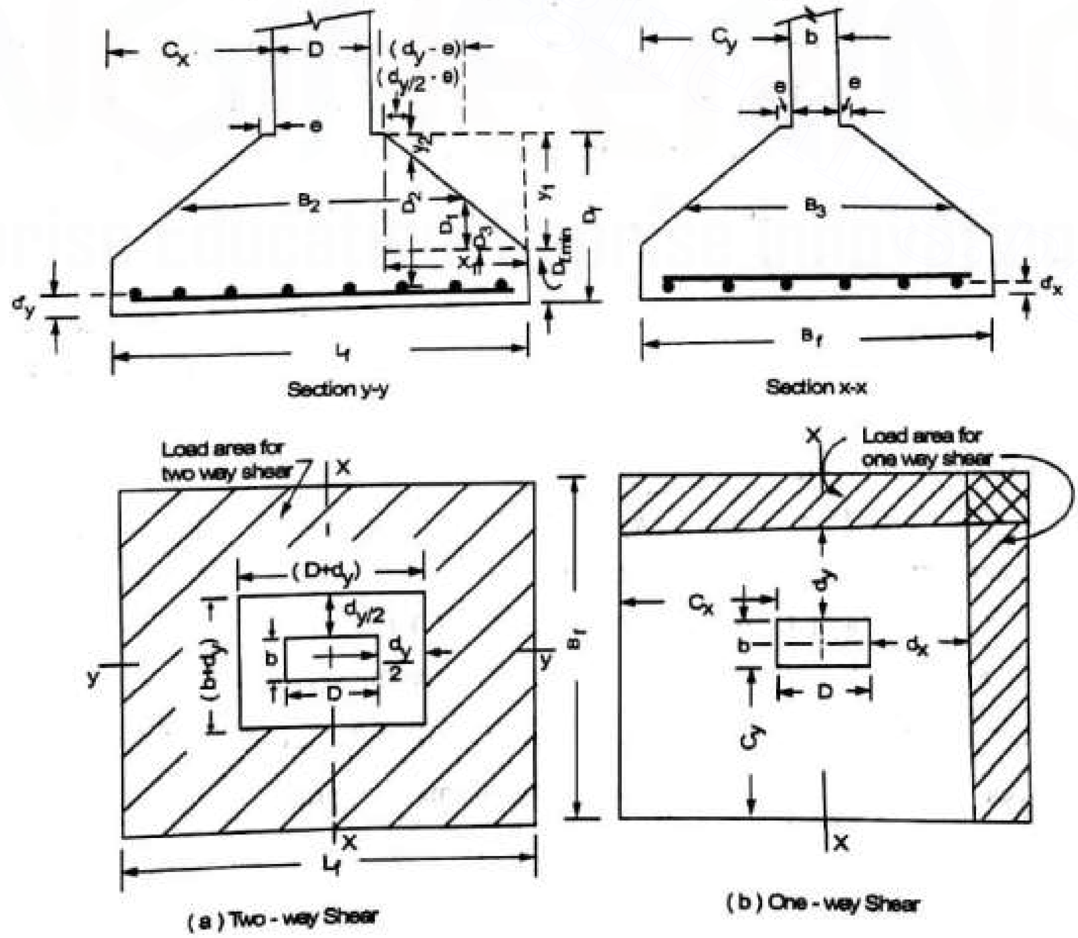


Fig. 6.5.1 Rectangular Sloped Footing

Notes :

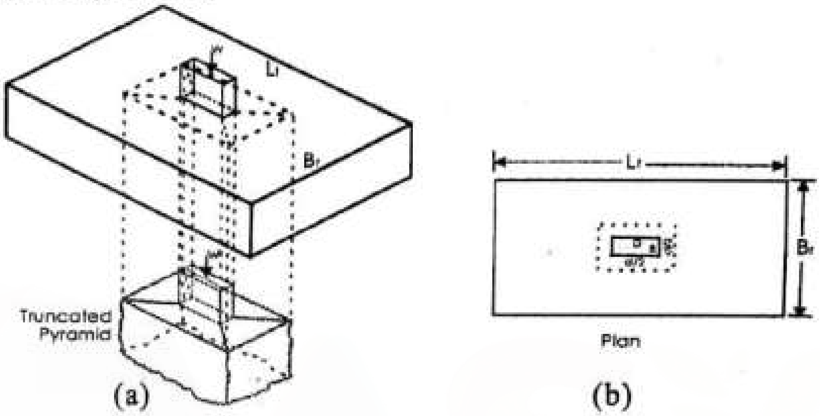
(1) In pad footing full width of footing is available for resisting bending moment while for sloped footing the resisting width at top is equal to column size plus twice the offset given at the top.

(2) In order to place the form work for column some top portion of the footing is kept leveled (called offset). The value of the offset depends upon the thickness of the form work which may be taken 50 mm or more.

6.5.3 Depth of Footing for Two – way Shear

A column supported by footing slab, tends to punch through the slab because of the shear stresses which act in the footing around the perimeter of the column. If fracture occurs, it takes the form of the truncated pyramid as shown in *Fig. 6.5.2a*. Test results indicate that the critical section can be taken at a distance *d*/2 from the face of the column *(Fig. 6.5.2b)*

Fig. 6.5.2 Two-way Shear Failure



Since the effective depth d_y is less than the effective depth d_x, the critical section is taken at a distance $d_y / 2$ from the periphery of the column. *(Fig. 6.5.1a)*

d_y = effective depth for upper layer of steel for bending about *y* - axis.

d'_y = effective cover for upper layer of steel for bending about *y* - axis.

d_x = effective depth for lower layer of steel for bending about *x* - axis.

d_x' = effective cover for lower layer of steel for bending about *x* - axis.

Perimeter at the critical section $= 2\,[\,b + d_y) + (D + d_y)\,] = 2\,(b + D + 2d_y)$ .... (6.5.6a)

∴ effective depth at peripheral section $= D_2 = D_f - y_2 - d_y'$ .... (6.5.6b)

where, $y_2 = (d_y / 2 - e)\, y_1 / x_1$, $y_1 = D_f - D_{f.min}$, $x_1 = (L_f - D - 2e)/2 = (C_x - e)$

Area resisting two – way shear $= A_2$ = perimeter x effective depth at the section

$= 2\,(b + D + 2d_y) \times D_2$ .... (6.5.6c)

Shear resisted by concrete $= V_{uc2} = \tau_{uc2} \times A_2$ .... (6.5.6d)

where , $\tau_{uc2} = k_s\, \tau'_{uc}$ and $k_s = (0.5 + b / D) > 1$, $\tau'_{uc} = 0.25\sqrt{f_{ck}}$

Design shear to which the column is subjected

$$V_{uD2} = w_u\,[\,L_f\, B_f - (D + d_y)\,(b + d_y)\,] \quad \text{.... .... (6.5.6e)}$$

If $V_{uc2} > V_{uD2}$ the section is safe else revise the section.

The minimum thickness of the footing at the edge shall not be less than 150 *mm*.

6.5.4 Area of Steel and Check for Development Length

Required area of steel for bending about *x* - axis

$$A_{stx} = \frac{0.5\, f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 \times M_{ux}}{f_{ck}\, b_1\, d_x^2}}\right] b_1\, d_x < A_{stx.min} \quad \text{.... .... (6.5.7a)}$$

Required area of steel for bending about y axis

$$A_{sty} = \frac{0.5 f_{ck}}{f_y}\left[1-\sqrt{1-\frac{4.6 \times M_{ux}}{f_{ck}\, b_1\, d_y^2}}\right] D_1\, d_y \nless A_{sty.min} \quad \text{... ... (6.5.7b)}$$

where, $b_1 = b + 2e$ and $D_1 = D + 2e$,

$A_{stx.min} = (0.85\, b_1\, d_x)/f_y$, $A_{sty.min} = (0.85\, D_1\, d_y)/f_y$

As the footing is designed as wide beam, minimum reinforcement as specified above shall be provided.

In rectangular footing the reinforcement parallel to the long direction shall be distributed uniformly across the width of the footing. In short direction, since the support provided to the footing by the column is concentrated near the middle, the moment per unit length is largest *i.e.* the curvature of the footing is sharpest immediately under the column and decreases with the increasing distance from the column. For this reason larger steel area is needed in the central band in the short direction and is determined in accordance with the equation given below :

$$\frac{\text{Reinforcement in central band width } B_f}{\text{Total reinforcement } (A_{sty}) \text{ in short direction}} = \frac{2}{L_f / B_f + 1} \quad \text{... ...(6.5.7c)}$$

The remainder of the reinforcement shall be uniformly distributed in the outer portion of the footing.

Required development length for bending about x - axis :

$(L_d)_{reqd} = L_{dx} = (0.87 f_y / 4\, \tau_{bd})\, \phi_x$, $(L_d)_{available} = C_x$

Required development length for bending about y – axis :

$(L_d)_{reqd.} = L_{dy} = (0.87 f_y / 4\, \tau_{bd})\, \phi_y$, $(L_d)_{available} = C_y$

where, ϕ_x and ϕ_y = diameter of bar for bending about x – axis and y – axis respectively

If $(L_d)_{available} > (L_d)_{reqd}$, the check for development length is satisfied.

The check for development length may become critical in the case of soil having large bearing capacity. In that case the size of the footing required works out to be small resulting in lesser cantilever projection.

If the development length requirement is not satisfied, it can be made to satisfy by three alternative methods :

(i) Decrease the bar diameter,

(ii) Increase the width or length of the footing such that the projection of the footing from the face of column (*i.e.* C_x or C_y) is not less than the required development length plus cover,

(iii) Provide 90° bend at the edge of footing,

(iv) Increase the area of steel so that the stress in bar will get reduced and the modified development length (L_{dm}) is approximately given by :

$$L_{dm} = \frac{A_{s.reqd}}{A_{s.provided}} \times L_d$$

Check that clear spacing between the bars is not less than 50 *mm*.

6.5.5 Check for One-way Shear for Bending about *y*-axis

One-way shear should be checked for bending about both x and y - axes. Since the effective depth d_y is less than d_x, shear for bending about y - axis is normally critical. The check for one - way shear is carried out on the same lines as in the case of beam.

Critical section for one - way shear about y - axis is taken at a distance d_y from the face of the column. If the cantilever portion of the footing for bending about y - axis is less than the depth of the footing one - way shear check is not necessary.

The check for one - way shear can be carried out using following steps :

Depth of footing above rectangular portion at critical section *(see Fig. 6.5.1)*

$$D_1 = y_1 - (d_y - e)\, y_1 / x_1 \qquad \text{... ... (6.5.8a)}$$

where, $y_1 = D_f - D_{f.min}$, $x_1 = C_y - e$

Width of the footing at critical section = $B_2 = D + 2d_y$ ... ... (6.5.8b)

Area of footing at critical section = Trapezoidal area + rectangular area

$$A_y = [(B_2 + L_f) \cdot D_1] / 2 + (D_{f.min} - d'_y)\, L_f \qquad \text{... ... (6.5.8c)}$$

where, d_y = effective cover for bars at top layer = $d'_x + \phi_y$

Percentage of steel = $p_{ty} = 100\, A_{sty} / A_y$

Design shear stress is calculated from *Table 4.4.1*

Shear resisted by concrete = $V_{ucy} = \tau_{ucy} \times A_y$ ... ... (6.5.8d)

Shear to which footing is subjected = $V_{uDy} = w_u L_f (C_y - d_y)$ ... ... (6.5.8e)

If $V_{ucy} > V_{uDy}$ the section is safe.

If $V_{ucy} < V_{uDy}$, the section is unsafe,

then either increase the steel or change the section of the footing and process repeated.

Note : *If the difference between the shear resistance of concrete section and design shear is less, the shear resistance of concrete can be increased by increasing the percentage of tension steel. However if this difference is large, the depth of the footing should be increased.*

6.5.6 Check for One - way Shear for Bending about x - axis

The critical section is taken at a distance d_x from the column. If $C_x < d_x$, one-way shear check is not required. The procedure for carrying out the check is similar to the one given *Sect.6.5.5*

Depth of footing above rectangular portion of footing

$$D_3 = y_1 - (d_x - e)\, y_1 / x_1 \qquad \text{... ... (6.5.9a)}$$

where, $y_1 = D_f - D_{f.min}$ and $x_1 = C_x - e$

Width at the top of footing = $B_3 = b + 2\, d_x$

Area of footing at critical section = $A_x = (B_3 + B_f)\, D_3 / 2 + (D_{f.min} - d'_x)\, B_f$ ... ... (6.5.9b)

Percentage of steel = $p_{tx} = 100\, A_{stx} / A_x$

Design shear stress τ_{ucx} is calculated from *Table 4.4.1*

Shear resisted by concrete = $V_{ucx} = \tau_{ucx} \times A_x$

Shear to which footing is subjected = $V_{uDx} = w_u B_f (C_x - d_x)$ ... ... (6.5.9c)

If $V_{ucx} > V_{uDx}$ the section is safe, else change the section *or* increase the steel.

6.5.7 Check for Bearing Pressure at Column Base

When the column rests on the footing it transfers its load only to the part area of footing. The adjacent concrete of footing provides lateral support to the directly loaded part of concrete. This causes increase in the strength of concrete which is loaded directly under the column.

This effect is taken into account and the code provides that : The compressive stress in concrete at the base of a column or pedestal shall be considered as being transferred by bearing to the top of the supporting pedestal or footing.

Based on this the code provides that :

Actual Bearing stress on the loaded area < Permissible bearing stress × $\sqrt{A_1 / A_2}$

i.e. $P_u / (bD) < (0.45 f_{ck}) \times \sqrt{A_1 / A_2}$ and $\sqrt{A_1 / A_2} \ngtr 2$ ... ... (6.5.10)

A_1 = Supporting area for bearing of footing, which in sloped or stepped footing, may be taken as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base, the area actually loaded and having side slope of one vertical to two horizontal (*see Fig. 6.5.3)*

$= L_f \times B_f$ or $(b + 4D_f)(D + 4D_f)$ whichever is less ... ... (6.5.10a)

or $= L_p \times B_p$ (where, L_p, B_p being the length and breadth of pedestal) ... ... (6.5.10b)

A_2 = Loaded area at the column base $= b \times D$

Eq.6.5.10 prevents crushing of concrete at contact surfaces of column base and top surface of footing. The factor $\sqrt{A_1/A_2}$ takes into account the increase in bearing capacity due to confinement of concrete under the loaded area by the surrounding concrete.

When the actual bearing stress is greater than the permissible bearing stress, reinforcement shall be provided for developing the excess force, either by extending the longitudinal bars into the supporting member, or by dowels.

When the excess force is transferred by the longitudinal column bars into the footing, the development length of the reinforcement shall be sufficient to transfer the compression or tension to the supporting member.

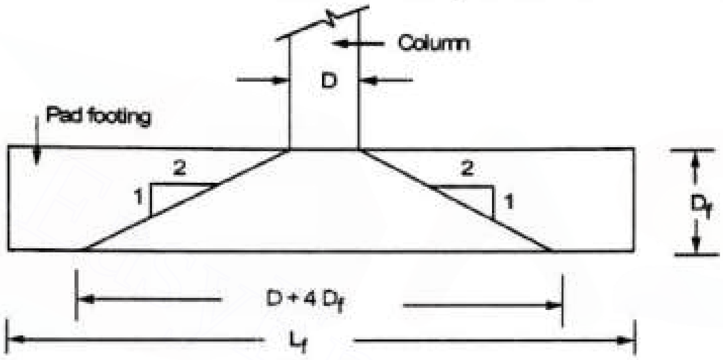


Fig. 6.5.3

Extended longitudinal reinforcement or dowels of at least 0.5 % of cross - sectional area of the supported column or pedestal and a minimum of 4 bars shall be provided. Where dowels are used, their diameter shall not exceed the diameter of column bars by more than 3 *mm*.

Column bars of diameter larger than 36 *mm*, in compression only can be dowelled at the footings with bars of smaller size of necessary area. The dowel shall extend into the column, a distance equal to the development length of the column bar and into the footing, a distance equal to the development length of the dowel. The details are shown in *Fig. 6.5.4*

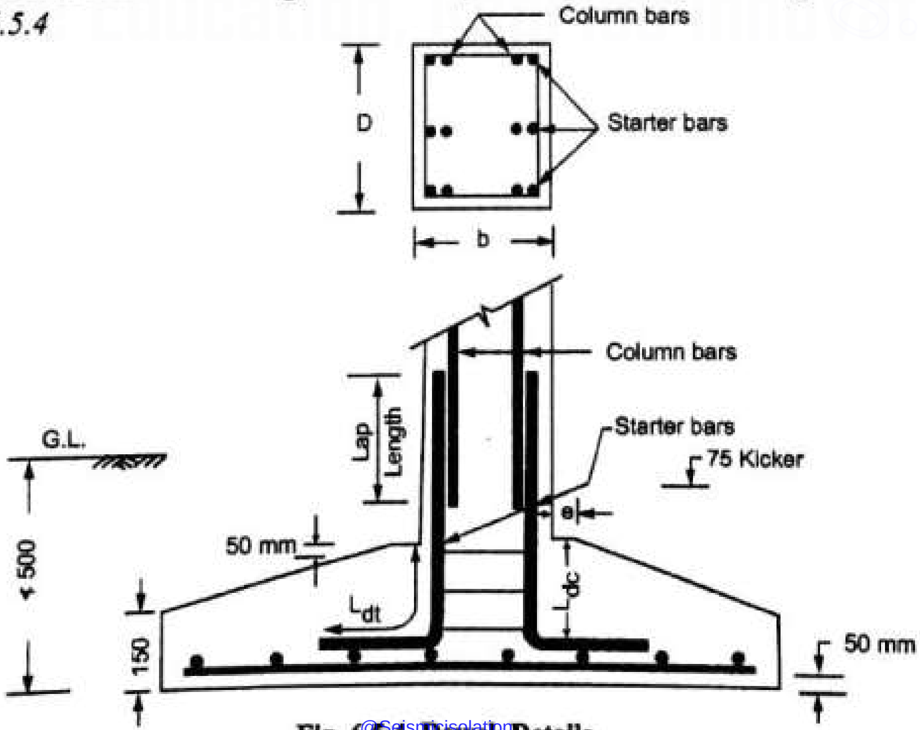


Fig. 6.5.4 Dowel Details.

6.5.8 References

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CHAPTER - 7

PROJECT - 1 : DESIGN OF SINGLE STOREY PUBLIC BUILDING

7.1 INTRODUCTION

7.1.1 General

Three projects have been selected illustrating design of three different types of buildings. In *Project - I*, a single storeyed public building has been designed in detail from first principles without the use of any design aid, using Limit State Method of design and S.I. units, conforming to IS : 456 - 2000.

Since, the object of this project is to illustrate the design of R.C. members rather than the analysis of a framed structure, a typical single storeyed structure has been taken for design.

The plan is so chosen that it incorporates design of different types of members namely :

Slabs : Cantilever *(S1)*, Simply Supported *(S2)*, One - way Continuous *(S3)*,
Two - way Continuous - corners restrained *(S4)*,
Two - way Simply Supported - corners free *(S5)*, and Stairs.

Beams : Simply Supported at both ends *(B26 , B27)*
Simply Supported at one end and Continuous at the other *(B1)*,
Continuous at both ends *(B2)*,
Large span flanged beam *(B19)*,
Continuous with two equal spans *(B22 – B23)*,
Continuous with two unequal spans *(B17 – B18)*, and
Continuous with non - central point load *(B5)*

Column : Axially Loaded *(C2)*, Eccentrically Loaded - Uniaxial Bending *(C8)*,
Eccentrically Loaded - Biaxial bending *(C15)*.

Column Footings : Axially loaded isolated sloped footing.

This design will give a good exercise for a beginner in the field of design in understanding the design principles and practices.

7.1.2 Data

1. *Type* : Single Storeyed R.C. Framed Structure.
2. *Plan* : As shown in *Fig. 7.1.1*
3. *Use* : Public purpose, Assembly hall
4. *Geometric Details* : Floor to floor height = 3.2 *Metres*
   Height of plinth = 0.6 *Metre* above ground level.
   Depth of foundation = 0.72 *Metre* below ground level.
5. *Loads* : Live Load - access provided = 1.5 kN/m^2
   - access not provided = 0.75 kN/m^2
   Floor finish = 1.75 kN/m^2
6. *Specifications for Materials and Building Components :*
   Roof : R.C. Slab, flat type with waterproofing course.
   Walls : Brick Masonry 230*mm* thick duly plastered.
   Concrete : Grade M20
   Steel : Grades, Main - Fe 415, Secondary - Fe250.
7. *Foundation* : Bearing Capacity of Soil = 150 kN/m^2
8. *Design Assumptions* : Slab simply supported over beams and beams simply supported over columns.
9. *Design Philosophy* : Limit State Method conforming to IS : 456- 2000
10. *Exposure Conditions* : Mild Environment.

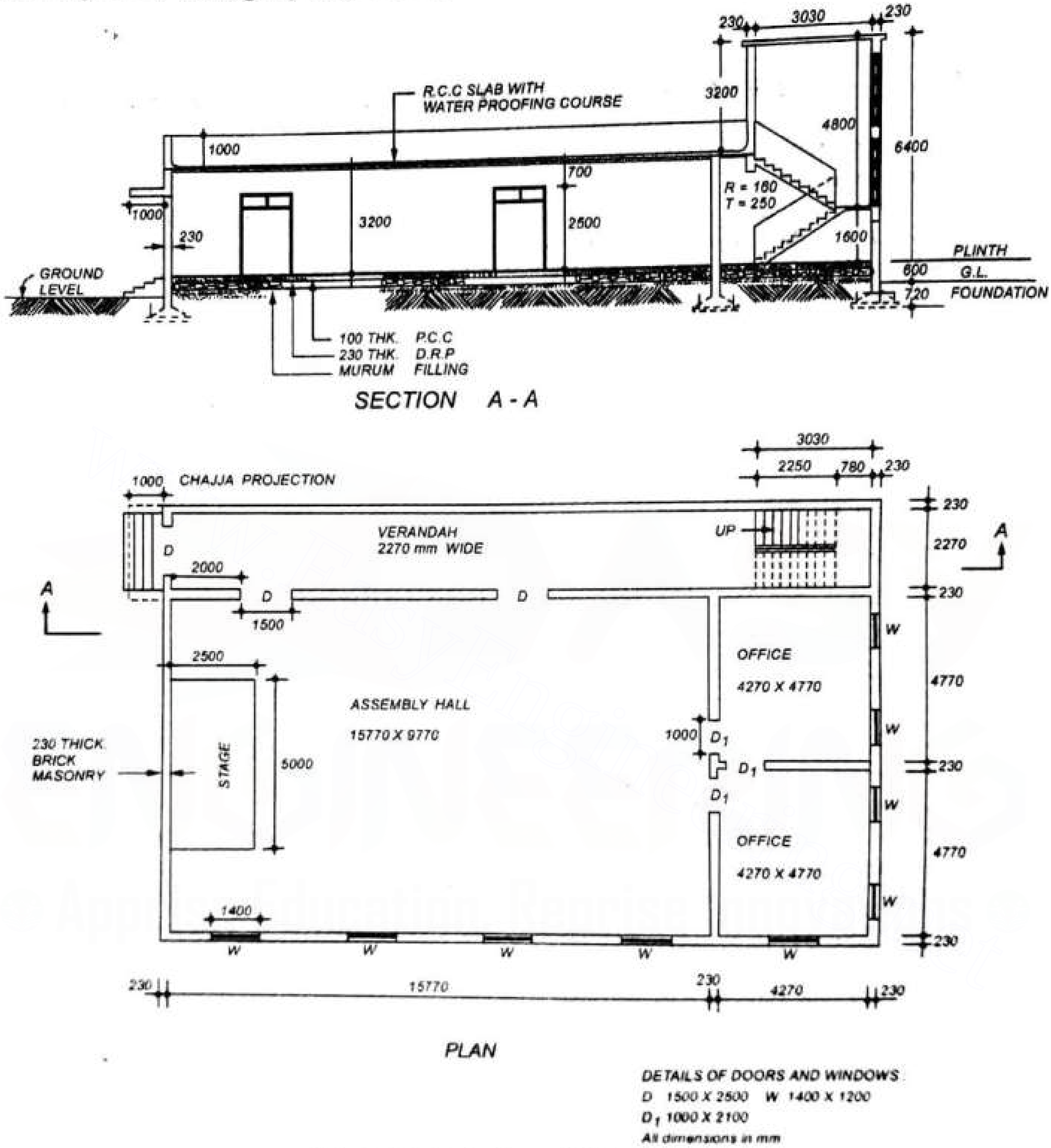
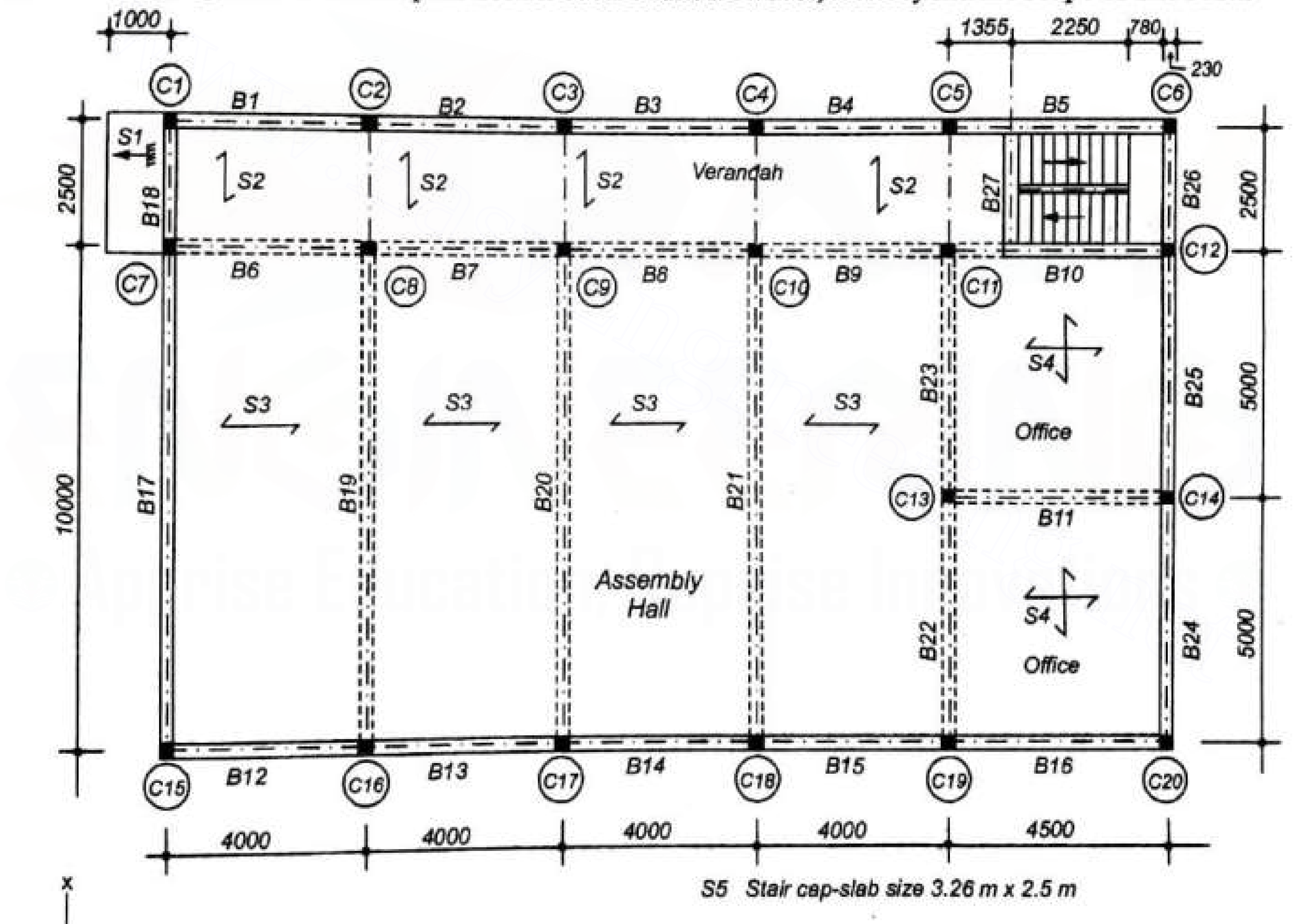


Fig. 7.1.1 Single Storey Public Building

7.1.3 Preliminaries

As explained in *Sect. 1.3*, the structural planning is to be done for the building plan shown in *Fig. 7.1.1*. In a building of this type having an assembly hall *or* a large size hall for public use, the positioning of columns is governed by the functional requirements that the entire hall space shall be unobstructed *i.e.* free of column. Therefore, the columns will be required to be positioned along the walls which should only be on the periphery of the hall. The exact positions of columns will be governed by positioning of beams which, in turn, is governed by the decision regarding spanning of slabs. The span of slab is normally between 3 *metre* to 4 *metre*, the lower value is for simply supported slabs while the higher one is for continuous slabs. For spans greater than 4 *metre*, it is advisable to provide two-way slab. In this case, since no column is to be provided in the middle of the hall main beams have to span across the width of the hall *i.e.* 10*m*. The solution will be to provide a one - way continuous slab over series of transverse beams of 10*m* span. For a length of hall of 16 metres, the solution of providing 3 intermediate beams at spacing 4 *m* will be more appropriate to give a span of 4*m* for a one - way continuous slab rather than providing 2 beams at spacing of 5.33 *m* or 4 beams giving spacing of 3.2*m*. As far as the slab over last span of 4.5*m* is concerned, one-way continuous slab also will be uneconomical, because it requires 10 *mm* more depth of slab and also requires larger quantity of steel hence it would be advantageous to provide a cross-beam above an intermediate wall between two office rooms and to support the slab and design it as a two-way continuous slab. The column positions (*C7* to *C12* and *C15* to *C20*) automatically get fixed under main beams and (*C13*, *C14*) at junctions of cross beam with main beams. Besides, since a longitudinal beam is required along the outer side of verandah to support the verandah slab, columns will be required to support this longitudinal beam. Architecturally and structurally, their ideal location would be just opposite to columns (*C7* to *C12*) supporting main beams. That would also help in providing tie beams across the verandah slab. The whole arrangement results into a structural plan of slabs, beams and columns as shown in *Fig. 7.1.2*. Alternatively, a grid floor is also possible for such a hall. However, it is beyond the scope of this book.



NOTES
(1) Marking of frame components according to practice followed in private sector.
(2) Concrete mix M20 and Steel reinforcement Main-Fe415 Dist.-Fe250 unless otherwise stated.
(3) All dimensions in mm

Fig. 7.1.2 Structural Plan

The design will be done in a sequence suitable for a beginner starting with design of slabs followed by design of stair, beams, columns and column footings. The frame components, namely, slabs, beams, and columns have been marked according to the practice followed by the private sector (See *Sect. 1.8.3.*)

7.2 DESIGN OF SLABS

This will comprise of design of slabs *S1*, *S2*, *S3* and *S4* in succession. Slab *S5* can only be designed after the planning of stair since slab *S5* forms a cap slab for the staircase.

7.2.1 Cantilever Slab - S1

Step No.	Design Calculations	Reference	Note
1.	*Slab Mark* : *S1*		
2.	*Type* : Cantilever		
3.	*Span* : $L = 1m = 1000\ mm$		
4.	*Trial Depth* : It is decided by serviceability criteria of deflection.		
	Basic L/d ratio , r_b = 7 for cantilever	*23.2.1 (a)	
	Assume $p_t = 0.15\%$ for M20 , Fe415 as load is very light		(1)
	Modification factor α_1 = 1.85 corresponding to $f_s = 240\ N/mm^2$	*Fig. 4.7.1*	
	Required eff. depth $d = \dfrac{\text{Span}}{\alpha_1 \times r_a} = \dfrac{1000}{1.85 \times 7}$ = say 80 *mm*	*Eq. 6.2.1*	
	Assuming eff. cover d' = 20 *mm* for Fe 415		(2)
	Required Total depth D = 80 + 20 = 100 *mm*		(3)
5.	*Load* : Consider a strip of slab of width $b = 1\ m = 1000\ mm$		
	Self weight = 25×0.1 = 2.50 *kN/m*	*Eq. 6.2.2*	
	Floor finish = 1.75 *kN/m*		
	Live load = 0.75 *kN/m*		(4)
	Total working load w = 5.00 *kN/m*		
	Design ultimate load $w_u = 1.5\ w = 1.5 \times 5.0 = 7.5\ kN/m$	*Table 2.4.1*	(5)
6.	*Design Moment* : $M_u = w_u L^2/2 = 7.5 \times 1^2/_2 = 3.75\ kN.m$		
7.	*Check for Concrete Depth from Bending Moment Criteria* :		
	$M_{ur.max} = R_{u.max}\, bd^2$, For slab, $b = 1000\ mm$		
	$R_{u.max}$ = 2.76 N/mm^2 for M20 - Fe 415	*Table 4.1.1*	
	$M_{ur.max} = 2.76 \times 1000 \times 80^2 \times 10^{-6}$		
	$= 17.6\ kN.m > M_u\ (= 3.75\ kN.m)$ ∴ safe		(6)
8.	*Main Steel* :		
	$A_{st} = \dfrac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \dfrac{4.6 \times 3.75 \times 10^6}{20 \times 1000 \times 80^2}}\right] \times 1000 \times 80$	*Eq. 6.2.3*	
	$= 135\ mm^2 \nless A_{st.min}$		
	Required $A_{st.min}$ = 0.12% of gross cross - sectional area	*Eq.5.2.1b*	
	$= 0.12 \times 1000 \times 100/100 = 120\ mm^2 < 135\ mm^2$		
	∴ $A_{st} = 135\ mm^2$		
	Assuming bar diameter = 8 *mm* , Area of bar $a_{st} = 50\ mm^2$		
	Required spacing $s = 1000 \times a_{st}/A_{st} = 1000 \times 50/135 = 370\ mm$	*Eq. 6.2.4a*	
	* Refers to clauses number given in IS : 456-2000		

Slab - S1 Continued....

Step No.	Design Calculations	Reference	Note
	Maximum permissible spacing = lesser of 3d (= 240 mm) or 300 mm	Eq.5.2.2a	
	∴ Provide # 8 at 240mm c/c, Area provided = 1000 × 50/240 = 208 mm^2		
9.	*Check for Deflection :*		
	$(p_t)_{reqd}$ = 100 × 135/(1000 × 80) = 0.17% > 0.15% (assumed)		(7)
	∴ Detailed check for deflection is carried out.		
	$(A_{st})_{reqd}$ = 135mm^2, $(A_{st})_{prov}$ = 208 mm^2		
	$(p_t)_{prov}$ = 100 × 208/(1000 × 80) = 0.26%		
	Steel stress = f_s = $0.58 f_y \times (A_{st})_{reqd} / (A_{st})_{prov}$		
	= 0.58 × 415 × 135/208 = 156 N/mm^2		
	Modification factor corresponding to p_t = 0.26% and f_s = 156 N/mm^2, $\alpha_1 = 2$	Fig. 4.7.1	
	Required d = 1000 / (2 × 7) = 71 mm < 80 mm ∴ safe		
10.	*Distribution Steel* : Assuming steel of grade Fe250,		(8)
	Required A_{st} = 0.15% of gross cross - section = 0.15 × 1000 × 100/100	Eq.5.2.1a	
	= 150 mm^2		
	Assuming 6mm bars. , a_{st} = 28.27mm^2, say 28 mm^2		
	s = 28 × 1000/ 150 = 186mm ∴ Provide s = 180 mm		
	Maximum permissible spacing = 5d or 450mm whichever is less	Eq.5.2.2b	
	= 5 × 80 or 450 mm < 180 mm ∴ O.K.		
	Provide ϕ6 mm at 180 mm c/c or #8 mm at 300 mm c/c		
11-12	*Check for Shear and Check for Development Length :*		
	These checks are normally satisfied in slab design, and hence they are usually omitted.		
	The sample calculations are given in design of slab S2.		
	*** Note : References with star * mark refer to clauses given IS : Code others refer to this book.**		

Explanatory Notes for Design of Slab S1 :

Note 1. The value of p_t satisfying requirements of both strength and serviceability normally lies between 0.15% to 0.45% for M20 Fe415. Assume smaller percentage for light loads and small span, and larger value for large span and heavy loads. With better judgement, lesser trials will be required. *Initially use graph given in Fig.4.7.1* for f_s = 290 , 240, and 145 N/mm^2 for Fe500, Fe415, Fe250 respectively.

Note 2. For mild environment and diameter of main reinforcement less than 12 mm,
Minimum nominal cover = 20 − 5 = 15 mm (see *Table C-1*)
Assuming maximum bar diameter equal to 10 mm for Fe 415 grade and 12 mm for Fe 250 grade
Effective cover d' = 15 + 10/2 = 20 mm for Fe 415 *or* Fe 500
and d' = 15 + 12/2 = 21 mm for Fe 250

Note 3. Values of effective depth d and total depth D are generally rounded off in multiples of 10mm *or* in module of 3"

Note 4. Since parapet wall extends only over beam *B18* (See *Fig. 7.1.1*), the cantilever slab *S1* is inaccessible, and therefore live load is taken as 0.75 kN/m^2.

Note 5. For strength design (*i.e.* for limit state of collapse in bending), the partial safety factor for load = 1.5. The design load, w_u, is ultimate load. w_u = 1.5 times the working load = 1.5 × w.

Note 6. To satisfy the check for concrete depth from bending moment considerations obtain d for a balanced section corresponding to design moment as :

$$d = \sqrt{M_u / (R_{u.max} \times 1000)}$$

If this calculated value of d works out to be less than the assumed value of effective depth, the requirements from B.M. criteria is satisfied.

In this case , $d = \sqrt{3.75 \times 10^6 / (2.76 \times 1000)} = 37\ mm < 80\ mm$

Alternatively, it is sufficient to show that the balanced moment of resistance

$(M_{ur.max} = R_{u.max}\ bd^2) >$ the design ultimate moment M_u.

This check is simple to apply and hence used in further design computations.

The value of M_u is normally expressed in $kN.m$. Since $R_{u.max}$ is in N/mm^2 and b and d are in mm, the product $R_{u.max}\ bd^2$ is multiplied by 10^{-6} to convert the value of $M_{ur.max}$ from $N.mm$ to $kN.m$. In this case even though, $M_{ur.max}$ is very large compared to the actual maximum design moment the depth cannot be reduced due to requirements of deflection. Reducing the depth not only increases area of steel required for strength but also steel percentage since $p_t\ \% = 100\ A_{st}/bd$. (Reduction in self weight due to reduction in depth of slab is very insignificant).

Note 7. It is sufficient to check that $(p_t)_{reqd} < (p_t)_{assumed.}$ This is because as p_t increases α_1 decreases with the result d increases. Therefore, when $(p_t)_{reqd}$ works out to be less than $(p_t)_{assumed}$ in the beginning, the modification factor α_1 increases thereby requiring depth less than the assumed trial depth. Therefore, check for deflection is automatically satisfied. If required $(p_t)_{reqd} > (p_t)_{assumed}$ detailed check as given above should be carried out and if required d works out to be greater than assumed d revise the calculations.

Note 8. Alternatively, if Fe415 grade of bars are to be used for distribution steel also, the smallest bar diameter available in market at present for this grade of steel being 8 mm. One may adopt # 8mm with minimum percentage 0.12% for Fe415 grade. In this problem $A_{st.min} = 120\ mm^2$ and spacing of bars works out to 410mm. Maximum permissible spacing for distribution steel is lesser of 5d and 450 mm.

Therefore, # 8 at 410 mm can be provided theoretically. In practice, however, maximum spacing is not allowed to exceed 300mm. Therefore, practical spacing of 300mm will be provided instead of 410 mm.

7.2.2 Simply Supported Slab - *S2*

Step No.	Design Calculations	Reference	Note
1. 2. 3. 4.	*Slab Mark* : *S2* *Type* : One - way, Simply Supported, Single Span, Mild environment *Span* : $L = 2.5\ m = 2500\ mm.$ *Trial Depth* : This is decided by deflection criteria Basic *L/d* ratio r_b = 20 for simply supported slab/beam Assuming $p_t = 0.3\%$ for M20-Fe415, $\alpha_1 = 1.48$ corresponding to $f_s = 240 N/mm^2$ Required eff. depth d = span/$(\alpha_1 \times r_b)$ = 2500/(1.48 x 20) = say 90 mm. Assuming eff.cover d' = 20 mm for Fe 415 Required total depth D = 90 + 20 = 110 mm	 *Eq.4.7.1* *Fig. 4.7.1* *Table C-1*	 (1,2) (3) (4)

Simply Supported Slab - S2 continued....

Step No.	Design Calculations	Reference	Note
5.	*Loads* : Consider one metre width of slab ($b = 1m = 1000\ mm$). Self weight = 25×0.11 = $2.75\ kN/m$ Floor finish = $1.75\ kN.m$ Live load = $1.50\ kN/m$ Total working load = $6.00\ kN/m$ Design ultimate load $w_u = 1.5w = 1.5 \times 6 = 9\ kN/m$		
6.	*Design Moment* : $M_u = w_u L^2/8 = 9 \times 2.5^2/8 = 7.03\ kN.m$		
7.	*Check for Concrete Depth* : For slab, $b = 1000\ mm$. $M_{ur.max} = R_{u.max} \times bd^2$. $R_{u.max} = 2.76\ N/mm^2$ for M20 Fe415. $M_{ur.max} = 2.76 \times 1000 \times 90^2 \times 10^{-6} = 22.3\ kN.m > M_u (= 7.03\ kN.m)$ ∴ safe	Table 4.1.1	
8.	*Main Steel* : $A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 7.03 \times 10^6}{20 \times 1000 \times 90^2}}\right] \times 1000 \times 90$ $= 229\ mm^2$ $(p_t)_{reqd.} = 100 \times 229 / (1000 \times 90) = 0.25\%$ Provide #8 mm at spacing s = $1000 \times 50/229 = 218\ mm$ say $200\ mm$. Permissible $s = 3d$ *or* $300mm$ whichever is less $= 3 \times 90$ *or* $300mm$ whichever is less>200 mm ∴ O.K Provide #8 mm at $200\ mm$ c/c, $(A_{st})_{prov.} = 1000 \times 50/200 = 250\ mm^2$	Eq. 6.2.3 Eq. 6.2.4a	
9.	*Check for Deflection* : $(p_t)_{reqd.} = 25\ \% < (p_t)_{assumed}$ (= 0.3%,) ∴ the deflection check can be considered to be satisfied. However, detailed check as per Code is worked out to substantiate the above statement. $f_s = 0.58 \times 415 \times 229/250 = 220\ N/mm^2$ $(p_t)_{prov} = 100 \times 250/(1000 \times 90) = 0.28\%$ for $p_t = 0.28\%$ and $f_s = 220\ N/mm^2$, $\alpha_1 = 1.62$ Required depth = $2500/(20 \times 1.62) = 77\ mm < 90\ mm$ ∴ safe	Eq.4.7.1b Fig.4.7.1	
10.	*Distribution Steel* : Required $A_{st} = 0.15 \times 1000 \times 110 / 100 = 165\ mm^2$ Assuming ϕ6 mm bars, $s = 28 \times 1000/165$ = say $160\ mm < (5d$ and $450mm)$ Provide distribution steel ϕ6 mm at $160\ mm$ c/c	Eq. 5.2.1a Eq. 6.2.4b	
11.	*Check for shear* : Design shear $V_{uD} = 9 \times 2.5/2 = 11.25$ kN Shear resistance of slab concrete $V_{uc} = k\,\tau_{uc}.bd$, $k = 1.3$ τ_{uc} depends upon $p_t = 100\ A_{st}/(bd)$. Assuming 50% bars only continued to support, $A_{stl} = (1000 a_{st}/s) / 2 = (1000 \times 50/200)/2 = 125\ mm^2$ $p_t = 100 \times 125/(1000 \times 90) = 0.14\ \%$., $\tau_{uc} = 0.28\ N/mm^2$ $V_{uc} = 1.3 \times 0.28 \times 1000 \times 90/1000 = 32.76\ kN >> 11.25kN$ ∴ safe	 Table 4.4.2 Table 4.4.1	(5)

Simply Supported Slab - S2 continued ....

Step No.	Design Calculations	Reference	Note
12.	*Check for Development Length* : For M20 - Fe415 $V_{u.max}$ = 9 x 2.5 / 2 = 11.25 kN, Since 50% bars are bent up, $M_1 = 7.03/2 = 3.52$ kN.m. Providing 90° bend, $L_o = b_s/2 - x_1 + 3\phi = 230/2 - 25 + 3 \times 8 = 114$ mm $L_d = 47\phi = 47 \times 8 = 376$mm, $1.3\ M_1/V + L_o = 1.3 \times 3.52 \times 1000/11.25 + 114 = 520$mm L_d (= 376 mm) < 520 mm ∴ safe.	 Eq.4.6.4a Table 4.6.2 Eq.4.6.3b	(5)

Explanatory Notes to Design of Slab - S2 :

Note 1 Since, this slab is primarily supported only over beams *B1-B2-B3-B4* on one side and beams *B6-B7-B8* and *B9* on the opposite sides and there is no transverse support (except beams *B18* and B27 at far ends), the slab spans only as one - way along the short span.

Note 2 Since slab *S3* adjacent to *S2* on the other side of support *B6-B7-B8- B9* spans in a direction at right angles to spanning direction of *S2*, it cannot be considered to give full fixity *i.e.* structural continuity to slab *S2* over supports *B6-B7-B8- B9*. It may be remembered that in design assumptions, it is assumed that slab rests simply on supporting beams at discontinuous end, and hence Beams *B1-B2-B3- B4* also form a simple support.
Thus, the slab *S2* will be designed as one - way slab simply supported at both ends.

Note 3 The effective span depends upon the type of connection between the slab and beam. If the provided connection is rigid (by anchoring top bars from slab into the beam by appropriate anchorage) the slab forms a part of structural frame and the effective span becomes centre to centre distance between supporting beams according to *Cl:22.2. (d)* of Code. Here in this problem, the connection between slab and beam is considered to be not rigid.
Therefore, *Clause.21.2(a)* of the Code applies.
Theoretically, effective span = (2500 – 230) + eff. depth = 2270 + 90 = 2360 *mm i.e.*2.36 *m*.

However, as per the practice centre to centre distance has been taken as effective span on safer side as it gives span greater than theoretical.

Note 4 For mild environment and for main reinforcement up to 12 *mm*, the nominal cover = 20 – 5 = 15 *mm*. *(see Note 2 of Table C-1).*
Assuming diameter of steel 10 *mm* effective cover = d' = 15 + 5 = 20 *mm*

Note 5 The check for shear and development length are normally satisfied in one-way slab and hence will be omitted for further slab design.

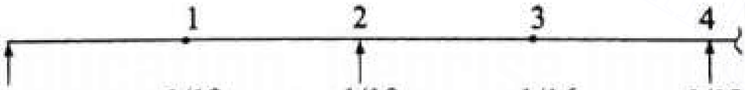
7.2.3 Continuous Slab - *S3*

This slab is designed by four different approaches:

(I) *Using I.S.Code coefficients,*
(II) *Exact Analysis with Redistribution of moments,*
(III) *Approximate Method, and*
(IV) *Very Approximate method.*

Design of Slab - S3 : Alternative - I : Using I.S. Code Coefficients

Step No.	*Design Calculations*	*Reference*	*Note*
1.	*Slab Mark* : S_3		
2.	*Type* : One - way Continuous with more than 3 equal spans.		(1)
3.	*Span* : L = 4.0 metres each = 4000 *mm*		
4.	*Trial Depth* : This is decided by deflection criteria. Basic L/d ratio for continuous slab $r_b = 26$ Assuming p_t = 0.3% for *M20* - Fe 415 for large span Modification factor α_1 = 1.5 using curve for $f_s = 240\ N/mm^2$ Required d = 4000/(1.5 × 26) = 102 *mm* Say 110 *mm* Assuming eff.cover d' = 20 *mm* for Fe415 Required total Depth D = 110 + 20 = 130 *mm*.	*Fig. 4.7.1*	
5.	*Loads* : Consider one metre width of slab (*i.e.* $b = 1m = 1000\ mm$) Dead Load : Self weight + Floor Finish $w_d = 25 \times 0.13 + 1.75 = 5.0\ kN/m$ Live Load : w_L = 1.50 *kN/m* Ultimate load : $w_u = 1.5\ (5 + 1.5) = 9.75\ kN/m$		(2)
6.	*Design Moments* : Bending moments at different sections are calculated using *I.S.Code* coefficients.		



Section	1	2	3	4	Reference	Note
B.M. Coef. for :						
Dead Load α_d	1/12	−1/10	1/16	−1/12	**Table -12 or*	
Live Load α_L	1/10	−1/9	1/12	−1/9	*Table 5.1.1*	(3)
Ult. moments in *kN.m*	13.6	−16	10.5	−14.0		
$A_{st}\ mm^2$	368	440	280	380		

Bending moment at any section is given by :

$$M_u = 1.5\,(\alpha_d\, w_d\, L^2 + \alpha_L\, w_L\, L^2)$$

B.M. at middle of outer span : (*i.e.* at *Section - 1*)

$$M_{u1} = 1.5\ (5 \times 4^2/12 + 1.50 \times 4^2/10) = 13.6\ kN.m$$

B.M at penultimate support : (*i.e.* at *Section - 2*)

$$M_{u2} = -1.5\ (5 \times 4^2/10 + 1.50 \times 4^2/9) = -16.0\ kN.m$$

B.M. at middle of inner span : (*i.e.* at *Section - 3*)

$$M_{u3} = 1.5\ (5 \times 4^2/16 + 1.50 \times 4^2/12) = 10.5\ kN.m$$

B.M. at intermediate support : (*i.e.* at *Section - 4*)

$$M_{u4} = -1.5\ (5 \times 4^2/12 + 1.50 \times 4^2/9) = -14.0\ kN.m.$$

Absolute maximum design B.M. = M_{u2} = 16.0 *kN.m*

Continuous Slab - S3 using I.S. Code coefficients continued....

Step No.	Design Calculations	Reference	Note

7. *Check for Concrete Depth :*

$M_{ur.max} = R_{u.max} \times 1000 \times d^2$. $R_{u.max} = 2.76\ N/mm^2$ for M20 - Fe415. — *Table 4.1.1*

$M_{ur.max} = 2.76 \times 1000 \times 110^2 \times 10^{-6} = 33.4\ kN.m > M_{u2}$ (= 16 $kN.m$) $\therefore$ safe

8. *Main Steel* : This is obtained at 4 different sections using

$$\text{Required } A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times M_u \times 10^6}{20 \times 1000 \times 110^2}}\right] \times 1000 \times 110$$ — *Eq. 6.2.3*

Values of A_{st} calculated at different sections are given in *Step.11*

The diameter-spacing of bars is worked out using *Eq.6.2.4* and shown in *Step-11*

9. *Check for Deflection* :

Required maximum A_{st} at mid-span of penultimate span = A_{st1} = 368 mm^2 — Note (4)

Required maximum p_t = 368 ×100/(1000 × 110) = 0.33% >assumed 0.3%, hence detailed deflection check is carried out.

$(p_t)_{prov}$ = 100 × 387/(1000 × 110) = 0.35%

f_s = 0.58 × 415 × 368/387 = 229 N/mm^2 — *Eq. 4.7.1b*

For p_t = 0.35% and f_s = 229 N/mm^2, α_1 = 1.48 — *Fig .4.7.1*

Required d = 4000 / (26 × 1.48) = 104mm < 110 mm $\therefore$ safe

10. *Distribution Steel* :

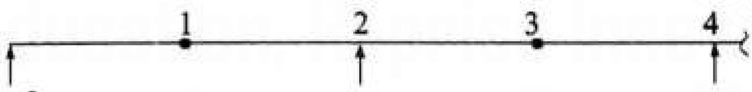
For Fe 415 grade, A_{st} = 0.12% of bD = 0.12 × 1000 × 130/100 = 156mm^2 — *Eq. 5.2.1b*

Provide #8 mm at 320mm giving A_{st} = 156.2mm^2

11. *Detailing of Reinforcement* :

This has been shown below for two different practices of detailing.

Section



	1	2	3	4
Required A_{st} in mm^2	368	440	280	380
I - Conventional Practice :				
Diam. and Pitch in mm	#8@130	#8@260 + #8@350	8@175	#8@350+ #8@350
Provided A_{st} in mm^2	(387)	(337)	(287)	(287)
Required extra A_{st} mm^2	---	103	---	93
# - s of extra bars ---		#8@450	---	#8@450
Provided extra A_{st} mm^2	---	(111)	---	(111)
Total A_{st} provided	387	448	287	398
II - New Practice				
# and s in mm	#8@130	#8@110	#8@175	#8@130
Provided A_{st} in mm^2	(387)	(457)	(287)	(387)

Note (5)

Explanatory Notes to Design of Continuous Slab - S3
Design - I : According to I.S. Code Coefficients :

Note 1 Though every slab panel is supported on all sides, the ratio long span to short span is greater than 2. Therefore, the slab will be designed as one - way slab.

Note 2 Since in continuous slabs, the B.M. coefficients for dead load and live load are different in IS:Code, *DL* and *FF* are added while *LL* is kept separate.

Note 3 It may be observed from the values of B.M. Coefficients that bending moments at all the four sections are different with the maximum occurring at penultimate support. Since Code does not allow redistribution of moments when above coefficients are used, it will be obvious that the area of reinforcement at four sections will be different.

Note 4 In the case of continuous beams the deflection can be maximum at mid-span, therefore, the maximum area of steel (or percentage of stress) at *mid-span* of penultimate support has been taken for deflection check.

Note 5 The different practices used in detailing have been given in *Sect. 5.2.5*

In conventional practice steel is provided at bottom in two adjacent spans according to requirements. Steel at supports at top is obtained partly by bending of alternate bars from the adjacent spans. However, since areas required in inner and outer spans at mid-span are different, spacings of bars in spans or two sides of penultimate supports are different. Therefore, the bars obtained at top of support by bending alternate bars from adjacent spans are not uniformly spaced. Besides, since steel required at supports is always greater than that required at mid-span , the bars taken at top by bending of alternate bars are inadequate. Extra steel required is normally very small it can be of any diameter. However, sometimes the spacing of extra steel is normally matched with that of bent up bars from adjacent spans, though theoretically not so necessary, it reduces the labour in laying the bars.

Alternatively, *different diameters of bars* are used to match the spacing of bars. This practice, many times, necessitates little larger area of steel and labour of bending of bars and use of different bar diameters in two spans. However, from requirements of ease of construction and even identifying use of different diameters of bars in different spans may be difficult and are likely to be skipped while checking the reinforcements. Therefore, this method of detailing is not advisable.

Some practitioners even provide steel all throughout (in all spans and at all supports) equal to maximum of above *i.e.* #8 at 110 *mm* for uniformity of spacing in all spans for ease of construction. (See *Sect.5.2.5* method *IV*). This, however, leads to use of 20% to 30% more steel and hence uneconomical and not recommended.

As mentioned earlier since the shear and development length requirements are satisfied they are omitted.

Design of Slab - S3 : Alternative - II.

Using Exact Analysis and Allowing Redistribution of Moments :

I It may be noted that in exact limit analysis, maximum load consists of *(DL + LL)* for which the load factor is 1.5 and hence $w_{max} = 1.5(DL + LL)$ while the minimum load is *DL* hence $w_{min} = DL$.

II The analysis has been done by two methods :

(a) *Using moment distribution method.*

(b) *Using coefficients given in Handbooks.*

III *Design Moments* : These are obtained by exact analysis allowing redistribution of moments.

Continuous Slab - S3 allowing redistribution of moments continued....

(a) Analysis using Moment Distribution Method.

This requires considerations of two loading cases, one for maximum B.M.at penultimate support and the other for maximum B.M. at intermediate support[7.1]. Every loading arrangement requires adoption of maximum and minimum design loads.

Loading Case - I : Maximum bending moment at penultimate support *B*.
The loading arrangement is shown in *Fig. 7.2.1*

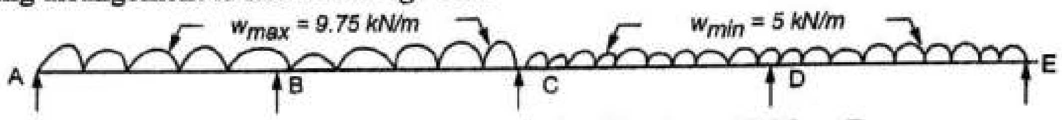


Fig. 7.2.1 Loading Arrangement for Maximum B.M. at B

w_{max} = 1.5 (DL + LL) = 1.5 (25 × 0.13 + 1.75 + 1.5) = 9.75 kN/m

w_{min} = DL = 25 × 0.13 + 1.75 = 5.00 kN/m

Fixed End Moments :

(a) Maximum :

$M_{FBA} = w_{max} L^2/8 = 9.75 \times 4^2/8 = 19.5\ kN.m = M_{FDE}$, $M_{FAB} = M_{FED} = 0$

$M_{FBC} = w_{max} L^2/12 = 9.75 \times 4^2/12 = 13.0\ kN.m = M_{FCB} = M_{FCD} = M_{FDC}$

(b) Minimum :

$M_{FBA} = w_{min} L^2/8 = 5 \times 4^2/8 = 10.00\ kN.m = M_{FDE}$, $M_{FAB} = M_{FED} = 0$

$M_{FBC} = w_{min} L^2/12 = 5 \times 4^2/12 = 6.67\ kN.m = M_{FCB} = M_{FCD} = M_{FDC}$

Distribution factors.

Joint	Member	RSF	SUM	DF
B	BA	3EI/L	7EL/L	3/7
	BC	4EI/L		4/7
C	CB	4EI/L	8EI/L	1/2
	CD	4EI/L		1/2
D	DC	4EI/L	7EI/L	4/7
	DE	3EI/L		3/7

Joint	A	B		C		D		E
Member	AB	BA	BC	CB	CD	DC	DE	ED
D.F.	-	3/7	4/7	1/2	1/2	4/7	3/7	-
FEM	0	19.50	-13.00	13.00	-6.67	6.67	-10.0	0
Bal		-2.79	-3.71	-3.16	-3.17	1.90	1.43	
C.o	-	-	-1.58	-1.86	0.95	-1.58	-	-
Bal		0.68	+0.90	0.45	0.46	0.90	0.68	
C.o			-0.22	0.45	0.45	0.23		
Bal		-0.10	-0.12	-0.45	-0.45	-0.13	-0.10	
Final mmts	0	17.29	-17.29	8.43	-8.43	7.99	-7.99	0

Continuous Slab - S3 allowing redistribution of moments continued....

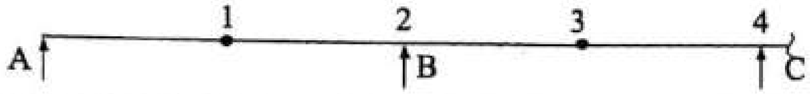
Maximum Span Moments =

Span AB : R_A $= 9.75 \times 4/2 - 17.29/4 = 15.18\ kN$, $x_{max} = 15.18/9.75 = 1.557m$

$M_{max} = 15.18 \times 1.557/2 = 11.82\ kN.m$

Span BC : R_B $= 9.75 \times 4/2 + (17.29 - 8.43)/4 = 21.715\ kN$, $x_{max} = 21.715/9.75 = 2.227\ m$

$M_{max} = 21.715 \times 2.227/2 - 17.29 = 6.89\ kN.m$



M_{EU}	11.82	−17.29	6.89	−8.43
dM 30% at 2	2.59	5.19	2.59	-
M_{DU} CaseI	14.41	−12.10	9.48	−8.43

Loading Case - II : *Maximum B.M. at inner support.*

The loading arrangement is shown in *Fig.7.2.2*

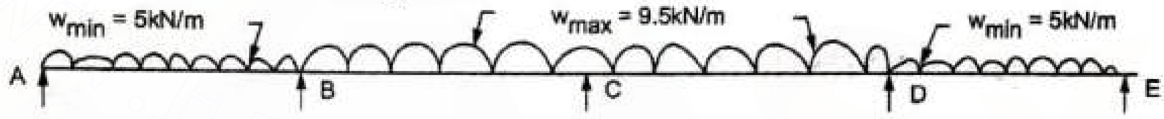


Fig. 7.2.2 Loading Arrangement for Maximum moment at C.

Joint	A	B		C		D		E
Member	*AB*	*BA*	*BC*	*CB*	*CD*	*DC*	*DE*	*ED*
DF	-	3/7	4/7	1/2	1/2	4/7	3/7	-
FEM	0	10.00	-13.00	13.00	-13.00	13.00	-10.00	0
Bal		1.29	1.71			-1.71	-1.29	
C.O				0.85	-0.85			
Final *mmts*	0	11.29	-11.29	13.85	-13.85	11.29	-11.29	0

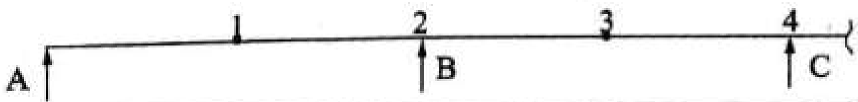
Maximum Span Moments :

Span AB: R_A $= 5 \times 4/2 - 11.29/4 = 7.18\ kN$, $x_{max} = 7.18/5 = 1.435m$

$M_{max} = 7.18 \times 1.435/2 = 5.15\ kN.m$

Span BC : R_B $= 9.75 \times 4/2 - (13.85 - 11.29)/4 = 18.86\ kN$, $x_{max} = 18.86/9.75 = 1.93m$

$M_{max} = 18.86 \times 1.93/2 - 11.29 = 6.91\ kN.m$



M_{EU}	5.15	-11.29	6.91	-13.85
dM = 30% at 4			2.10	4.2
M_{DU} for *Case II*	5.15	-11.29	9.01	-9.65
M_{DU} for *Case I*	14.41	-12.10	9.48	-8.43
Final Design moment	14.41	-12.10	9.48	-9.65

Note : *(1) When design moments are reduced by 30% there will be practically no increase in maximum span moments[7.1]. Hence the loading case for maximum span moment is not considered.*

(2) Once the design moments are obtained the diameter-spacing combination of steel can be obtained from Tables.

(3) The procedure for checking the deflection will be the same as given in Step -9 of Method - I. Therefore, they have not been given in the remaining methods.

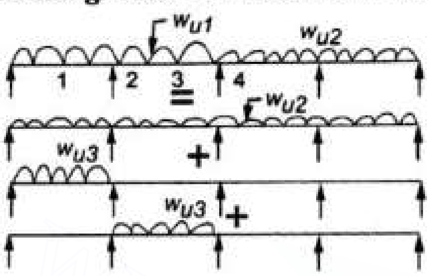
Continuous Slab - S3 allowing redistribution of moments continued....

(b) *Analysis using coefficients* : The final moments can also be worked out by using standard coefficients given in the Handbooks. The *advantage* is that we directly get the B.M. at mid-span of the beams. But it has got the *disadvantage* that the desired loading cases have to be simulated as per the standard cases given in the Handbook. In this case the coefficients given in author's H.B.[7.2] *reproduced in Table D-2a* are used.

$w_{u1} = w_{max} = 9.75\ kN/m$, $w_{u2} = w_{min} = 5.00\ kN/m$, $w_{u3} = w_{max} - w_{min} = 9.75 - 5.00 = 4.75\ kN/m$

$w_{u2} L^2 = 5 \times 4^2 = 80\ kN.m$, $w_{u3} L^2 = 4.75 \times 4^2 = 76\ kN.m$

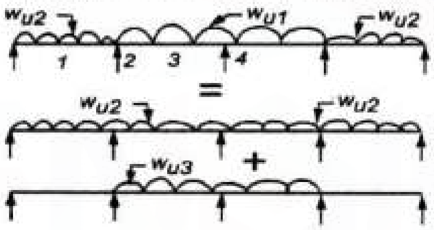
Loading case - I : Maximum Bending moment at Penultimate Support. The loading is shown on left



	1	2	3	4
α_2	.077	-0.107	0.036	-0.071
α_3	0.094	-0.067	-0.025	+0.018
α_4	-0.025	-0.049	0.074	-0.054
M_{EU}	11.40	-17.38	6.60	-8.42
dM = 30% at 2	2.61	5.21	2.61	-
Design moment in *kN.m*	14.01	-12.17	9.21	-8.42

$M_{EU} = \alpha_2 w_{u2} L^2 + (\alpha_3 + \alpha_4) w_{u3} L^2$
$= \alpha_2 \times 80 + (\alpha_3 + \alpha_4) \times 76$

Loading case - II : Maximum Bending moment at inner Support. *(see Table D-2)*



	1	2	3	4
α_2	0.077	-0.107	0.036	-0.071
α_3	−0.018	-0.036	0.056	-0.107
$M_{EU} = \alpha_2 w_2 L^2 + \alpha_3 w_{u3} L^2 = \alpha_2 \times 80 + \alpha_3 \times 76$ M_{EU}	4.79	-11.30	7.14	-13.81
dM = 30%at 4			2.07	4.14
Design Moment *Case - II*	4.79	-11.30	9.21	-9.67
Design Moments *Case - I*	14.01	-12.17	9.21	-8.42
Final Design Moments As per coef. method M_u	14.01	-12.17	9.21	-9.67
As per Moment Distribution (M.D.)method	14.41	-12.10	9.48	-9.65
Required area of steel mm^2 as per M.D. method	392	325	251	253
No-Diameter combination	#8@125	#8@250+#8@380	#8@190	#8@380+#8@380
Area of steel provided mm^2	402	333	265	265

Comments on Design of Slab S3 : *Maximum of 30% redistribution of moment has been carried out as allowed by Cl.37.1.1(c) of the code. This means reduction of support moment by δM=30% and increasing the mid-span moment approximately by δM/2 in both adjacent spans.This helps to get span moment greater than support moment thereby avoiding requirement of any extra steel at support, when the method of alternate bent up bars from mid-spans to support are used.(see Sect. 5.2.5(i)).*

For soldier's practice of detailing (see Sect. 5.2.5(ii)) it is necessary to equalise to bending moment at support and at mid-span, so that the requirement of area of steel at mid-span and support sections remains the same. For this, reduce the support moment by 2/3 of absolute difference between span moment and support moment and increase the span moment by 1/3.In this case maximum span moment for the alternative span loaded with w_{max} *and* w_{min}, *the maximum span moment in AB works out to 13.4 kN.m. The maximum support moment (without redistribution) is -17.29 kN.m.*

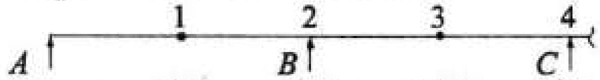
$\therefore$ *Absolute difference in moments* $= 17.29 - 13.4 = 3.89\ kN.m$

$\therefore$ *Revised support moment* $= 17.29 - 3.89 \times 2/3 = 14.7\ kN.m$ *and*

Revised span moment $= 13.4 + 3.89/3 = 14.7\ kN.m.$

Slab - S3 Alternative Design - III : Design using Approximate Method (see Table D-5)

Step No.	Design Calculations	Reference	Note
1.to 5	*Same as in Design - I*		
6.	*Design Moments* : It is common practice with many designers to take the design moment of $\pm w_u L^2/10$ in outer spans and $\pm w_u L^2/12$ in inner span for simplicity. The ultimate moments M_u and area of steel worked out as under : $w_u = 9.75\ kN.m$ $w_u L^2 = 9.75 \times 4^2 = 156\ kN.m$		



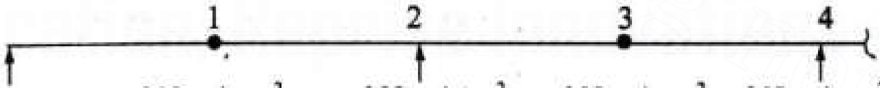
	1 (A–B)	2 (B)	3	4 (C)	Reference
B.M. Coefficient α	1/10	-1/10	1/12	1/12	Sect.3.3.2
M_u in $kN.m$ = 156 x α	15.6	-15.6/14.3	13.0	-13.0	
Required A_{st} in mm^2	428	428/389*	351	351	Eq.6.2.3

Note : Values marked with asterisk (*) are for steel at support equal to average of the values at Sect. 1 and Sect. 2 , while those without asterisk are for greater of the two moments. When Conventional practice of bar detailing of cranking alternate bars from adjacent spans is to be followed, it is convenient to take average of the two moments.

Slab S3 : Alternative Design IV - Design using Very Approximate method.

Step No.	Design Calculations	Reference	Note
1 to 5	*Same as design - I*		
6	*Design Moments* : The maximum approximate moment being $w_u L^2/10$, the same is provided at all mid - spans as well as at all supports. The required area of steel will be 428 mm^2 at all sections.	Eq. 6.2.3	

Comparison of Bending Moments Area of steel by Different Methods:



	1	2	3	4
	$kN.m/mm^2$	$kN.m/mm^2$	$kN.m/mm^2$	$kN.m/mm^2$
Case-I Using I.S.Code coefficients	13.6/368	-16.0/440	10.5/280	-14.0/380
Case-II Exact Method with 30% redistribution	14.41/392	-12.10/325	9.48/251	-9.65/255
Case - III Approximate Method	15.6/428	-15.6/428	13.0/351	-13.6/351
Case - IV Very Approximate method	15.6/428	-15.6/428	15.6/428	-15.6/428

Area of steel has been obtained using formula :

$$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times M_u \times 10^6}{20 \times 1000 \times 110^2}}\right] \times 1000 \times 110 \qquad \text{... ... Eq. 6.2.3}$$

Comments : *A comparison of different methods of design will reveal that Design II using exact method of analysis and allowing redistribution of moments is most economical method of design. However, for hand computation Design - III based on approximate method of analysis is simple. It saves time, computational efforts, and errs on the safer side.*

The very approximate method is too much over-safe. The only advantage is that by bending alternate bars at supports the requirements of steel at support is met with and there is no need to provide extra steel at support, hereby saving in labour.

7.2.4 Two-way Continuous Slab S4

Step No.	Design Calculations	Reference	Note
1.	*Slab Mark* : *S4*		
2.	*Type* : Two -way Continuous with Corners restrained.		
3.	*Spans* : Short Span $L_x = 4.5m$, Long Span $L_y = 5\ m$. Aspect ratio $L_y / L_x = 5.0/4.5 = 1.11$., $LL = 1.5\ kN/m^2$, $FF = 1.75\ kN/m^2$		
4.	*Trial Depth* : In case of a two - way slab, *L/d* ratio for deflection criteria is related to short span.Since short span L_x in this case = 4.5 *m* is greater than 3.5*m* (even though live load is less than $3\ kN/m^2$), deflection requirement is governed by *Clause 23.2.1*.instead of *Clause 24.1*. In the case of two-way slabs, the design moments are small compared to those in one-way slabs, percentage of steel required in two-way slabs ,in general, is very low (less than 0.25% for M20 - Fe415). Since even the live load is also small, only 0.2 % steel will be assumed.	Sect.6.2.3	
	For $p_t = 0.2\%$, $\alpha_1 = 1.7$ corresponding to $f_s = 240\ N/mm^2$	Fig.4.7.1	
	Basic *L/d* ratio $r_b = 26$ for continuous slab.	Sect.4.7.1	
	Required $d = L_x/(\alpha_1 \times r_b) = 4500/(1.7 \times 26)$ = say 110 *mm* Assuming $d' = 20\ mm$ for Fe 415, Required $D = 110 + 20 = 130 mm$. Effective depth to the outer layer of bars $d_o = 110mm$, Effective depth to the inner layer of bars $d_i = 100mm$.		
5.	*Loads* : Consider one metre width of slab. $w_u = 1.5\ (25 \times 0.13 + 1.75 + 1.5) = 1.5 \times 6.5 = 9.75\ kN/m^2$		
6.	*Design Moments* : Boundary Condition No. = 4. Discontinuous on two adjacent edges. $w_u L_x^2 = 9.75 \times 4.5^2 = 197.4\ kN.m$	Table D-7	(1)

Span Position	B.M.Coef. (α)	$M_u = \alpha\, w_u L^2_x$ kN.m
Short - Support	0.0537	.0537 × 197.4 = 10.60 *kN.m*
-Midspan	0.0405	.0405 × 197.4 = 8.00 *kN.m*
Long - Support	0.047	.047 × 197.4 = 9.28 *kN.m*
- Midspan	0.035	.035 × 197.4 = 6.91 *kN.m.*

Step No.	Design Calculations	Reference	Note
7.	*Check for Concrete Depth* : $M_{ur.max} = R_{u.max}\, bd^2$. For slabs, $b = 1000\ mm$ $R_{u.max} = 2.76\ N/mm^2$ for M20 - Fe 415 For outer bar, $M_{ur.max} = 2.76 \times 1000 \times 110^2 \times 10^{-6} = 33.4\ kN.m > 10.6\ kN.m$. ∴ safe For inner bars, $M_{ur.max} = 2.76 \times 1000 \times 100^2 \times 10^{-6} = 27.6\ kN.m$		
8.	*Main Steel* : Required $A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times M_u \times 10^6}{20 \times 1000 \times d^2}}\right] \times 1000 \times d$	Eq.6.2.3	

Two-way Continuous Slab S4 continued....

Step No.	Design Calculations	Reference	Note
	Location / M_u (kN.m) / d (mm) / Reqd. A_{st} (mm^2) / Diam.-Spacing (mm mm) / Prov. A_{st} (mm^2)		
	Along Short span - Support: 10.60, 110, 282, # 8 @ 175, 287		(2)
	- Midspan: 8.00, 110, 210, # 8 @ 240, 209		
	Along Long span - Support: 9.28, 110, 245, # 8 @ 200, 251		(3)
	- Midspan: 6.91, 100, 200, # 8 @ 250, 201		

Location	M_u kN.m	d mm	Reqd. A_{st} mm^2	Diam.-Spacing mm mm	Prov. A_{st} mm^2
Along Short span - Support	10.60	110	282	# 8 @ 175	287
- Midspan	8.00	110	210	# 8 @ 240	209
Along Long span - Support	9.28	110	245	# 8 @ 200	251
- Midspan	6.91	100	200	# 8 @ 250	201

Distribution Steel : Provide # 8 at 320 *mm* same as for slab *S3* (4)

9. *Check for deflection :*

Required p_t at mid-span of short span $=210\times100/(1000\times110) = 0.19\% < 0.2\%$ (5)

Since $(p_t)_{reqd.} < (p_t)_{assumed}$, check for deflection is satisfied.

10. *Check for Shear :* $\beta = L_y/L_x = 5.0/4.5 = 1.11$

(a) (i) *Long Edge - Continuous :*

$V_{u.max} = 1.2 \times w_u L_x [\beta/(2\beta+1)]^{7.3}$

$V_{u.max} = 1.2 \times 9.75 \times 4.5 \times [1.11/(2\times1.11+1)] = 1.2 \times 15.12 = 18.15\ kN.$

$A_{st} = 287\ mm^2$, $p_t = 100\times287/(1000\times110) = 0.26\%$, $k = 1.3$ — Table4.4.2

$\tau_{uc} = 0.36 N/mm^2$, $V_{uc} = k\,\tau_{uc}\,bd.$ $k = 1.3$ for $D < 150\ mm$ — Table4.4.1

$V_{uc} = 1.3\times0.36\times1000\times110/1000 = 51.5\ kN > 18.15$ ∴ safe

(a) (ii) *Long Edge-Discontinuous* : $V_{u.max} = 0.9\times15.12 = 13.61\ kN$

$A_{stx} = 209\ mm^2$ at midspan.

Assuming 50% bent up to resist moment due to partial fixity,

$A_{st1} = 104.5\ mm^2$, $p_t = 100\times104.5/(1000\times110) = 0.095\%$

$\tau_{uc} = 0.28\ N/mm^2$, $k = 1.3$

$V_{uc} = 1.3\times0.28\times1000\times110/1000 = 40\ kN > 30.61\ kN$ ∴ safe

(b) (i) *Short Edge - Continuous* : $V_{u.max} = 1.2\,w_u\,L_x/3$

$V_{u.max} = 1.2\times9.75\times4.5/3 = 1.2\times14.6 = 17.5\ kN$

$A_{st} = 251\ mm^2$, $p_t = 0.23\%$, $\tau_{uc} = 0.34\ N/mm^2$

$V_{uc} = 1.3\times0.34\times1000\times110/1000 = 48.6\ kN$

$V_{uc} = 48.6\ kN > 17.5\ kN$ ∴ safe

(b) (ii) *Short Edge-Discontinuous* : $V_{u.max} = .9\times14.6 = 13.14\ kN$

$A_{sty} = 201\ mm^2$ at mid-span

Assuming 50% bent up to resist moment due to partial fixity,

$A_{st1} = 100\ mm^2$, $p_t = 100\times100/(1000\times110) = 0.1\%$ ∴ $\tau_{uc} = 0.28\ N/mm^2$, — Table4.4.1

$V_{uc} = 1.3\times0.28\times1000\times110/1000 = 40.0\ kN > 13.14\ kN.$ ∴ safe

11. *Check for Development Length :*

(a) (i) *Long Edge - Continuous* : Required $L_d = 47\times8 = 376\ mm$

Available $L_d = 0.3L$ of slab $S3 = 0.3 \times 4m = 1200\ mm > 376\ mm$ ∴O.K.

(a) (ii) *Long Edge - Discontinuous* : $L_d = 47\times8 = 376\ mm$

Assuming 50% bars bent up, $M_1 = 8.0/2 = 4.0\ kN.m.$, $V = 30.61\ kN.$

$L_o = 230/2 - 25 + 3\times8 = 114\ mm$ — Eq.4.6.4a

Two-way Continuous Slab S4 continued....

Step No.	Design Calculations	Reference	Note
12.	L_d = 376 *mm* < 496 *mm* (=1.3 × 4 × 1000/13.61 + 114) ∴ Safe. (b) *(i) Short Edge - Continuous :* Required L_d = 47 × 8 = 376 *mm* Available $L_d = 0.3L_y$ = 0.4 x 5000 = 2000 *mm* ∴ Safe. (b) *(ii) Short Edge - Discontinuous :* L_d = 47 × 8 = 376 *mm* Assuming 50% bars bent up, M_1 = 6.91/2 = 3.5 *kN.m.* , V = 13.14 *kN.* L_d = 376 *mm* < 460 *mm* (=1.3 × 3.5 × 1000/13.14 + 114) ∴ Safe. *Torsion Steel :* (a) At corners near columns *C12* and *C20*, since slab is discontinuous over both the edges, full torsion steel equal to 0.75 A_{stx} = 0.75 × 210 = 158 mm^2 will be required in both directions at right angles in each of the two meshes, one at top and the other at bottom for a length = $L_x/5$ = 4500/5 = 900 *mm*. (b) At corners near columns *C11, C14*, and *C19*, required area of torsion steel is just half of torsion steel in (a) above since the slab is discontinuous over only one of the two edges at these corners.	*Eq. 4.6.3b*	(6)

Explanatory Notes on Design of Slab S4 :

Note 1. It may be noted that the major portion near the stairs is discontinuous at the edge of beam *B10*, there being staircase steps on the other side. Therefore, the boundary condition corresponds to that of *Case - 4* of *Table D -7*.

Note 2. The slab *S4* is continuous with slab *S3* over beam *B22-B23*. Therefore, A_{st} to be provided at support will be greater of the two steel areas calculated for two slabs, *i.e.* 282mm^2 for *S4* and for *S3*, it is 380 mm^2 according to alternative - I : Using IS Code coefficients.

In the case of conventional practice of bar detailing, alternate bars from slabs *S3* and *S4* will be bent up and extra steel provided, if required, to meet the difference between the required area and the area available by alternate cranking of bars from adjacent spans.

Note 3. In conventional practice of bar detailing, steel at support over beam *B11* will be obtained by bending alternate bars (#8 at 500*mm*) from mid-span of long span to common support *B*11 to give 201 mm^2 and balance 245–201 = 44 mm^2 provided by way of extra steel. In practice, many times, the steel required at support *i.e.* 8*mm* at 200*mm* is provided at midspan also so that the same steel can be obtained at supports by cranking of alternate bars from two adjacent spans.

Note 4. Main steel A_{stx} and A_{sty} is theoretically required to be provided only in middle strips of widths 0.75L_y and 0.75L_x respectively. Consequently, there is no steel parallel to the edges in edge strips of width *L*/8. Therefore, to hold main bars which are transverse to the edges, distribution steel shall be provided at top and bottom in this edge strips.

Note 5. For serviceability, p_t shall be the steel at middle of short span.

Note 6. Torsion steel is provided at the corners in one of the following ways.

(i) Two meshes of size 900 *mm* × 900 *mm* (=L_x /5), with # 8 bars at 300 *mm* (giving A_{st} = 167 mm^2 > 0.75 × 201) laid othogonally in each of the two layers, shall be provided at each corner.

(ii) Provide main steel in both long span and short span through out (instead of providing only in the middle strip) and continue all bars at bottom right up to the slab and within a strip of L_x /5 each way and then bending them back through 180° like a U-fork and continue them at top for a length L_x/5. By this arrangement, available $A_T = A_{stx}$ in one direction and A_{sty} in the another direction in each of the two meshes at top and bottom instead of 0.75 A_{stx} required. However, since A_{sty} is usually greater than 0.75 A_{stx}, the requirement of torsion steel is met with. At corners where torsion steel is required to be only 3/8th A_{stx}, only alternate bars in strips L_x/5 will be continued and bent back in the form of a U-fork. This arrangement requires less labour and supervision during laying of bars. Besides it is economical too.

7.2.5 Design of Slab - *S5* : Cap Slab Over Staircase

Step No.	Design Calculations	Reference	Note

1\. *Slab Mark : S5*

2\. *Type* : Two-way, Simply Supported, Single Panel, Corners free to lift — Note (1)

3\. *Spans* : Short $L_x = 2.5\ m$, Long Span $L_y = 3.26\ m$. — Note (2)

Aspect ratio $= L_y/L_x = 3.26/2.5 = 1.3$

4\. *Trial Depth* :

Since short span is less than 3.5 metres and live load is less than 3 kN/m^2,depth will be governed by deflection requirements of L/D ratio — Reference: *Clause24.1; Note (2)

Allowable L/D ratio = 35 for simply supported slab for steel of grade Fe250 and 0.8 × 35 = 28 for steel of grade Fe 415. — Reference: Table 6.2.2

∴ Required total depth $D = L_x/28 = 2500/28 = 90\ mm$.

Provide minimum depth of 100 mm.

Assuming $d' = 20\ mm$ for Fe 415, effective depth for :

- short span $d_o = 100 - 20 = 80\ mm$, and for
- long span $d_1 = 80 - 10 = 70\ mm$

5\. *Consider 1m width of slab*

Self weight	$= 25 \times 0.1$	$= 2.50\ kN/m$
Floor finish		$= 1.75\ kN/m$
Live load		$= 0.75\ kN/m$
Total working load		$= 5.00\ kN/m$
Ultimate load	w_u	$= 1.5 \times 5 = 7.5\ kN/m$

6\. *Design Moment* : $M_u = \alpha\, w_u L_x^2$, $w_u L_x^2 = 7.5 \times 2.5^2 = 46.875\ kN.m$

Span	B.M. coef. (α)	B.M. M_u in $kN.m$
Short	0.093	$0.093 \times 46.875 = 4.36\ kN.m$
Long	0.055	$0.055 \times 46.875 = 2.58\ kN.m$

Reference: Table D-6

7\. *Check for Concrete Depth* :

$M_{ur.max} = R_{u.max}\, bd^2$. For slabs , $b = 1000\ mm$.

$R_{u.max} = 2.76\ N/mm^2$ for M20 - Fe 415

$M_{ur.max} = 2.76 \times 1000 \times 80^2 \times 10^{-6} = 17.7\ kN.m > 4.36\ kN.m.$ ∴ Safe

$M_{ur.max} = 2.76 \times 1000 \times 70^2 \times 10^{-6} = 13.5\ kN.m > 2.58\ kN.m.$ ∴ Safe

8\. *Main Steel* : This is obtained for both spans using formula

$$\text{Required } A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times M_u \times 10^6}{20 \times 1000 \times d^2}}\right] \times 1000 \times d$$

Location	M_u kN.m	d mm	Reqd.A_{st} mm^2	Diam. mm	Spacing mm	Prov.A_{st} mm^2
Along Short Span	4.36	80	160	# 8	@ 240*	209
Along Long Span	2.58	70	107	# 8	@ 210*	239

* The maximum spacing is governed by 3 d — Note (3)

This steel will be provided throughout the width of slab and not only in middle strips as in case of Slab - *S4*.

Explanatory Notes for Design of Slab - S5

Note 1. The slab over the staircase room is simply resting on four walls 230*mm* wide on all the four sides. There are no beams supporting this slab. As such the corners are considered to be unrestrained against torsion and hence taken as free to lift.

Note 2. Long span of this slab is governed by the position of the supporting short wall which in turn depends upon the position of the beam supporting flight - II of the staircase. The supporting beam has been provided immediately at the end of going.
Therefore, the long span for slab = going + landing + wall thickness = 2250 + 780 + 230
= 3260 *mm* = 3.26 *m*.

Note 3. The steel required for resisting moment for both the spans is too less. The spacing is governed by maximum permissible spacing of 3 *d*. Theoretically, even 6 *mm* bars of grade Fe250 would have been sufficient. However, they are not used for main steel in practice as they do not have any rigidity for working and hence not favoured as main steel.

7.2.6 Design of Stair-case

Step No.	Design Calculations	Reference	Note
	Data : Width of stairs = 2270/2 − 135 = 1000 *mm*		
	Staircase room 3030 *mm* × 2270 *mm*.		
	Floor to floor height H = 3.2 *m* = 3200 *mm*		
	Live Load for assembly buildings = 5 kN/m^2	*Table A-2*	
	Functional Design : Let Rise R = 160*mm* and Tread T = 250 *mm*.	*Sect 1.3.4*	
	Sec ϕ = $\sqrt{250^2 + 160^2}$ / 250 = 1.187		
	No. of risers required = H/R = 3200/160 = 20		
	No. of risers in each of the two flights = 10		
	No. of Treads per flight = 10 − 1 = 9.		
	Going = 250 × 9 = 2250 *mm*.		
	Width of landing at end = 3030 − 2250 = 780 *mm*	*Fig.7.1.2*	
	Flight - I is supported on the floor at bottom and on beam *B26* at mid-landing level.		
	Span L = 2250 + 780 + 230 = 3260 *mm* horizontally.		
	Flight - II is simply supported on beam *B26* at mid-landing level at bottom and simply supported on floor beam *B27* at top.		
	Span L = 3260 *mm* horizontally		(1)
	Design of Flight - I		
1.	*Type* : One - way single span, simply supported inclined slab.		
2.	*Span* : 3.26 *m* horizontally = 3260 *mm*.		
3.	*Trial Depth* : (*i.e* Waist thickness of slab)		
	Basic L/d ratio r_b = 20 for simply supported slab.		
	Assuming p_t = 0.4% since the load is heavy on stairs.		
	Modification factor α_1 = 1.32 for f_s = 240 N/mm^2	*Fig. 4.7.1*	
	Required effective depth $d = L/(\alpha_1 \times r_b)$=3260/(1.32 x 20) say 130 *mm*		
	Assuming d' = 20 *mm* for Fe415, provide D = 130 + 20 = 150 *mm*		
	(Normally, trial depth for stairs may be taken between $L/20$ to $L/25$).		
4.	*Loads* : Self weight : w_d = 25D × Sec ϕ = 25 × 0.15 × 1.187 = 4.45 kN/m^2		
	Weight of steps = 25R/2 = 25 × 0.16/2 = 2.00 kN/m^2		

Design of Stair-case continued...

Step No.	Design Calculations	Reference	Note
	Live load $= 5.00\ kN/m^2$ Floor finish $= 1.00\ kN/m^2$ Total working load $w = 12.45\ kN/m^2$ Total design load $w_u = 1.5 \times 12.45 = 18.68\ kN/m^2$		(2)
5.	*Design Moment* : Consider of slab 1 m width , $w_u = 18.68\ kN/m$ $M_{u.max} = w_u L^2/8 = 18.68 \times 3.26^2/8 = 24.8\ kN.m$		
6.	*Check for Concrete Depth* : $M_{ur.max} = 2.76 \times 1000 \times 130^2 \times 10^{-6}$ $= 46.6\ kN.m > M_u\ (= 24.8\ kN.m)\ \therefore$ safe		
7.	*Main Steel* : Required $A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 24.8 \times 10^6}{20 \times 1000 \times 130^2}}\right] \times 1000 \times 130$ $= 583\ mm^2$ Provide # 10 mm at 130 mm c/c , Area provided $= 604\ mm^2$		
8.	*Check for deflection* : $(p_t)_{reqd} = 100 \times 583/(1000 \times 130) = 0.448\% > (p_t)_{assumed}\ (= 0.4\%)$ $\therefore$ Detailed check for deflection has been carried out Steel stress of service $f_s = 0.58 \times 415 \times 583/604 = 232\ N/mm^2$ $(p_t)_{prov} = 100 \times 604/\ (1000 \times 130) = 0.46\%$ for $f_s = 232\ N/mm^2$ and $p_t = 0.46\%$, $\alpha_1 = 1.31$ Required $d = 3260/(1.31 \times 20) = 125\ mm < 130\ mm\ \therefore$ safe.	Fig.4.7.1	
9.	*Distribution steel* : For Fe415 , $p_{t.min} = 0.12\%$. $A_{st} = 0.12 \times 1000 \times 150 / 100 = 180\ mm^2$ Provide # 8 mm at 275 mm c/c giving $A_{st} = 182\ mm^2$		
	Design of Flight - II		(3)
1.	*Type* : One - way, Simply supported at one end (B26) and also simply support at the other end (B27).		
2.	*Span* : 3.26 metres horizontally. (3260 mm)		
3.	*The design will be the same as for flight - I*		

Explanatory Notes for Staircase Design

Note 1. The guide lines for fixing the dimensions of component parts of stairs have been given in *Sect.1.3.4*. These should be taken into considerations for fixing the rise, trade, slope and width of the staircase.

Note 2. Theoretically, the load on landing portion of the flight consists of horizontal slab excluding the weight of step $= 1.5(25 \times 0.15 \times 1.187 + 1 + 5) = 15.68\ kN/m^2$ instead of $18.68\ kN/m^2$. Thus load distribution along the length of the flight is nonuniform necessitating determination of end reactions, location of the point of zero shear *i.e.* the point of maximum bending moment and maximum span moment from first principles. Since a small reduction in load and that too over a small length near the support reduces the maximum span moment hardly by 5% to 10% and support moment only up to 5%.This saving is not enough to justify the cost of extra time and efforts required for rigorous calculations. Therefore, the whole span is considered to be loaded by UDL of 18.68 kN/m^2. It is considered that simplicity accompanied by additional safety is preferable to apparent marginal economy in practical design.

Note 3. There are two more alternatives for supporting flight - II.

(a) In first alternative, the beam *B27* at top can be omitted and stair may be directly supported on the slab *S2* which spans parallel to the risers (*i.e.* perpendicular to the flight) across beams *B5* and *B10*. This will cause increase in span and corresponding increase in the required depth of the flight - II. Besides, the slab *S2* which will act as supporting wide beam for the flight will be required to be designed to carry the load from flight - II in addition to the load of the cross wall at the end of flight - II. This will require very heavy reinforcement in that strip. The solution may work out to be uneconomical.

(b) The second alternative will be to provide beam *B27* across the columns *C5* and *C11* and support the flight - II on the same. This will not only increase the span and hence depth of the flight but it will also increase the size of the stair room from 2270 *mm* × 3030 *mm* to 2270 *mm* × 4270 *mm* resulting in increase in length of side long walls and of size of slab *S5*. But analysis of beams *B5* and *B10* will get simplified because now there will be no point load from *B27* and UDL will be over the entire length of the beam rather than over only part of the span as it is in the present solution. However, this solution is uneconomical.

7.2.7 Schedule of Slabs : The schedule of slabs is given in *Table 7.2.1*

Table 7.2.1 : Schedule of Slabs

Slab	*Total Depth*	*Reinforcement*								*Type*
		Along Short Span		*Along Long Span*		*Over-Support*				
						Penultimate Support Along short span		*Interior support Along long span*		
	mm	*Diam mm*	*Spacing mm*	*Diam mm*	*Spacing mm*	*Diam mm*	*Spacing mm*	*Diam mm*	*Spacing mm*	
S1	100	# 8 @ 240		ϕ6 @ 180		--		--		Cantilever
S2	110	# 8 @ 200		ϕ6 @ 160		--				One way S.S
S3	130	(a) Middle of End Span # 8 @ 130 (b) Middle of Interior Span # 8 @ 175		#8 @ 320 (Distribution steel)		#8@260+8@350 +#8@450(extra)		-- - #8@350+#8@350 + #8@450(extra)		One-way Continuous
S4	130	# 8 @ 240		#8 @ 250		#8 @ 175		#8 @ 200		Two-way Continuous
S5	100	# 8 @ 240		#8 @ 210		---		---		Two-way S.S
Stairs	150	# 10 @ 130		#8 @ 275		---		---		One way S.S

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7.3 DESIGN OF BEAMS

7.3.1 Categorization and Grouping of Beams

As explained in *Sect. 6.3.2*, the following assumptions and approximations have been made prior to design of beams.

Since, the structure is just a single storeyed structure, the component members are assumed to be simply connected except at the continuous ends of continuous beams. Each span of a continuous beam has been treated as independent with continuity at continuous end and simply supported end condition at the discontinuous end.

Accordingly, beams *B26, B27, B11, B19, B20* are assumed to be simply supported at ends.Since the analysis of a 5 span continuous beam *B1 - B2 - B3 - B4 - B5 - B6 and B7 - B8 - B9 - B10*, and *B12 - B13 - B14 - B15 - B16* with approximately equal spans and unequal light *UD* loads and point loads, is laborious and time consuming, it is sufficient enough to treat each of the above beams as made up of :

(a) Beams *(e.g. B1, B6, and B12)* as independent beam, simply supported at left end and continuous at the other and carrying only *UD* load;

(b) Intermediate equal span beams *(e.g. B2 - B3 - B4; B7 - B8 - B9*; and *B13 - B14 - B15)* each one continuous at both ends, and

(c) Beams *(e.g. B5, B10, B16)* to be continuous at left end and simply supported at the other.

Even though, spans and end conditions of these beams are the same, beam *B16* does not carry any point load but only triangular load which can be converted into equivalent UD load. Therefore, it will be considered in a separate group in Category-II while beam *B5* and *B10* will be categorised separately as they carry point loads as well. The beams *B17* and *B18* is considered as a two-span continuous beam with unequal spans.It is also designed with different alternatives giving comments.

Beams *B22-B23* and *B24-B25* are both continuous beams with two equal spans. They will, therefore, be put under one category. However, design of two pairs will be separate because the loads on them are different.

Thus, beams have been categorised on the basis of their *end conditions*, *loading*, and *section type* as shown in table below :

Category No.	*EC No.*	*Group No.*	*Beam Mark*	*Load Type*	*Section Type*	*Span m*	*Explanatory Note No.*
I	1	(a)	*B26*	UDL	Rectangular	2.5	1, 2, 3
		(b)	*B27*	UDL	Rectangular	2.5	3
		(c)	*B11*	UDL	Flanged	4.5	4
		(d)	*B19,B20,B21*	UDL	Flanged	10.0	4
II	2	(a)	*B1*	UDL	Rectangular	4.0	1, 5
		(b)	*B6*	UDL	Rectangular	4.0	1, 5
		(c)	*B12*	UDL	Rectangular	4.0	1, 5
		(d)	*B16*	UDL	Flanged	4.5	1, 5
III	3	(a)	*B2, B3, B4*	UDL	Rectangular	4.0	1, 6
		(b)	*B7, B8, B9*	UDL	Rectangular	4.0	1, 6
		(c)	*B13,B14,B15*	UDL	Rectangular	4.0	1, 6
IV	4	(a)	*B22-B23*	UDL	Flanged	5.0	7
		(b)	*B24-B25*	UDL	Flanged	5.0	8
		(c)	*B17-B18*	UDL	Flanged	10and2.5	9
		(d)	*B5*	UDL+PTL	Rectangular	4.5	10
		(e)	*B10*	UDL+PTL	Flanged	4.5	10

Since, the objective of this first project is to explain the design procedure using first principles for the benefit of beginners, specimen design calculations are given for one beam in each category as follows :

I : B26, B27, B19; II : B1; III : B7 - B8 - B9; IV : B22-B23,B17-B18, and B5.

Design of the other remaining beams is presented without the use of design aids in a brief format as well as using design aids in a tabular format which outlines the algorithm for computer solution.

Explanatory Notes to Categorisation Tables :

Note 1. All beams bearing this Explanatory Note No.1 have slabs either on one or both sides. They could have been designed as flanged beams by ensuring that the condition of minimum transverse steel required in the slab for flange action is satisfied. However, as explained in *Sect. 3.3.3.*, since spans are small and loading is also light, these beams are designed treating them as rectangular beams.

Note 2. Beam *B26* is at mid-landing. It is discontinuous at both ends. No rigid connection is provided at the ends and it is, therefore, considered as a beam simply supported at both ends.

Note 3. Beam *B27* is supported by beams *B10* and *B5*. The ends of this beam are simply connected to supporting beams. Rigid connection is not provided to avoid development of torsion in supporting beams. Therefore, it is considered as beam simply supported at both ends. According to *Note-1* above flange action has been disregarded for this beam also. However, if flange action is required to be utilized, it may be noted that there is a landing slab for full length of beam on one side while on the other side there is an inclined stair slab only over length equal to the width of flight. It will not, therefore, be considered to act as *T*-beam but the beam can be treated as *L* beam provided, of course, the condition of minimum transverse steel is satisfied.

Even though beams *B27* and *B26* are of the same type (rectangular), equal spans, same end conditions and same type of loading (UDL), they are put under different subgroups because loading due to stair on *B26* is from both *flights I and II* acting from one side while load on *B27* is from stair *flight II* only and that too over part of the length. The slab *S2* on other side is actually not supported on beam *B27*. It spans across *B5* and *B10* beam. Consequently, it does not transfer any load to beam *B27* under consideration if no transverse steel is provided at top between slab *S2* and beam *B27*. However, if certain minimum steel is provided at top in *S2* and beam *B27* to avoid cracking, a load over a triangular area with equivalent UDL = $w_u L_x / 6$ is assumed to be transferred to beam *B27*, where w_u is the intensity of load on *S2* per sq.m and L_x is the span of *S2*.

Note 4. Beams *B11, B19, B20, B21* are all discontinuous at both ends and hence assumed as simply supported at both ends. Since load on them is heavy they are designed as flanged *(T)* beams. Since span of *B11* is different from that of *B19*, it is subgrouped separately. On the contrary, *B19, B20* and *B21* being identical in all respects they have been grouped together.

Note 5. All beams in *Category II* are of the same type (rectangular excepting *B16*), having equal spans, same end condition and same type of loading (UDL). However, they are subgrouped further as 2(a), 2(b) and 2(c) because magnitudes of loads on beams in these three subgroups are different. Beam *B1* has load only from *S2* besides self weight and parapet load, while beam *B6* has no parapet load but *UD* load from *S2* and part triangular load from *S3*. Even though, slab *S3* spans parallel to *B6*, a load over a triangular portion having equivalent UDL equal to $w_u L_x/6$ is considered to be transferred to beam *B6* due to provision of minimum transverse steel across *B6* to avoid cracking. *B12* has parapet load and a triangular load from slab *S3* as in case of beam *B6*. Beam *B16* has totally different span and load and hence sub-grouped separately.

Note 6. Beams in *Category - III* are same as those in *Category - II* except the difference in end conditions. Therefore, they are also subgrouped separately on the lines of beams in *Category II*.

Note 7. Beams *B22, B23* form a two span continuous beam on which the loads are transferred by slabs *S3* and *S4* from two sides. The beam acts as rectangular beam at intermediate support and as *T* - beam at midspan.

Note 8. From the plan, it may appear that beams *B25* and *B26* can be considered to be continuous. However, it may be noted that beam *B26* is at mid landing level while *B25* is at floor level, and hence it is not continuous at all beyond column *C12*. Therefore, *B25* is treated as discontinuous and simply supported over column *C12*. However, beams *B24* and *B25* are continuous having slab *S4* only on one side and spanning transversely. It is, therefore, designed as a two span continuous beam acting as rectangular beam at intermediate support and *L* beam at mid-span.

Note 9. *B17 - B18* has been designed giving three alternative solutions namely :

(1) Two span continuous beam with unequal depths.

(2) Two span continuous beam with equal depths.

(3) Both beams assumed to be simply supported at ends.

Finally the results have been compared.

Note 10. There is no slab at floor level connecting *B5* because of stair room except for approximately 1*m* portion of landing slab. The beam, therefore, acts as rectangular beam carrying UDL and a point load transferred from beam *B27*.

Beam *B10* does not have slab on stair side. It has a slab *S4* on the other side spanning transversely. It has, therefore, been designed as *L* beam.

7.3.2 Common Data for Design of Beams

The common data repeatedly required for design of selected sizes of beams is worked out so that the same can be directly used.

(1) *Trial Section* : The basic principle in assuming trial sections is that types of sections should be *minimum* to utilise the form work repeatedly. The centering process also becomes very simple because the beam bottoms are at the same level. This reduces the cost considerably. All spans can be grouped within a variation of ± 20% and same section be adopted for beams in one group of spans. For example, in this design, one depth (380 *mm*) is adopted for all beams having spans 4 to 5 metres and another depth (700 *mm*) is adopted for beams with span 10 metres. Width of all beams has been kept constant = 230 *mm*. Thus, the number of sections adopted has been kept minimum.

(2) *Effective Cover d'* : In beams with spans up to 5 metres, Number - Diameter combination of bars can be easily adjusted so that all bars are placed in one row only. In such case, cover of nearly D/10 *i.e.* 40*mm* is considered adequate for beams of size 230 × 380 *mm* and 70 *mm* for section 230 × 700 *mm*.

(3) *Effective depth d* $= D - d' = 380 - 40 = 340$ *mm* for beams 380 *mm* deep,
$= 700 - 70 = 630$ *mm* for 700 *mm* deep beams.

(4) *Depth of flange* D_f = Depth of slab supported.

These are as follows :

Slab *S1* = 100 *mm* ,
Slab *S2* = 110 *mm* ; Slabs *S3* and *S4* = 130 *mm* ;
S5 = 100 *mm*. Stair Slab = 150 *mm*. and

(5) *Loads* : (a) Loads from roof/floor slab (self weight + finish + live load) :

Slab	*S1*	*S2*	*S3*	*S4*	*S5*	*Stairs*
Total Working load	5.00	6.00	6.50	6.50	5.00	12.45 kN/m^2
Total ultimate load	7.5	9.0	9.75	9.75	7.5	18.68 kN/m^2

(b) *Wall* : 250 *mm* thick 1.0 *m* high. $w_w = 20 \times 25 \times 1.0 = 5\ kN/m$
250 *mm* thick 3.2 *m* high $w_w = 20 \times 25 \times 3.2 = 16\ kN/m$
(Wall thickness includes thickness of plaster)

Grill : 75 *mm* thick 1.0 *m* high $w_g = 2/3 \times 25 \times 0.075 \times 1 = 1.25\ kN/m$

(c) *Self Weight of the beam* : It is only the weight of rib below the slab. Since the slab thickness varies, the depth of rib is approximately taken equal to (*D* - 100) on the safer side assuming minimum thickness of slab = 100 *mm*.where *D* - total depth of beam.

Weight of beam adopted in this design is as follows :

For beam 230 × 380 *mm*, $w_s = 25 \times 0.23 \times (0.38 - 0.1) = 1.60\ kN/m$

For beam 230 x 700 *mm* , $w_s = 25 \times 0.23 \times (0.7 - 0.13) = 3.28\ kN/m$

(6) *Approximate Design Moment Coefficients :*

End Condition No.	Beam Type	Support	Midspan
1	Beam Simply Supported at both ends : UD Load Central Point Load	0 0	1/8 1/4
2	Beam Simply Supported at one end and – Continuous at the other : UD Load – Central Point Load	–1/10 –1/6	1/10 1/7
3	Beam Continuous at both ends : UD Load Central Point Load	–1/12 –1/8	1/12 1/8

(7) *Design Constants for a balanced section :* For M20 Fe 415

% Redistribution of moments			
0%	20%	30%	*Equation*
$k_{u.max}$ = 0.48, $R_{u.max}$ = 0.76 N/mm^2 $p_{t.max}$% = 0.96%	$k_{u.limit}$ = 0.40 $R_{u.limit}$ = 2.4 N/mm^2 $p_{t.limit}$ = 0.80%	$k_{u.limit}$ = 0.30 $R_{u.limit}$ = 1.89 N/mm^2 p_t = 0.60%	*Eq. 4.1.2d* *Eq. 4.1.5c* *Eq. 4.1.6e*
For beam 230 × 380 *mm* with *d* = 340 *mm* , $M_{ur.max}$ = 73.4 *kN.m*	$M_{ur.limit}$ = 63.7 *kN.m*	$M_{ur.limit}$ = 50.25 *kN.m.*	*Eq. 4.1.5d*

(8) *Minimum Stirrups* : For b = **230 *mm***, for **6 *mm*** diameter two legged M.S.(Fe250) stirrups ($A_{sv} = 2 \times 28 = 56\ mm^2$), required spacing is

$$s = 0.87 f_y \times A_{sv} / (0.4b) = 0.87 \times 250 \times 56 / (0.4 \times 230) = \text{say } \mathbf{130\ mm.}$$

Thus, minimum stirrups spacing for beam width of 230 *mm* is: ϕ 6 *mm* at 130 *mm* c/c

(9) *Shear Resistance of Minimum Stirrups :* $V_{usv.min}$ = 0.4 *bd kN.* ... ... *(Eq. 4.4.8)*

For beam 230 × 380 *mm* (*d* = 340*mm*) , $V_{usv.min}$ = 0.4 × 230 × 340/1000 = 31.28 *kN.*

For beam 230 × 700 *mm* (*d* = 630 *mm*) , $V_{usv.min}$ = 0.4 × 230 × 630/1000 = 57.96 *kN*

(10) *Required Development length for* M20 - Fe 415 is $L_d = 47\ \phi$ ... ... *(Table 4.6.2)*

For 12 *mm*, bar required L_d = 47 × 12 = 564 *mm*

For 16 *mm*, bar required L_d = 47 × 16 = 752 *mm*

For 25 *mm* bar required L_d = 47 × 25 = 1175 *mm*

Table - 7.3.1 : Properties of Beam Section 230 mm × 380 mm (d = 340 mm) used in This Design

No.Diam mm	A_{st} mm	p_t %	M_{ur} kN.m	V_{uc} kN	$V_{usv.min}$ kN	$V_{ur.min}$ kN	Min.Stir ϕ -s(mm)	Reqd L_d
2-#12	226	0.289	26.10	29.62	31.28	60.93	φ6@130	564
2-#10 +1#12	270	0.345	30.77	31.72	31.28	63.00	φ 6@130	564
2-#10+2#12	383	0.490	42.24	37.1	31.28	68.38	φ6@130	564
3-#12	339	0.433	37.87	35.02	31.28	66.30	φ6@130	564
2-#16	402	0.514	44.10	37.89	31.28	69.17	φ6@130	752
4-#12	452	0.578	48.83	39.49	31.28	70.77	φ6@130	564
5-#12	565	0.722	58.96	43.09	31.28	74.37	φ6@130	564

7.3.3 Design of Typical Beams with Detailed Theoretical Calculations

The beams have been designed according to the procedure given in *Sect. 6.3.4*

7.3.3.1 *Category - I : Beams Simply Supported at Both Ends :*

Beam - B26 Simply Supported at both ends

Step	Design Calculations	Reference
1.	*Beam Mark* : *B26* at mid - landing level.	
2.	*End Condition* : Simply Supported at Both Ends. $EC = 1$.	*Sect. 6.3.2*
3.	*Span* : $L = 2.5\ m = 2500\ mm$.	
4.	*Section* : Assumed $b = 230\ mm, D = 380\ mm, d' = 40\ mm, d = 340\ mm$.	
5.	*Loads* : Due to supported slabs - Left : Landing of stair flights - I and II. $w_l = 12.45\ kN/m^2,\ L = 3.26\ m$. - Right : Nil. *Load from* : Stair Flights - I and II, $12.45 \times 3.26/2 = 20.3\ kN/m$ Grill 4.8 *m* high $4.8 \times 1.25 = 6.0\ kN/m$ Self weight, $25 \times 0.23 \times (0.38 - 0.1) = 1.6\ kN/m$ Total Working load $w = 27.9\ kN/m$ Design Ultimate load $w_u = 1.5 \times 27.9 = 41.9\ kN/m$	
6.	(a) *Design Moment* : $M_u = w_u L^2/8 = 41.9 \times 2.5^2/8 = 32.7\ kN.m$ (b) Maximum Ultimate Moment of Resistance of Rect. Section, $M_{ur.max} = R_{u.max}\ bd^2$: For M20 Fe 415, $R_{u.max} = 2.76\ N/mm^2$ $= 2.76 \times 230 \times 340^2 \times 10^{-6} = 73.4\ kN.m > M_u$ The beam is singly reinforced and the section is under - reinforced.	 *Table 4.1.1*
7.	*Main Steel* : Required $A_{st} = \dfrac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \dfrac{4.6 \times 32.7 \times 10^6}{20 \times 230 \times 340^2}}\right] \times 230 \times 340$ $= 289\ mm^2$ (b) No.- Dia. : Provide <u>3-#12</u> $(A_{st} = 3 \times 113 = 339\ mm^2)$ Provide Bottom St. : 2-#12 + Bottom Bt.1-#12.(from 0.5 *m* from support) Provide Top St. : 2-#10 as anchor bars. (c) No of rows and No. of bars in each row : Provide all 3 bars in 1 row. (d) Check for width : Required $b = n_l \times \phi + (n_l + 1) \times 25 + 2 \times \phi_{st}$ For $n = 3$, $b = 3 \times 12 + 4 \times 20 + 2 \times 6 = 128 mm < 230\ mm$ provided ∴ O.K. (e) Check for Effective Cover d' : For mild environment nominal cover = 20 *mm* Assuming $\phi 6\ mm$ stirrups d' = Nominal cover + Dia. of stirrups + $\phi / 2$ $= 20 + 6 + 12/2 = 32\ mm < 40\ mm$ ∴ O.K.	 *Eq. 4.1.6a* *Eq. 6.3.1* *Table C - 1*
8.	*Design for Shear* : $V_{u.max} = w_u\ L/2 = 41.9 \times 2.5/2 = 52.4\ kN$. $V_{uc} = \tau_{uc}\ bd$, τ_{uc} depends upon p_t % at support. $A_{stl} = 2$ bars of # 12*mm* $= 2 \times 113 = 226\ mm^2$ (tension steel at support) $p_t = 100 \times 226/(230 \times 340) = 0.289\%$	

Beam - B26 continued....

Step	Design Calculations	Reference
	$\tau_{uc} = 0.36 + \dfrac{(0.48-0.36)}{(0.50-0.25)} \times (0.289 - 0.25) = 0.3787\ N/mm^2$	Table 4.4.1
	$V_{uc} = 0.3787 \times 230 \times 340/1000 = 29.61\ kN$	
	$V_{usv.min} = 0.4\,bd = 0.4 \times 230 \times 345/1000 = 31.28\ kN$	Eq.4.4.8
	$V_{ur.min} = 29.61 + 31.26 = 60.87\ kN > V_{u.max}\ (= 52.4\ kN)$	
	∴ Minimum stirrups are sufficient.	
	Provide ϕ 6 mm Fe 250, 2 - legged stirrups ($A_{st} = 56\ mm^2$)	
	Pitch $s = 0.87 f_y a_{st} / (0.4b) = 0.87 \times 250 \times 56/(0.4 \times 230) = 132\ mm$	Eq. 4.4.7
	= say 130 mm < .75 × 340 and < 300 mm ∴ O.K.	
	Provide ϕ6 mm 2 - legged stirrups @ 130 mm c/c	
9.	*Check for Development Length :*	
	Required $L_d = 47\,\phi = 47 \times 12 = 564\ mm < 1.3\,M_1/V + L_o$	Eq. 4.6.3b
	Now , M_1 = M.R. of A_{st1} at support , A_{st1} = 2- #12 = 226 mm^2	
	$= 0.87 \times 415 \times 226 \times 340\left(1 - \dfrac{415 \times 226}{20 \times 230 \times 340}\right) \times 10^{-6}$	Eq. 4.1.3b
	$= 26.1\ kN.m$	
	$V = V_{u.max} = 52.4\ kN.$	
	$L_o = L_d/3 - b_s/2$ where, b_s = width of support	
	Assuming b_s = 230mm , $L_o = 564/3 - 230/2 = 73\ mm.$	
	Available $L_d = \dfrac{1.3\,M_1}{v} + L_o = \dfrac{1.3 \times 26.1 \times 1000}{52.4} + 73 = 720\ mm$	
	∴ 564 mm < 720 mm ∴ safe.	
10.	*Check for Deflection :*	
	$f_s = 0.58 \times 415 \times 289/339 = 205\ N/mm^2$	Eq. 4.7.1b
	$(p_t)_{prov} = 100 \times 339/(230 \times 340) = 0.43\%$	
	For $f_s = 205\ N/mm^2$ and $p_t = 0.43\%$, $\alpha_1 = 1.5$	Fig. 4.7.1
	Basic L/d ratio, r_b = 20 for simply supported beams	Eq. 4.7.1a
	∴ Required $d = 2500/(20 \times 1.5) = 84\ mm << 340\ mm$ ∴ safe	
11.	*Load on column* = Load on C6 and C12 = 41.9 x 2.5/2 = 52.4 kN	

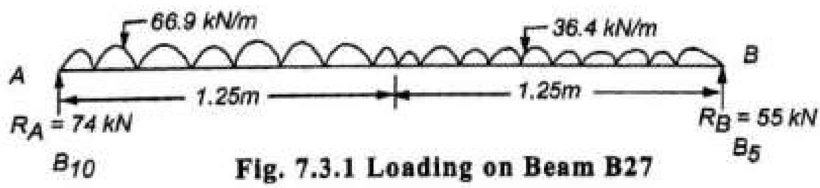
Remarks : *(1) Since the factor of 0.58 in Eq. 4.7.1b has been arrived as $f_s = f_y/(1.5 \times 1.15)$. Therefore, graphs given in Fig. 4..7.1 for f_s = 290 N/mm², 240 N/mm² and 145 N/mm² can safely be used for steel grade Fe 500, Fe 415 and Fe 250 respectively corresponding to percentage of steel required.*[7.1]

e.g. $(p_t)_{reqd}$ = 100 × 289/(230 × 340) = 0.37% , For p_t = 0.37 % and f_s = 240 N/mm² from Fig. 4.7.1
α_1 = 1.4 Required d = 2500/(20 × 1.4) = 89mm < 340 mm ∴ safe

(2) For Fe415 i.e. f_s = 240 N/mm² and p_t = 1% modification factor α_1 = 1. The required p_t% rarely exceeds 1%. Therefore, it is sufficient to check that actual L/d < 20. Since L/d ratio is less than 16, the deflection check may be skipped.

(3) It may be felt that the section 230 mm × 380 mm providing $M_{ur.max}$ = 73.4 kN.m is much larger than required M_u = 32.7 kN.m and a reduced section can be economical. Even the section 230 mm × 300 mm giving $M_{ur.max}$ = 42.91 kN.m greater than M_u (=32.7 kN.m) is sufficient. But area of steel works out to 406 mm² requiring to provide 4 - # 12 mm bars giving p_t% = 0.75% which is much greater than 0.43% in the present case. Since the cost of the steel is high and as long as minimum reinforcement clause does not govern it is considered that it is preferable to provide less quantity of steel and adopt the same section to reuse the form work. As such the adopted section is considered to be appropriate.

Beam - B27 Simply supported at both ends

Step	Design Calculations	Reference
1.	*Beam Mark* : *B27*	
2.	*End Condition* : Simply Supported at Both Ends, $EC = 1$.	
3.	*Span L* : 2.5 metres = 2500 *mm*	
4.	*Section* : Assumed $b = 230mm$, $D = 380mm$, $d' = 40\ mm$, $d = 340\ mm$.	
	Nominal cover for mild environment = 20 *mm*	*Table C-1*
	Assuming maximum diameter of stirrups of 8 *mm* and diameter of main steel 20 *mm*,	
	Effective cover = $d' = 20 + 8 + 20/2 = 38\ mm$ say 40 *mm*	
	Effective depth = $d - d' = 380 - 40 = 340\ mm$	
5.	*Load* :	
	On left half span 1.25 m length carrying stair flight - II :	
	Roof slab load *S2*: $w_{s1} = w_{eqb1} = (w\,L_x/6)$	*Eq.5.3.1a*
	$w_{s1} = 6 \times 2.5/6 = 2.5\ kN/m$	
	Stairs slab load : $w_{s2} = 12.45{\times}3.26/2 = 20.3\ kN/m$	
	Wall load : $w_w = 5 \times 3.2 = 16.0\ kN/m$	
	Stairs cap slab *S5* $w_{eqb} = 5 \times 2.5/3 = 4.17\ kN/m$	
	Self weight- beam: $w_b = 1.6\ kN/m$	*Sect. 7.3.2(5c)*
	Total working load w = (slab *S2* + stairs slab right + wall + cap slab + self)	
	$w = (2.5 + 20.3 + 16.0 + 4.17 + 1.6) = 44.6\ kN/m$	
	$(w_u)_{right} = 1.5 \times 44.6 = 66.9\ kN/m$ on 1.25*m*	
	One right half span length 1.25m without stair flight but with wall carrying cap slab :	
	Roof slab *S2* on left $w_{s1} = 2.5\ kN/m;$	
	Stairs Slab Right $w_{s2} = 0$	
	Wall load 3.2*m* high, $w_w = 5 \times 3.2 = 16.0\ kN/m$	
	Cap slab *S5* : $w_{eqb} = 5 \times 2.5/3 = 4.17\ kN/m$	
	Self - beam : $w_b = 1.60\ kN/m$	
	Total working load $w = \underline{24.27}\ kN/m$	
	$(w_u)_{right} = 1.5 \times 24.27 = 36.4\ kN/m$	
	The details of load are shown in *Fig. 7.3.1*	
		
	Fig. 7.3.1 Loading on Beam B27	
6.	*Design Moment* :	
	Point load on $B5 = R_B = [66.9{\times}1.25^2/2{+}36.4{\times}1.25{\times}(1.25 + 1.25/2)]/2.5 = 55\ kN.$	
	Point load on $B10 = R_A = 66.9 \times 1.25 + 36.4 \times 1.25 - 55 = 74\ kN$	
	$x_{max} = 74/66.9 = 1.1m$ from A	
	$M_u = 74 \times 1.1 - 66.9 \times 1.1^2/2 =$ say 41 *kN.m*.	
	$M_{ur.max} = 73.4\ kN.m$ (calculated earlier for beam *B26*) $> M_u\ (= 41\ kN.m)$	
	∴ Section shall be under-reinforced.	

Beam - B27 continued....

Step	Design Calculations	Reference
7.	*Main Steel :* Required $A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 41 \times 10^6}{20 \times 230 \times 340^2}}\right] \times 230 \times 340$ $= 371\ mm^2$ Provide 4 - # 12 *mm* and 2 - # 10*mm* anchor bars at top Check for width and cover has not be repeated since it is the same as that for Beam - *B26*.	
8.	*Design for Shear :* $V_{u.max} = R_A = 74\ kN$ $V_{uD} = 74 - 66.9 \times (230/2 + 340)/1000 = 43.6\ kN < V_{ur.min}$ (= 70.77 *kN*) ∴ Provide Minimum stirrups ϕ6 *mm* at 130*mm* c/c as in beam *B26*.	Table 7.3.1
9.	*Check for Development Length :* Maximum bar diameter # 12 *mm*. Required $L_d = 564mm$. $L_o = 73\ mm$ Available $L_d = 1.3\ M_1/V + L_o, V = V_{u.max} = 55\ kN$, $M_1 = 41\ kN.m$ $564\ mm < 1042\ mm = \frac{1.3 \times 41 \times 1000}{55} + 73$	Sect.7.3.3(9) Eq. 4.6.3b
10.	*Check for deflection :* Actual *L/d* ratio = 2500/340 < 20 ∴ safe	
11.	*Load on Supporting Beams :* Load on *B5* = 55 *kN* and on *B10* = 74 *kN*	

Beam - B11 Simply Supported at both ends

Step	Design Calculations	Reference
1.	*Beam Mark :* *B11* *End condition* : Simply Supported at both ends. *EC* = 1.	
2.	*Span* : 4.5 metres = 4500*mm*.	
3.	*Section* : 230 × 380 *mm*, *d* = 340 *mm*, D_f = 130 *mm* (Slab - *S4*).	
4.	*Loads* : Slab *S4* (Tri.) from both sides + self weight of beam (=1.6 *kN/m*) $w = 6.5\ kN/m^2$, $L_x = 4.5\ m$. $w_u = 1.5\ (2 \times w_{eqb} + w_s) = 1.5(2wL_x/3 + w_s)$ $= 1.5\ (2 \times 6.5 \times 4.5/3 + 1.6) = 31.65\ kN/m$ For shear, $w_{us} = 1.5\ (2 \times 6.5 \times 4.5/4 + 1.6) = 24.3\ kN/m$.	 Eq. 5.3.3 Eq. 5.3.4
5.	*Design Moment* : $M_u = 31.65 \times 4.5^2/8 = 80.1\ kN.m$ Beam acts as *T* beam at mid span (Slab *S4* is on both sides). ∴ The beam will be designed as *T* beams. $b_f = L_o/6 + 6D_f + b_w$ for *T* - beams. $L_o = L$ for simply supported beams = 4500 *mm*. $b_f = (4500/6 + 6 \times 130 + 230) = 1760\ mm$ It will now be verified whether the neutral axis lies in flange or web. For $x_u = D_f, (M_{ur1}) = 0.36 \times 20 \times 1760 \times 130\ (340 - 0.42 \times 130) \times 10^{-6}$ $= 470\ kN.m > M_u$ $\therefore x_u < D_f$	 Eq. 4.3.1 Eq. 4.3.8

Beam - B11 continued....

Step	Design Calculations	Reference
6.	Required $A_{st} = \frac{0.5 \times 20}{415}\left[1-\sqrt{1-\frac{4.6 \times 80.1 \times 10^6}{20 \times 1760 \times 340^2}}\right] \times 1760 \times 340$	Eq. 4.3.5
	$= 669\ mm^2$	
	Provide 4–#16 mm in 1 row. Provided $A_{st} = 804\ mm^2$	
	Provide Top 2-#10 mm as anchor bars.	
7.	*Design for Shear* :	
	$V_{u.max} = w_u L/2 = 24.3 \times 4.5/2 = 54.7\ kN$	
	$A_{st1} = A_{st} = 804\ mm^2$, $p_t = 100 \times 804/(230 \times 340) = 1.03\%$	
	For M20 and $p_t = 1.03\%$, $\tau_{uc} = 0.626\ N/mm^2$	Table 4.4.1
	$V_{uc} = 0.626 \times 230 \times 340/1000 = 48.9\ kN.$	
	$V_{usv.min} = 0.4 \times 230 \times 340/1000 = 31.28\ kN$	Eq.4.4.8
	$V_{ur.min} = 48.9 + 31.28 = 80.18\ kN > V_{u.max}$ (= 54.7 kN).	
	∴ Provide minimum stirrups of ϕ 6 mm at 130 mm c/c	
8.	*Check for Development* : Required $L_d = 752\ mm$ for 16 mm bar	Table 7.3.1
	$M_1 = M_{ur} = 0.87 \times 415 \times 804 \times 340 \times \left(1 - \frac{415 \times 804}{20 \times 230 \times 340}\right) \times 10^{-6}$	Eq. 4.1.3b
	$= 77.64\ kN.m$	
	Available $L_d = \left(\frac{1.3 \times 77.64 \times 1000}{54.7}\right) + 73 = 1918\ mm$	Eq. 4.6.3b
	Required $L_d = 752\ mm < 1918\ mm$ ∴ safe	
9.	*Check for Deflection* :	
	For the flanged section detailed check for deflection is carried out.	
	Basic L/d ratio = 20, $(p_t)_{provided} = (100 \times 804) / (230 \times 340) = 1.03\%$	
	$f_s = 0.58 \times 415 \times 669 / 804 = 200\ N/mm^2$	Eq. 4.7.1b
	for $p_t = 1.03\%$ and $f_s = 200\ N/mm^2$, $\alpha_1 = 1.18$	Fig.4.7.1
	$b_w/b_f = 230 / 1760 = 0.13$, $\alpha_3 = 0.8$	Fig.4.7.3
	$d = \frac{L}{\text{Basic } L/d \times \alpha_1 \times \alpha_3} = \frac{4500}{20 \times 1.18 \times 0.8}$	Eq. 4.7.1a
	$= 238mm < 340\ mm$ ∴ safe	
10.	*Load on column* : Load on column C13 and C14 = $V_{u.max} = 54.7\ kN$	

Remarks : *Since beams are safe in deflection and satisfy the requirements for development length these checks will not be taken for further design of beams.*

Beam - B19, B20, B21 Simply Supported at Both Ends

Step	Design Calculations	Reference
1.	*Beam Mark*: *B19.*	
2.	*End Condition* : Simply Supported at Both Ends. $EC = 1$.	
3.	*Span* : $L = 10$ *metres* $= 10000$ *mm*.	
4.	*Section* : Assumed $b = 230$ *mm*, $D = 700$ *mm*, $d' = 70$ *mm* $\therefore d = 700 - 70 = 630$ *mm*, $D_f = 130$ *mm*.	
5.	*Loads* : Slabs on both sides *S3* (One - way) $w = 6.5\ kN/m^2$, $L_x = 4m$. Slab load -Left $w_{s1} = 0.60 \times wL_x = 0.60 \times 6.5 \times 4 = 15.60\ kN/m$ -Right $w_{s2} = 0.55 \times wL_x = 0.55 \times 6.5 \times 4 = 14.30\ kN/m$ Since dead lead is much larger than live load the shear coefficient 0.55 corresponding to dead load for inner span is taken for the slab on the right *(See Table 5.1.1b)*. Wall - Nil $w_w = 0$ Self (beam) $w_b = 25 \times 0.23 \times (0.7 - 0.13) = 3.28\ kN/m$ Total working load $w = 15.60 + 14.30 + 0 + 3.28 = 33.18\ kN/m$ Total ultimate load $w_u = 1.5 \times w = 1.5 \times 33.18$ = say $50\ kN/m$.	
6.	*Design Moment* : $M_u = 50 \times 10^2/8 = 625\ kN.m$. Beam acts as *T*- beam. $L_o = L = 10000\ mm$. $b_f = L_o/6 + 6D_f + b_w = 10000/6 + 6 \times 130 + 230 = 2677\ mm$. For $x_u = D_f$, $(M_{ur1}) = 0.36 f_{ck} b_f D_f (d - 0.42 D_f)$ $= 0.36 \times 20 \times 2677 \times 130\ (630 - 0.42 \times 130) \times 10^{-6}$ $= 1441 kN.m > M_u\ (= 625\ kN.m)$ $\therefore x_u < D_f$	*Eq. 4.3.8*
7.	*Main Steel* : (a) Area : Required $A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 625 \times 10^6}{20 \times 2677 \times 630^2}}\right] \times 2677 \times 630$ $= 2849\ mm^2$ Provide 6-#25 in two rows with maximum of 3 bars in bottom row. Area provided $= 2945\ mm^2$ Check for width $= [3 + (3 + 1)] \times 25 + 2 \times 8 = 191\ mm < 230\ mm$ $\therefore$O.K. Nominal cover for mild environment $= (\phi$ or $20\ mm) = 25\ mm$ *Note : The nominal cover specified is 20 mm but as per Note -1 in Table C-1 it shall not be less than bar diameter.* Assuming stirrup diameter equal to 8 *mm* Required effective cover for two rows $= 25+8+25+25/2 = 70.5\ mm \cong$ assumed 70 *mm* $\therefore$ O.K	*Eq. 4.3.5* *Eq. 6.3.1* *Table C-1*
8.	*Curtailment of bars (If desired)* : Minimum number of bars required to be continued into the support $N_l = N_{max}/3 = 6/3 = 2$. Assuming only 2 bars to be curtailed and 4 to be continued.	*Sect.4.6.5b*

Beam - B19, B20, B21 continued....

Step	Design Calculations	Reference
	The distance of Theoretical Point of Cutoff *(TPC)* from the centre of support $x_1 = \frac{L}{2}\left(1-\sqrt{\frac{N_1'}{N_{max}}}\right)$ where, N_1' = No. of bars to be curtailed = 2. and N_{max} = No. of bars at the point of maximum B.M. = 6. $x_1 = (L/2)(1-\sqrt{2/6}) = 0.211L = 0.211 \times 10 = 2.11\ m = 2110mm$. The bars to be curtailed will be required to be extended from *TPC* through a minimum distance of effective depth *d or 12ϕ* whichever is greater. ∴ Actual point of curtailment *(APC)* = 2110 – 630 = 1480 *mm*	Sect.4.6.6
9.	***Design for Shear :*** $V_{u.max} = 50 \times 10/2 = 250$ kN The critical section for shear occurs at a distance *d* from the face of support. $V_{uD} = V_{u.max} - w_u (b_s/2 + d)$ $= 250 - 50\ (230/2 + 630)/1000 = 212.8\ kN$ Shear resistance of concrete depends on the area of tension steel A_{st1} at support and hence upon the scheme of curtailment of bars if curtailment is to be done, *or* upon the number of bent up bars to be used for shear, if proposed. Here, the design illustrates calculations for design of shear reinforcement for all alternatives, namely, *(i) without any curtailment or use of bent-up bars,* *(ii) without any curtailment but using bent-up bars,* *(iii) with curtailment and no bent-up bars,* The comparison will be made to bring out the economics. ***Case - I : No Curtailment and No Bent - up Bars*** $A_{st1} = A_{st.max} = 2945\ mm^2$ $p_t = 100\ A_{st}/(bd) = 100 \times 2945/(230 \times 630) = 2.03$ % For M20 and $p_t = 2.03$ % , $\tau_{uc} = 0.792\ N/mm^2$ $V_{uc} = 0.792 \times 230 \times 630/1000 = 114.76\ kN$ $V_{usv.min} = 0.4 \times 230 \times 630 = 57.96\ kN$ $V_{ur.min} = 114.76 + 57.96 = 172.72\ kN < V_{uD}$ (=212.8 *kN*) ∴ Minimum stirrups will not be sufficient. They will be required to be designed. $V_{usv} = V_{us} = V_{uD} - V_{uc} = 212.8 - 114.76 = 98.04\ kN$ Assuming # 8 *mm* 2-legged vertical stirrups of grade Fe415 , $A_{sv} = 100\ mm^2$, Required $s = 0.87 f_y A_{sv}\ d/V_{usv} = 0.87 \times 415 \times 100 \times 630/(98.04 \times 1000)$ = 232 *mm*. say 230 *mm* < 300 *mm* and < 0.75 x 630 ∴ safe Provide # 8 *mm* 2-legged stirrups at 230 *mm* c/c Zone of these designed stirrups is given by : $L_{s1} = (V_{u.max} - V_{ur.min})/w_u = (250 - 172.72)/50 = 1.55\ m$ Number of stirrups in this zone $N_{s1} = (L_{s1} - b_s/2 - 50)/s_1 + 1$ = say 7 (since first stirrups will be placed at a distance 50 *mm* from the inner face of support) ***Comments :*** *Even though the code has specified the limitation on maximum spacing of stirrups it has not given the minimum spacing SP-34[7.3] shows that the first stirrups shall be placed at a distance of 50 mm from the face of support or in other words 50 mm = s/2 .This indirectly implies that the minimum spacing of stirrups shall be not less than 100 mm for proper concreting.*	 Table 4.4.1 Eq. 4.4.8 Eq.6.3.2a

Beam - B19, B20, B21 continued....

Step	Design Calculations	Reference
	Actual length of region covered up to the *8th* stirrup $= 230/2 + 50 + (7-1) \times 230 \quad = 1545\ mm$ Theoretical region covered by these 7 stirrups is $s_1/2 = 230/2$ $= 115\ mm$ more than actual length $= 1545 + 115 \quad = 1660\ mm$ which is greater than theoretical length L_{s1} required $\quad = 1550\ mm$ ∴ o.k. Beyond this zone, minimum stirrups will be sufficient. However, they are not necessary in the region where V_u is less than 0.5 V_{uc} near mid-span. In this region nominal stirrups may be provided just to hold main bars and anchor bars to form a cage.	
	∴ Length of Zone of nominal stirrups $L_{s3} = 0.5\ V_{uc}/w_u = 0.5 \times 114.76/50 = 1.147 m$.	*Eq. 6.3.2c*
	Length of Zone of minimum stirrups $L_{s2} = L/2 - L_{s1} - L_{s3}$ $L_{s2} = 10000 / 2 - 1545 - 1147 = 2308\ mm$.	
	For φ 6*mm* 2-legged stirrups ($A_{sv} = 56\ mm^2$) of grade 250, the required spacing of minimum stirrups is given by : $\quad s = 0.87 \times 250 \times 56 / (0.4 \times 230)$ $=$ say $130\ mm < (0.75 \times 630$ *or* $300\ mm)$	*Eq. 4.4.7*
	∴ Provide φ 6 *mm* 2-legged stirrups at 130 *mm* c/c as minimum stirrups. ***Comments :*** *If #8 mm stirrups are to be used then the spacing of # 8 mm stirrups works out to 392 mm which is required to be limited to 300 mm as per the provisions of code and therefore becomes uneconomical hence φ 6 mm bars are used.* *In the zone of Nominal stirrups , provideφ 6 mm 2-legged stirrups at maximum persmissible spacing of 300 mm (< 0.75 d or 300 mm)* Number of stirrups in *Zone-II* of minimum stirrups $N_{s2} = (L_{s1} + L_{s2}$ – actual region covered by stirrups in Zone - I) / Number of minimum stirrups $= 1550 + 2308 - 1545\ ) / 130 = 18$ nos Actual region covered by stirrups in 1st and 2nd zone = 1545+18x130=3885 *mm* Number of stirrups in *Zone-III* of nominal stirrups $N_{s3} = (10000/2 - 3885)/300 = 4$Nos. It is the number only on one side of the centre line and therefore it can be in multiply of 0.5. Total number of stirrups provided = 2 x 7 = 14 Nos of # 8 *mm* in *Zone - I* + 2 x 18 (in Zone - II) + 2 x 4 (in *Zone -III*) = 44 nos of φ6 *mm* Distance covered = 12 x 230 + 36 x 130 + 8 x 300 + 230 + 100 = 10170 *mm* ≅10000 *mm*	
	Case - II : No Curtailment but Use of Bent-up Bars Consider bending of 2 bars of #25*mm* which can be used for resisting shear also $A_{st1} =$ 4 of # 25 = 4 x 491 $= 1964\ mm^2$	
	$p_t =$ 100 x 1964 / (230 x 630) = say 1.35 % , $\tau_{uc} = 0.69\ N/mm^2$. $V_{uc} =$ 0.69 x 230 x 630 / 1000 = 100.00 *kN*, $V_{usv.min} =$ 57.96 *kN* as calculated in *Case -I* above. ∴ $V_{ur.min} =$ 100 + 57.96 = 157.96 *kN* < V_{uD} (= 212.8*kN*) calculated above ∴ Minimum stirrups will not be sufficient. The shear reinforcement will be required to be designed. $V_{us} = V_{uD} - V_{uc} = 212.8 - 100 = 112.8\ kN$ Before deciding the scheme of bending of bars, it is necessary to determine the zone of shear reinforcement. The length of this zone is given by : $L_{s1} = (V_{u.max} - V_{ur.min}) / w_u = (250 - 157.96)/50 = 1.84\ m = 1840\ mm$	*Table 4.4.1*

Beam - B19, B20, B21 continued....

Step	Design Calculations	Reference
	Since this is greater than 2*d* (=2 x 630 = 1260 *mm*), bending only one bar near the support and then again bending another bar at a distance of *d* from the point of bend of the first bent-up bar will enable to cover this shear zone by bent-up bars to a maximum extent. This will also economize the stirrups. The first bar will be bent at a distance not exceeding 2 *d* from the centre of support *(see Fig. 5.5.4)*. Since, two bent-up bars are to be used one after the other, they should cover the entire zone of shear reinforcement. Therefore, it is proposed to bend first bar at a distance of 2 *d* = 2 x 630 = 1260 *mm* and the second bar at a distance of *d* from the first bent - up bar ∴ Total distance covered = 2 x 630 + 630 = 1890 *mm* > 1840 *mm* Shear resistance of 1 bar of # 25 *mm* (A_{st} = 491 mm^2) bent at 45° will be $V_{usb} = 0.87 f_y A_{sb} \sin\alpha$ = 0.87 x 415 x 491 x sin 45° / 1000 = 125.33 *kN* > $V_{us/2}$ (= 56.4 *kN*) Therefore, stirrups will have to be provided for , V_{usv} = 0.5 V_{us} = 0.5 x 112.8 = 56.4 *kN* < $V_{usv.min}$ Since, this value is less than $V_{usv.min}$ (=57.96 *kN*) minimum stirrups will be sufficient. At a distance of 1260 *mm* where the first bar is bent, V_u = 250−50x1.26=187*kN* At this section, 5 bars of # 25 are available and, therefore, shear strength V_{uc} of concrete will be greater than that at support (= 100 *kN*) for 4 bars of #25*mm*. However, this increase is ignored for simplicity. ∴ V_{us} = 187 − 100 = 87 *kN* Shear strength of 1 bent-up bar V_{usb} = 125.33 *kN* as calculated earlier. This itself is greater than V_{us}. However, according to the Code V_{usb} ≯ 0.5 V_{us}. Therefore, stirrups will be required to provided for 0.5 V_{us} = 0.5 x87 = 43.6 *kN*. This is less than $V_{usv.min}$ = 57.96 *kN*. Therefore, in this region also minimum stirrups are adequate. ∴ Provide ϕ6*mm* at 130 *mm* c/c of grade Fe250 as calculated in *Case - I* from support to the point of second bend. Since in *Zone - I* also minimum stirrups are sufficient due to bent - up bars and therefore , in *Zone - I* and *Zone - II* minimum stirrups will be provided ∴ Minimum stirrups required in *Zone - I* and *Zone - II* = (250 − 114.76/2)/50 = 3.852 *m* = 3852 *mm* ∴ No of ϕ 6 *mm* stirrups = N_{s1} = (3852 − 230/2 − 50) / 130 + 1 = 29 Nos. Actual length covered = (29 − 1) x 130 + 230 / 2 + 50 = 3805 *mm* This is effective for a length equal to 3805 + 130/2 = 3870 *mm* > 3853 *mm* Length of Zone of nominal stirrups remains the same equal to 1.147 as obtained in *Case - I* N_{s3} = (5000 − 3805) / 300 = 4 Nos ∴ *Total number of ϕ 6mm stirrups required = (29 + 4) x 2 = 66 Nos* ∴ Provide ϕ 6 *mm* 2-legged stirrups at 130 *mm* c/c from centre of support to 3800 *mm* and in central portion of 2400 *mm* provide ϕ 6 *mm* @ 300 *mm* c/c. Check for actual length covered = 2 x (29−1) x130 + (2 x 4) x 300 +2x50+230 = 10010 *mm* ≅ 10000 *mm* ∴ o.k. ***Remarks :*** *In practice it is preferable not to use bent-up bars of diameter > 20 mm because during bending process minor cracks may develop at the bend.*	 *Eq. 4.4.4*

Beam - B19, B20, B21 continued....

Step	Design Calculations	Reference

Case - III : Curtailment of Bars - No Bent-up Bars

At the critical section for shear at a distance of 230 / 2 + 630 = 745 from the centre of support, V_{uD} = 250 - 50 x 0.745 = 212.8 *kN* .

On curtailment of 2 bars, available bars at support are 4 of # 25 *(see Case - II)*, for which V_{uc} = 100.00 *kN* and V_{uD} = 212.8 *kN*, ∴ V_{us} = 212.8−100 = 112.8 *kN*

Since no bent - up bars have been provided V_{usb} = 0 and V_{usv} = V_{us} = 112.8 *kN*

For # 8*mm* 2-legged stirrups(A_{sv}=100 *mm*^{2}),of grade Fe415 required spacing is given by :

$$s = 0.87 \times 415 \times 100 \times 630/(112.8 \times 1000) = 200\ mm$$

For 4 of #25 at supports,

$V_{ur.min}$ = 157.96 *kN* and L_{s1} = (250 − 157.86) / 50 = 1.84 *m (see Case - II)*

Now, theoretical point of cutoff *(TPC)* from bending moment consideration is at a distance of 2110 *mm* from the centre of support.

Actual point of cutoff = *APC* = 2110 − 630 = 1480 *mm*. Since bars are curtailed at this point, one of the three conditions as per *Sect. 4.4.6* are required to be satisfied. — **Clause 26.2.3.2*

Condition - (a)

According to this condition, shear at *TPC* shall not exceed 2/3 *rd* the shear strength *i.e.* shear resistance at that section shall be least 1.5 times the shear at the section.

At *TPC*, V_u = 250 − 50 x 2 .11 = 144.5 *kN* .

∴ Required V_{ur} = 1.5 x 144.5 = 216.75 *kN*

V_{uc} = 100.00 *kN* for 4 bars of #25 as in *Case - II* above

∴ Required V_{us} = 216.75 − 100.00 = 116.75 *kN*

For #8*mm*2-legged stirrups (A_{sv} = 100 *mm*^{2}) of grade Fe415 required spacing is given by :

$$s = 0.87 \times 415 \times 100 \times 630 / (116.75 \times 1000) = \text{say } 190\ mm$$

This will be required between *TPC* to *APC i.e.* in the region between points 1480 *mm* and 2110 *mm* from the centre of supports.

Thus, spacing of # 8 stirrups is required to be 200 *mm* for a distance of 1480*mm* from centre of support to the actual point of cutoff and 190 *mm* for a distance 630 *mm* between *APC* to *TPC*. For simplicity and ease of fixing,190 *mm* spacing may be provided right from the support to *TPC* for a length of 2110 *mm* > L_{s1}

Since length of region of designed shear reinforcement L_{s1} is just 1840 *mm*.

Shear at *TPC* = 250 − 50 x 2.11 = 144.5 *kN* < $V_{ur.min}$ (=157.96 *kN*)

∴ Minimum stirrups ϕ 6 *mm* at 130 *mm* c/c would be provided beyond *TPC*

Number of #8*mm* stirrups required in length of 2110 *mm* (say *Zone - I)*

N_{s1} = (2110−230/2−50)/190+1= say 12 Nos.

Actual length of region covered by these stirrups ,

= 230 / 2 + 50 + (12 − 1) x 190 = 2255 *mm*>2110 *mm*

Length of zone of *nominal* stirrups remains the same equal to 1147 *mm*.

Length of zone of *minimum* stirrups = 10000 / 2 − 2255 − 1147 = 1598 *mm*

Number of ϕ 6 *mm* stirrups in *Zone - II* of *minimum* stirrups

N_{s2} = 1598 / 130 = say 12 Nos.

Actual length covered by stirrups in *Zone - I* and *Zone - II*

= 2255 + 130 x 12 = 3815 *mm*

Number of stirrups required in *Zone - III* of *nominal* stirrups

= (10000 / 2 − 3815) / 300 = say 4.

Beam - B19, B20, B21 continued....

Step	Design Calculations	Reference
	Total number of stirrups required = 2 x 12 (in *Zone - I*) = 24 Nos.of # 8 *mm* and 2 x (12 +4) = 32 Nos of ϕ 6 *mm* Total distance covered = 22 x 190 + 24 x 130 + 8 x 300 + 230 + 100 = 10030 *mm* ≅ 10 000 *mm* *OR Condition (b) :* At *TPC*, V_u = 144.5 *kN* while $V_{ur.min}$ = 157.96 *kN* as in obtained earlier. Therefore, minimum stirrups would have been adequate had the bars not been curtailed. Because of curtailment, additional stirrups are required to be provided, with area of minimum stirrups doubled *or* the spacing is halved. This gives ϕ 6*mm* at 65 *mm or* # 8 at 190 *mm* c/c.At the same time, it is necessary that he resultant pitch (*i.e.* 190 *mm*) shall not exceed *d*/8 β, where β = (No.of bars curtailed / Total No.of bars prior to curtailment) = 2/6 = 1/3. ∴ $s \leq$ 630 / (8 x 1/3) = 236 *mm* ∴ *s* = 190 *mm* is O.K. Thus, spacing for # 8 *mm* stirrups is required to be 200 *mm* from the support to *APC* for a distance 1480 *mm* and for satisfying the requirement of the Condition - (b), the spacing of # 8 stirrups is required to be 190 *mm* for a distance 630 *mm* from *APC* to *TPC*. For ease of fixing, a constant spacing of 190 *mm* is suggested from support to *TPC*. Rest of the details remain the same as given in *(a)* above *Condition - (C)* This condition is applicable only when area of extended bars beyond the point of cut off is double the required area. In this case, area of bars continued is just equal to that required beyond point of cut off. Therefore, shear reinforcement cannot be designed according to the requirements of this condition. ***Comparision of Reinforcement for 3 Cases :*** For each stirrups, vertical straight length between centres of horizontal legs *E* = 230 – 2 x 25 – 8 = 172 *mm* for# 8 *mm* stirrups and 174 *mm* for 6*mm* Horizontal straight length between centre of vertical legs *A* = 700 – 2 x 25 – 8 = 642 *mm* for # 8*mm* and 644 for ϕ 6 *mm* stirrups Length of # 8*mm* stirrups = 2(*A*+*E*) + 24ϕ = 2 (642 +172) + 24 x 8 = 1820 *mm* Length of ϕ6*mm* stirrups = 2(644 + 174) + 24 x 6 = 1780 *mm* Unit wt. of ϕ 8*mm* bar = 0.785 x area of bar in cm^2 x 1 *m* = 0.785x (π x 8^2/4)/100 = 0.395 *kg/m* Weight of # 8*mm* stirrups = 0.395 x 1820 / 1000 = 0.72 *kg/No* Unit wt. of ϕ 6 *mm* bar = 0.785 x 28.27 / 100 = 0.22 *kg/m* Weight of ϕ 6 *mm* stirrup = 0.22 x 1780 / 1000 = 0.39 *kg/No* Unit weight of 25 *mm* bar = 0.785 x (π x 25^2/ 4) / 100 = 3.853 *kg/m* Total length of main bar = end to end length – end clearances + 90° bend allowances (= 6ϕ) at each end = $L + b_s - 2\phi$ (end clearance) + 2 x 6 x ϕ = 10000+ 230 – 2 x 25 + 2 x (6 x 25) = 10480 *mm* = 10.48 *m*	* *IS* : 2502 *Table VIII*

Beam - B19, B20, B21 continued....

Step	Design Calculations	Reference
	Case - I : No Curtailment and No Bent-up Bars	
	Total number of stirrups = 14 of # 8*mm* and 44 of ϕ 6*mm*	
	Total weight of stirrups = 14 x 0.72 + 44 x 0.39 = 27.24 *kg*	
	Total weight of main bars = No. of bars x unit wt.x length in *m*.	
	= 6 x 3.853 x 10.48 = 242.3 *kg*	
	Total weight of main bars and stirrups = 242.3 +27.24 = 269.54 *kg*	
	Case - II : No Curtailment but use of Bent-up Bars :	
	Excess length of bent-up bar over straight bar	
	= vertical distance between top bar and bottom bar x (1.414–1).	
	Both the bent bars are in upper tier with its centre at a distance of 25 + 6 +25 + 25/2	
	= 68.5 *mm* from bottom face.	
	The top bars are at a distance 25 + 6 + 25/2 = 43.5 *mm* from top face.	
	Therefore, vertical distance between centre of top bar and centre of bent bar	
	= 700 – 68.5 – 43.5 = 588 *mm*	
	Excess length of each bent bar = 588 x (1.414 – 1) x 2 = 486 *mm* say 490 *mm*	
	Total length of each bent bar = 10480 + 490 = 10970 *mm* = 10.97 *m*	
	Total weight of all bars = 3.853 x (4 x 10.48 + 2 x 10.97) = 246.0 *kg*	
	Total weight of 66 Nos of ϕ 6 *mm* stirrups = 0.39 x 66 = 25.74 *kg*.	
	Total weight of main bars and stirrups = 246.0 + 25.74 = 271.74 *kg*.	
	Case - III : *Curtailment and No Bent-up Bars :*	
	Length of curtailed bars = 2(10000 / 2 – 1480) = 7040 *mm* = 7.04 *m*.	
	Total weight of main bars = 3.853 x (4 x 10.48 + 2 x 7.04) = 215.77 *kg*	
	Total number of stirrups = 24 of #8*mm* + 32 of ϕ6*mm*	
	Total weight of stirrups = 24 x 0.72+32 x 0.39 = 29.76 *kg*	
	Total weight of main bars and stirrups = 215.77 + 29.76 = 245.53 *kg*	
	Comparison of weight of steel.	

Case No.	wt of main bars	wt. of Stirrups	Total wt. in kg.
I	242.3	27.24	269.54
II	246.5	25.74	271.74
III	215.77	29.76	245.53

Step	Design Calculations	Reference
	Percentage saving for *Case - III* in comparison with *Case - I* = 9%	
	There is , practically, no difference between results of *Case - I* and *Case - II*.	
	It may be noted that in continuous beam, bent - up bars serve to act as negative moment steel at support.	
10.	*Load on column :*	
	Load on columns at each end = $V_{u.max}$ = 250 *kN*	
	Therefore, load on column *C8, C9, C10, C16, C17, C18*	
	= 250 *kN* at each end	

7.3.3.2 Category - II : Beams simply supported at one end and Continuous at the other. - Uniformly distributed load only.

Beam - B1 Simply Supported at One-End and Continuous at the Other

Step	Design Calculations	Reference
1.	*Beam Mark* : *B1* simply supported	
2.	*End Condition* : One end simply supported and the other continuous. $EC = 2$	
3.	*Span* : $L = 4.0\ m$,	
4.	*Section* : Assumed $b = 230\ mm$, $D = 380\ mm$, $d' = 40\ mm \therefore d = 340\ mm$ Nominal cover for mild environment $= 20\ mm$ Assuming maximum diameter of stirrups of 8 *mm* and diameter of main steel 20 *mm* effective cover $= d' = 20 + 8 + 20/2 = 38\ mm$ say $40\ mm$ $\therefore$ effective depth= $d - d' = 380 - 40 = 340\ mm$	*Table C-1*
5.	*Loads* : Self + Parapet 1 *m* high + *S2* (UDL) $w_u = 1.5\ (1.6 + 5.0 + 6 \times 2.5/2) = 21.15\ kN/m$	
6.	*Design Moment* : $M_{u.max} = w_u L^2/10 = 21.15 \times 4^2/10 = 33.84\ kN.m$ at midspan as well as at continuous end. $M_{ur.max} = 73.4\ kN.m > M_u = 33.84\ kN.m$ The section will be designed as singly reinforced rectangular section Provide 3 of # 12 *mm* in 1 row, $A_{st} = 339\ mm^2$ Check for width : Required $b = 3 \times 12 + (3+1) \times 20 + 2 \times 6 = 128\ mm < 230\ mm$ $\therefore$ safe Check for cover : Required $d' = 20+6+12/2 = 32\ mm$ < assumed 40 *mm* $\therefore$ safe Effective depth, $d = 380 - 32 = 348\ mm > 340\ mm$	*Sect. 7.3.2(7)*
7.	*Main Steel* : Required $A_{st} = \dfrac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \dfrac{4.6 \times 33.84 \times 10^6}{20 \times 230 \times 340^2}}\right] \times 230 \times 340$ $= 300\ mm^2$ at mid-span and at support $> A_{st.min}$ $(=160\ mm^2)$ *Detailing* : (see table below)	
8.	*Check for shear* : *At continuous end* $V_{u.max} = 0.6\ w_u L = 0.6 \times 21.15 \times 4 = 50.8\ kN$ $A_{stl} = 2$ -#12 + 2 -#10 , $V_{ur.min} = 60.93\ kN$ (which is for 2-12#) $\therefore$ Minimum stirrups are sufficient. $\therefore$ Provide ϕ 6 at 130 *mm* c/c *(b) At discontinuous end* : $V_{u.max} = 0.45\ w_u L = 0.45 \times 21.15 \times 4 = 38.1\ kN < V_{ur.min}$ (=60.93 *kN* for 2- #12) $\therefore$ Minimum stirrups ϕ 6*mm* at 130 *mm* c/c is adequate.	*Table 7.3.1*
9.	*Load on Column* : at continuous end column $C2 = V_{u.max} = 50.8\ kN$ at simply supported end column *C1* $= 38.1\ kN$	

Detailing :

	Simple Support	*Mid-span*	*Continuous support towards B2*
Top	2#10 + 1- #12 (bent)	2- # 10	2 # 10 + 1 # 12 (bent) +1-#12 (bent) from *B2*
Bottom	2 - #12	2#12+1 #12(to be bent)	2 - #12

Beam - B6 Simply Supported at One- End and Continuous at the Other

Step	Design Calculations	Reference
1.	*Beam Mark* : B6	
2.	*End Condition* : One end simply supported and the other continuous. $EC = 2$	
3.	*Span* : $L = 4\ m$.	
4.	*Section* : 230 *mm* x 380 *mm*.	
5.	*Loads* : Self + Slab *S2* (UDL) + Slab *S3* (half triangular load)	
	$w_u = 1.5\ (1.6 + 6 \times 2.5/2 + 6.5 \times 4.00/6) = 20.15\ kN/m$	
6.	*Design Moment* :	
	$M_u = 20.15 \times 4^2 / 10 = 32.24\ kN.m$ at mid-span and at continuous end.	
7.	*Main steel* :	
	M_{ur} provided by 3#12 = 37.87 $kN.m$ at mid-span > 32.24 $kN.m$	Table 7.3.1
	M_{ur} provided by 2#10 + 2#12 = 42.24 $kN.m > M_u$	

	Simple Support	Mid-Span	Continuous support towards B7
Top	2#10	2#10	2#10 + 1#12 (bent) +1#12 /2.7 *m*(Extra)
Bottom	2#12	2-#12 +1#12*(to be bent)	2#12

Step	Design Calculations	Reference
8.	*Check for shear : At continuous end* : A_{st} = 2#10 + 2#12	
	$V_{u.max} = 0.6\ w_u\ L = 0.6 \times 20.15 \times 4 = 48.36\ kN < V_{ur.min}$ (= 68.38 kN)	Table 7.3.1
	∴ Provide φ 6*mm* at 130 *mm*.c/c	
	At simply supported end : A_{st} = 2- #12	
	$V_{u.max} = 0.45 \times 20.15 \times 4 = 36.27\ kN < V_{ur.min}$ (= 60.93 kN)	
	∴ Provide minimum stirrups φ 6 *mm* at 130 *mm* c/c	
9.	*Load on column* :	
	(a) At Continuous end = 0.6 [1.5 (1.6 + 6.0 x 2.5/2)x 4] = 32.8 kN on column *C8*	
	(b) At Discontinuous end = 0.45[1.5(1.6) = 6 x 2.5/2 x4] = 24.6 kN on column *C7*	

Beam - B12 Simply Supported at One- End and Continuous at the Other

Step	Design Calculations	Reference
1.	*Beam Mark* : B12	
2.	*End Condition* : One end simply supported and the other continuous $EC = 2$	
3.	*Span* : $L = 4\ m$.	
4.	*Section* : 230 *mm* x 380 *mm*	
5.	*Loads* : Self + parapet + slab - *S3* (half triangular from one- way)	
	$w_u = 1.5\ (1.6+5+6.5 \times 4/6) = 16.4\ kN/m.$	
6.	*Design Moment* :	
	$M_u = 16.4 \times 4^2 / 10 = 26.24\ kN.m$ at mid-span and at continuous end.	
7.	*Main Steel* :	
	M_{ur} provided by 2#12 = 26.10 $kN.m \cong 26.24\ kN.m$ and	
	for 2#10 + 1#12, M_{ur} = 30.77 $kN.m$	Table 7.3.1

	Simple Support	Mid-Span	Continuous support towards B13
Top	2#10	2#10	2#10 +1#12 /2.7*m* (extra)
Bottom	2#12	2-#12	2#12

Beam - B12 continued....

Step	Design Calculations	Reference
8.	*Design for Shear* : (a) At Continuous End : $V_{u.max} = 0.6 \times 16.4 \times 4 = 39.36\ kN$ $A_{stl} = 2$ - #10 + 2-#12 , $V_{ur.min} = 68.38\ kN$ ∴ Provide Minimum stirrups ϕ 6 *mm* at 130 *mm c/c* (b) At simply Supported End : $V_{u.max} = 0.45 \times 16.4 \times 4 = 29.5\ kN$. A_{stl} = 2-#10 +1#12 for which, $V_{ur.min} = 63.00\ kN$. ∴ Provide minimum Stirrups ϕ 6 *mm* at 130 *mm*	Table 7.3.1
9.	*Load on column* : (a) At Continuous end = 0.6 [1.5(1.6 + 5)] x 4 = 23.8 *kN* on column *C16* (b) At Discontinuous end = 0.45 x 9.9 x 4 = 17.8 *kN* on column *C15*	

Beam - B16 Simply Supported at One- End and Continuous at the Other

Step	Design Calculations	Reference
1.	*Beam Mark: B16* :	
2.	*End Condition* : One end simply supported and other continuous. *EC* = 2	
3.	*Span* : $L = 4.5\ m$. Assumed Section 230 *mm* x 380 *mm* , $d = 340\ mm$	
4.	*Loads* : self + parapet + slab - *S4* (Tri. Two -way) $w_u = 1.5\,(1.6 + 5 + 6.5 \times 4.5/3) = 24.52\ kN/m$ $w_{us} = 1.5\,(1.6 + 5 + 6.5 \times 4.5/4) = 20.9\ kN/m$	
5.	*Design moments*, $M_u = 24.52 \times 4.5^2/10 = 49.6\ kN.m$ at mid-span and at continuous support. *Mid-span section* : *L-section* $L_o = 0.7 \times 4500 = 3150\ mm$ $b_f = L_o/12 + 3D_f + b_w = 3150/12 + 3 \times 130 + 230 = 882\ mm$ $(M_{ur})_{x_u = D_f} = 0.36 \times 20 \times 882 \times 130 \times (340 - 0.42 \times 130) \times 10^{-6}$ $= 235.6\ kN.m > M_u\ (= 49.6\ kN.m)$ ∴ $x_u < D_f$	Eq. 4.3.8
6.	*Main Steel* : Required $A_{st} = \dfrac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \dfrac{4.6 \times 49.6 \times 10^6}{20 \times 882 \times 340^2}}\right] \times 882 \times 340$ $= 416\ mm^2$	Eq. 4.3.5
	Provide 4#12 *mm* , Area provided = 452 mm^2 *Support Section* : Rectangular section $M_{ur.max} = 2.76 \times 230 \times 340^2 \times 10^{-6} = 73.4\ kN.m > M_u\ (49.6\ kN.m)$ ∴ Section is singly reinforced.	
	Required $A_{st} = \dfrac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \dfrac{4.6 \times 49.6 \times 10^6}{20 \times 230 \times 340^2}}\right] \times 230 \times 340$ $= 460\ mm^2$	Eq. 4.1.6a
	Provide 2 #10 + 1 #12 (bent) (to be continued for 1.35 *m* in span of *B15*) + 1#16 (extra) 2.7 *m* (1.35 *m* on both sides of support) Area provided = 471 mm^2	

Beam - B16 continued....

Step	Design Calculations	Reference
7.	*Main steel :*	

	Simple support	Mid-span	Continuous support
Top	2#10	2-#10	2-#10 + 1# 12 (bent) + 1#16/2.7 *m** (extra)
Bottom	3 #12	3#12 + 1 #12 bent	3#12

Step	Design Calculations	Reference
8.	*Design for Shear : (a) At continuous End* : A_{st} = 471 mm^2	
	$V_{u.max}$ = 0.6 x 20.9 x 4.5 = 56.40 kN. < $V_{ur.min}$ (= 70.77 kN for A_{st} = 452 mm^2)	Table 7.3.1
	∴ Provide minimum shear reinforcement of ϕ 6 *mm* @130 *mm* c/c	
	(b) At simply supported end :	
	$V_{u.max}$ = 0.45 x 20.9 x 4.5 = 42.3 kN < $V_{ur.min}$ (= 66.30 kN for 3#12)	Table 7.3.1
	∴Provide ϕ 6 *mm* @ 130 *mm*.c/c	
9.	*Load on Column :*	
	Continuous end column = 56.4 kN on column *C19*	
	Simply supported end column = 42.3 kN on column *C20*	

7.3.3.3 Category - III : Beams - B2, B3, B4 : Beam Continuous at Both Ends

Step	Design Calculations	Reference
1.	*Beam Mark* : *B2, B3, B4*	
2.	*End Condition* : Continuous at Both Ends. *EC* = 3	
3.	*Span* : L = 4 *m* = 4000 *mm*.	
4.	*Section* : Assumed b = 230 *mm* , D = 380 *mm* , d' = 40 *mm* , d = 340 *mm*.	
5.	*Loads* : Same as that for Beam *B1*. $w_u = w_{us}$ = 21.15 kN/m	
6.	*Design Moment :*	
	$M_u = w_u L^2/12 = 21.15 \times 4^2 / 12 = 28.2$ kN.m < $M_{ur.max}$(=73.4 kN.m)	
	∴ It can be designed as rectangular section	
	Required $A_{st} = \frac{0.5 \times 20}{415}\left[1-\sqrt{1-\frac{4.6 \times 28.2 \times 10^6}{20 \times 230 \times 340^2}}\right] \times 230 \times 340$	Eq. 4.1.6a
	= 246 mm^2	
	Provide 3-#12 *mm* , Area Provided = 339 mm^2	
7.	*Main steel :* (For illustration details are given for beam *B2*)	

	Continuous towards B1	Mid-span	Continuous end towards B3
Top	2 - #10 + 1 - #12 bent + 1#12 (from *B1*)	2-#10	2#10+1#12 bent* +1#12 bent (from *B3*)
Bottom	2-#12	2-#12 +1#12bent*	2-#12

Remarks :

i) For beam B3 the bars available at top = 2#10 +2#12 (bent from adjacent spans)

ii) For beam B4, at the continuous support common to B4 and B5 the details of reinforcement are given in design of beams B5, since B5 is subjected to loads of different intensities.

Beams - B2, B3, B4 continued....

Step	Design Calculations	Reference
8.	*Design for Shear* : At both supports : A_{stl} = 2-#12 +2 - #10. $V_{u.max}$ = 21.15 x 4/2 = 42.30 *kN* < $V_{ur.min}$ (=68.38 *kN*) ∴ Provide minimum stirrups ϕ 6 *mm* at 130 *mm*. c/c	Table 7.3.1
9.	*Load on column* : At each end = 42.3 *kN* on columns *C2, C3, C4, C5*	

Beams - B7, B8, B9 Continuous at Both Ends

Step	Design Calculations	Reference
1.	*Beam Mark* : *Beams - B7, B8, B9*	
2.	*End condition* : Beam continuous at both ends *EC* = 3	
3.	*Span* : *L* = 4.0 *m* = 4000 *mm*.	
4.	*Section* : 230 *mm* x 380 *mm*	
5.	*Loads* : w_u = 20.15 *kN/m* same as that for beam *B6*	
6.	*Design Moment* : M_u = 20.15 x 4²/12 = 26.9 *kN.m* < $M_{ur.max}$ = 73.4 *kN.m* $A_{st} = \frac{0.5 \times 20}{415}\left[1-\sqrt{1-\frac{4.6 \times 26.9 \times 10^6}{20 \times 230 \times 340^2}}\right] \times 230 \times 340$ = 234 mm^2	Eq. 4.1.6a
	Provide 2#12 at bottom , Area provided = 226 mm^2 ≅ 234 mm^2 Provide 2#10 +1#12 extra at support,Area provided = 270 mm^2 > 234 mm^2 M_{ur} = 30.77 *kN.m*	Table 7.3.1
7.	*Main steel* :	

	Continuous end towards B6	Mid-span	Continuous ends towards B8
Top	2#10 *1-#12 /2.7 *m*	2-#10	2-#10 + *1#12/2.7 *m*
Bottom	2-#12	2-#12	2-#12

*At top of supports of *B7* (towards*B8*), *B8, B9* (towards *B8*) provide extra top bars of 1-#12 of length 2.7**m* 1.35 *m* on both sides of centre of span.For details of reinforcement at common supports of *B9* and *B10* see design of *B10*.

Step	Design Calculations	Reference
8.	*Design for Shear* : At supports : A_{stl} = 2-#10*mm* + 1-# 12*mm* . $V_{u.max}$ = w_u *L*/2 = 20.15 x 4.0/2 = 40.3 *kN*. < $V_{ur.min}$ (=63*kN*) ∴ Provide minimum stirrups ϕ 6 at 130 *mm* c/c	Table 7.3.1
9.	*Load on column* : Load on column = 0.5[1.5(1.6+6 x2.5/2)]x 4 = 27.30*kN*,on columns *C8,C9,C10,C11*	

III (c) Beams - B13, B14, B15 Continuous at both Ends

Step	Design Calculations	Reference
1.	*Beam Mark* : *B13, B14, B15*	
2.	*End Condition* : Beam continuous at both ends, *EC* =3	
3.	*Span* : *L* = 4 *m* = 4000 *mm*.	
4.	*Section* : 230 *mm* x 380 *mm*. , *d* = 340 *mm*	
5.	*Loads* : w_u = 16.4 *kN/m* - Same as that for Beam *B12*	

Beam - B13, B14, B15 continued....

Step	Design Calculations	Reference
6.	*Design Moment :* $M_u = 16.4 \times 4^2 / 12 = 21.9$ *kN.m* at mid-span and at supports. $A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 21.9 \times 10^6}{20 \times 230 \times 340^2}}\right] \times 230 \times 340$ $= 188\ mm^2 > A_{st.min}\ (=160\ mm^2)$	*Eq. 4.1.6a*
7.	*Main Steel :* (see table below)	
8.	*Design for Shear :* $V_{u.max} = w_u \times L/2 = 16.4 \times 4/2 = 32.8\ kN < V_{ur.min}$ (= 60.93 kN for 2#12) ∴ Provide minimum stirrups ϕ 6 *mm* at 130 *mm.* c/c	*Table 7.3.1*
9.	*Load on Column :* Load on Column : 0.5 [1.5(1.6 + 5)x 4] = 19.80 *kN* columns *C16,C17,C18,C19*	

	Continuous end	Mid-span	Continuous end
Top	2#10+1-#12/2.7*m*(Extra)	2#10	2#10 + 1#12/2.7*m*(Extra)
Bottom	2#12	2#12	2#12

7.3.3.4 *Category - IV Miscellaneous Beam : Beam B22-B23*

Step	Design Calculations	Reference
1.	*Beam Mark : B22 - B23* * The beam is analysed and designed by : *(i) Exact method without redistribution of moments.* *(ii) Exact method with redistribution of moments.* *(iii) Approximate method and results compared.*	
2.	*End Condition :* Two Span Continuous Beam with equal spans. Ends simply supported and results are compared	
3.	*Span* : $L = 5\ m = 5000\ mm$ each.	
4.	*Section*: Assumed 230 *mm* x 380 *mm* ,$d = 340\ mm$, $d' = 40\ mm$,$D_f = 130\ mm$	
5.	*Loads* : Maximum load 1.5 x *(DL + LL)* and minimum load (*DL* only) *Slab Left - S3* Type of slab - One -way : $L_x = 4.0\ m$,End condition-Continuous at both ends. End shear factor = 0.55 : Load Type-UD load. $DL + LL = 6.5\ kN/m^2$ only $DL = 5\ kN/m^2$ *Slab Right - S4* Type of Slab : Two -way : $L_x = 4.5\ m$, $L_y = 5\ m$ End condition - Continuous over *B22 - B23* and Simply Supported at ends. End shear factor = 0.6: Load Type - Trapezoidal. $DL + LL = 6.5\ kN/m^2$: only $DL = 5\ kN/m^2$ Equivalence factor for converting trapezoidal load into *UD* load $k_l = [1\text{-}1/(3\beta^2)]$ $\beta = L_y / L_x = 5.0 / 4.5 = 1.11$, $\therefore k_l = [1 - 1/(3 \times 1.11)]^2 = 0.73$ -Self (beam) $w_b = 1.6\ kN/m.$ Wall load = Nil	*Eq. 5.3.5*

Beam - B22, B23 continued....

Step	Design Calculations	Reference
6.	-Maximum load $w_{u.max}$ = 1.5 (DL + LL) = 1.5 (self + slab S3 + slab S4) $w_{u.max}$ = 1.5 [1.6 + 0.55 x 6.5 x 4 + 0.6 x 6.5 x 4.5 x 0.73] = 43 kN/m $w_{u.min}$ = DL = [1.6 + 0.55 x 5 x 4 + 0.6 x 5.0 x 4.5x0.73] = 22.4kN/m. *Design Moments :* ***(A) Analysis by Exact Method without allowing Redistribution of Moments*** In this analysis, we will have to consider all possible loading arrangements to get values of (maximum) design moments at mid-span and at support. *Case - I : Maximum Bending Moment at Intermediate Support* This is given by maximum loads on both the spans. For this loading, $M_u = w_{u.max} L^2/8 = 43 \times 5^2/8 = 134.4$ kN.m *Case - II : Maximum Bending Moment at Middle of Left Span* This is given by maximum load on left span and minimum on the other. Since section, span and end conditions at the far end are the same for both the spans AB and BC the distribution factor at intermediate joint B for each of the two spans is equal to 1/2. *Fixed End Moments :* $M_{FBA} = 43 \times 5^2/8$ = 134.4 kN.m $M_{FBC} = 22.4 \times 5^2/8$ = 70 kN.m *Moment Distribution :*	

Joint	A	B		C
Member	AB	BA	BC	CB
Distribution Factors	0	0.5	0.5	
Initial F.E.M.		134.4	− 70.0	0
Distr.		−32.2	− 32.2	
Final Moments in kN.m	0	102.2	− 102.2	0

Maximum Span Moment :

V_{AB} = 43 x 5/2 − 102.2/5 = 87.1 kN , V_{BA} = 43 x 5 − 87.1 = 127.9 kN

x_{max} = 87.1 / 43 = 2.02 m , L_o = 2 x 2.02 = 4.04m

M_{max1} = 87.1 x 2.02/2 = 88.0 kN.m

Case-III : Maximum Bending Moment at Middle of Right Span

This is given by maximum load on the right span and minimum on the left span.

By symmetry, M_{max2} = 88.0 kN.m

Thus, the design moments are 134.4 kN.m at intermediate support and 88.0kN.m at middle of both the spans.

(B) Analysis by Exact Method allowing Redistribution of Moments

Allowing 30% redistribution at intermediate support,

Design moment at support = 0.7 x 134.4 = 94.08 kN.m

Mid-span moment : V_{AB} = 43 x 5/2 − 94.08/5 = 88.7 kN

V_{BA} = 43 x 5 − 88.7 = 126. 3 kN

x_{max} = 88.7/43 = 2.062 m , L_o = 2 x 2.062 = 4.124 m

$M_{u.max}$ = 88.7 x 2.062/2 = 91.45 kN.m

Beam - B22, B23 continued....

Step	Design Calculations	Reference
	(C) Analysis by Approximate Method Design Moment at support as well as at mid-span $= \pm w_u L^2/10$ $M_{max} = 43 \times 5^2/10 = \pm 107.5\ kN.m$ $V_{AB} = 43 \times 5/2 - 107.5/5 = 86\ kN$ $V_{BA} = 43 \times 5 - 86 = 129\ kN$	
7.	*Main Steel :* *(A) For Exact Analysis without Redistribution of Moments :* $M_{ur.max} = 2.76 \times 230 \times 340^2 \times 10^{-6} = 73.4\ kN.m$	Sect. 7.3.2
	(a) At Intermediate Support : $M_{u.max} = 134.4\ kN.m > 73.4\ kN.m$ ∴ The section will be required to be Doubly Reinforced $M_{u2} = M_{u.max} - M_{ur.max} = 134.4 - 73.4 = 61\ kN.m$ *Tension Steel :* *(a) At support :* $x_{u.max} = 0.48 \times 340 = 163.2\ mm$	
	$A_{st1} = \dfrac{73.4 \times 10^6}{0.87 \times 415 \times (340 - 0.42 \times 163.2)} = 749\ mm^2$	Eq.4.2.3a
	Assuming $d_c = 40\ mm$ $A_{st2} = \dfrac{61 \times 10^6}{0.87 \times 415 \times (340 - 40)} = 563\ mm^2$	Eq.4.2.3b
	$\therefore A_{st} = A_{st1} + A_{st2} = 749 + 563 = 1312\ mm^2$ *Compression Steel :* $d_c/d = 40/340 = 0.117$ for which , $A_{sc} = 1.063 \times 563 = 599\ mm^2$	Table 4.2.3
	(b) At Midspan : $M_{u.max} = 88\ kN.m > 73.4\ kN.m$ ∴ Assistance of flange will be taken in resisting bending moment $b_f = (4040/6 + 6 \times 130) + 230 = 1683\ mm$ $(M_{ur})_{x_u = D_f} = 0.36 \times 20 \times 1683 \times 130\ (340 - 0.42 \times 130) \times 10^{-6}$ $= 449.6\ kN.m > M_{u.max}\ (= 88\ kN.m)$ $\therefore x_u < D_f$ and beam acts like a rectangular section with $b = b_f$.	Eq.4.3.1 Eq.4.3.8
	Also , $x_{u.max} = 0.48\ d$ for M20-Fe415 $= 0.48 \times 340 = 163\ mm > D_f$. ∴ o.k.	Table 4.1.1
	$A_{st} = \dfrac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \dfrac{4.6 \times 88 \times 10^6}{20 \times 1683 \times 340^2}}\right] \times 1683 \times 340$ $= 737\ mm^2$	
	(c) Detailing of Bars : (see table below)	

Required A_{st} mm^2	At mid-span	At intermediate support
At top	---	1312
At bottom	737	599

Beam - B22, B23 continued....

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<tr><th>Step</th><th>Design Calculations</th><th>Reference</th></tr>
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(B) For Exact Analysis with 30% Redistribution of Moments :

(a) At intermediate Support :

$M_{u.max} = 94.08\ kN.m$, $k_{u.limit} = 0.6 - 0.3 = 0.3$

$x_{u.limit} = 0.3 \times 340 = 102\ mm$

Since redistribution of 30% is allowed ,

$M_{ur.limit} = [0.36 f_{ck}\ k_{u.limit}\ (1 - 0.42\ k_{u.limit})]\ bd^2$

$= 0.36 \times 20 \times 0.3 \times (1 - 0.42 \times 0.3) \times 230 \times 340^2 \times 10^{-6}$

$= 50.2\ kN.m$

∴ The section will be required to be Doubly Reinforced

$M_{u2} = 94.08 - 50.2 = 43.88\ kN.m$

Tension steel :

$A_{st1} = \dfrac{50.2 \times 10^6}{0.87 \times 415 \times (340 - 0.42 \times 102)\ 100} = 468\ mm^2$

$A_{st2} = 43.88 \times 10^6 / [0.87 \times 415 \times (340 - 40)] = 405\ mm^2$

$A_{st} = A_{st1} + A_{st2} = 468 + 405 = 873\ mm^2$

Compression steel :

$d_c / d = 40/340 = 0.117$ for which

$A_{sc} = 1.063 \times 405 = 431\ mm^2$

(b) At mid-span :

$M_{u.max} = 91.45\ kN.m > 73.4\ kN.m$

∴ The assistance of flange will have to be taken.

$b_f = (4124/6 + 6 \times 130) + 230 = 1697\ mm$

As seen in solution (A) above, $x_u < D_f$ even for $b_f = 1683\ mm$

∴The in this case also, $x_u < D_f$ and the beam acts like a rectangular beam with $b = b_f$

$A_{st} = \dfrac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \dfrac{4.6 \times 91.45 \times 10^6}{20 \times 1697 \times 340^2}}\right] \times 1697 \times 340 = 766 mm^2$

Required area in mm^2	At mid-span	At support
At top	–	873
At bottom	766	431

(C) For Approximate Analysis ($w_u L^2$ /10 at mid-span and at support)

(a) At support :

$M_{u.max} = w_{u.max} L^2/10 = 43 \times 5^2/10 = 107.5\ kN.m$ (= 0.8 x 134.4)

(This correspond to 20% redistribution of moment)

$k_{u.limit} = 0.6 - 0.2 = 0.4$, $x_{u.limit} = 0.4 \times 340 = 136\ mm$

For this, $M_{ur.limit} = 0.36 \times 20 \times 0.4 \times (1-0.42 \times 0.4) \times 230 \times 340^2 \times 10^{-6}$

$= 63.7\ kN.m < 107.5\ kN.m$

∴ The section will be required to be doubly reinforced.

$M_{u2} = 107.5 - 63.7 = 43.8\ kN.m$

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Eq. 4.1.5d

Table 4.2.3

Eq. 4.3.1

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Beam - B22, B23 continued....

Step	Design Calculations	Reference
	Tension steel :	
	$A_{st1} = \dfrac{63.7 \times 10^6}{0.87 \times 415 \times (340 - 0.42 \times 136)} = 624\ mm^2$	
	$A_{st2} = \dfrac{43.8 \times 10^6}{0.87 \times 415 \times (340 - 40)} = 404 mm^2$	
	$\therefore A_{st} = A_{st1} + A_{st2} = 624 + 404 = 1028\ mm^2$	
	$d_c/d = 40/340 = 0.117,\ \therefore A_{sc} = 1.063 \times 404 = 430\ mm^2$	Table 4.2.3
	(b) At mid-span :	
	$M_{u.max} = 107.5\ kN.m > M_{ur.max} = 73.38\ kN.m\ (= 2.76 \times 230 \times 0.34^2)$	
	Therefore, assistance will have to be taken of the flange.	
	$L_o = 0.7\ L = 0.7 \times 5000 = 3500\ mm$	
	$b_f = (3500/6 + 6 \times 130) + 230 = 1593\ mm$	
	$(M_{ur})_{x_u = D_f} = 0.36 \times 20 \times 1593 \times 130 \times (340 - 0.42 \times 130) \times 10^{-6}$	
	$= 425 kN.m > M_{u.max}\ (= 107.5\ kN.m) \quad \therefore x_u < D_f$	
	$A_{st} = \dfrac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \dfrac{4.6 \times 107.5 \times 10^6}{20 \times 1593 \times 340^2}}\right] \times 1593 \times 340 = 908 mm^2$	
	(c) Detailing :	

Required A_{st} in mm^2	Simple support	At Mid-span	At Continuous end
At top	----	-----	1028
At bottom	----	908	430
No. - Dia.At top	2#10	2-#10	2 – #10 + 3 – #20
NO. - Dia.At bottom	3#20	3-#20	2-#20
Provided A_{st} at top	157	157	1099
at bottom	942	942	628

Comparision of Three Methods :

(a) Bending Moment and Shear Force

Method	Bending Moment in kN.m			Shear force in kN	
	AB	Mid-span	BA	AB	BA
Exact Method	0	88.00	-134.4	87.1	127.9
30 % Redistribution	0	91.45	-94.1	88.7	126.3
Approximate Method	0	107.5	-107.5	86.0	129.0

(b) Area of Main Steel in mm^2

		Mid-span	Support
Exact Method	Top	---	1312
	Bottom	737	599
30 % Redistribution	Top	---	873
	Bottom	766	431
Approximate Method	Top	---	1028
	Bottom	908	430

Beams - B22, B23 continued....

Step	Design Calculations	Reference
8.	*Design for shear : (for approximate method)* Since the beam is supporting a two-way slab *S4*, the equivalent uniformly distributed load for shear is different than that considered for bending. The equivalence factor = k_2 =[1−1/(2 β)] = [1 − 1/(2 x 1.11)] = 0.55 Total UD load on beam for shear w_{us} = self load from slab *S3* + load from slab *S4* w_{us} = 1.5 [1.6 + 0.55 x6.5 x 4 + 0.6 x 6.5 x 4.5 x 0.55] = 38.3 *kN/m*	Eq. 5.3.6
	(a) At continuous End : $V_{u.max}$ = 0.6 w_{us} L = 0.6 x 38.3 x 5 = 114.9 *kN* for design moment of $w_u L^2/10$ A_{stl} = 1099 mm^2 , p_t = 100 x 1099 / (230 x 340) = 1.4 %, τ_{uc} = 0.7 N/mm^2 V_{uc} = 0.7 x 230 x 340 / 1000 = 54.74 *kN* $V_{usv.min}$ = 0.4 x 230 x 340 / 1000 = 31.28 *kN* $V_{ur.min}$ = 54.74 + 31.28 = 86.02 *kN.m* < $V_{u.max}$ (=119.7 *kN*) V_{uD} = 119.7 − 38.3 (0.23/2+.340)= 102.3 *kN* > $V_{ur.min}$ (= 86.02 *kN.m*) ∴ Design stirrups will be required to be designed. V_{us} = $V_{uD} - V_{uc}$ = 102.3 − 54.74 = 47.6 *kN* Assuming #8 *mm* - 2 legged Fe415 grade stirrups (A_{sv} = 100 mm^2) Spacing *s* = 0.87 x 415x100 x340/(47.6 x 1000) = say 250 *mm* <(0.75*d* and 300*mm*) Length of shear Zone L_{sl} = ($V_{u.max} - V_{ur.min}$)/w_{us} = (119.7−86.02)/38.3 = 0.88 *m* ∴Provide #8*mm* at 250 *mm* c/c for a length 900*mm*(5Nos.)from the centre of support Provide minimum stirrups φ 6*mm* at 130 *mm* c/c beyond	Table 4.4.1
	(b) At simply Supported End : $V_{u.max}$ = 0.45 w_{us} L = 0.45 x 38.3 x 5 = 86.2 *kN* A_{stl} = 3-#20 = 942 mm^2 , p_t = 100 x 942 / (230 x 340) = 1.2% , τ_{uc} = 0.66 N/mm^2 V_{uc} = 0.66 x 230 x 340/1000 = 51.6 *kN* $V_{usv.min}$ = 31.28 *kN* , ∴ $V_{ur.min}$ = 51.6 + 31.28 = 87.9 *kN* $V_{u.max}$ < $V_{ur.min}$ ∴ Minimum stirrups are sufficient. Provide φ 6*mm* - 2 legged stirrups at 130 *mm* c/c	Table 4.4.1 Table 7.3.1
9.	*Check for deflection :* p_t % = 100 A_{st} / ($b_f d$) = 100 x 942 / (1593 x 340) = 0.17% f_s = 0.58 x 415 x 908/942 = 232 N/mm^2 ∴ α_1 = 1.9 b_w / b_f = 230 / 1593 = 0.14 < 0.3 ∴ α_3 = 0.8 ∴ d = $\frac{5000}{26 \times 1.9 \times 0.8}$ = 126 *mm* < 340 *mm* ∴ safe	Fig. 4.7.1 Fig. 4.7.3
10.	*Load on Column :* At continuous end $V_{u.max}$ = 114.9 *kN* on column *C13* At discontinuous end $V_{u.max}$ = 86.2 *kN* on column *C19, C11*	

7.3.3 Miscllaneous : Beam B24 - B25

Step	Design Calculations	Reference
1.	*Beam Mark :* Beams B24 - B25 :	
2.	*End Condition:* Two span continuous beam equal spans - Ends simply supported- *Approximate Method*	
3.	*Span :* 5 m = 5000 mm each.	
4.	*Section* : 230 x 380 mm , d = 340 mm , d' = 40 mm	
5.	*Loads* : Self + parapet wall + S4 (Trapezoidal) For bending, $w_{u.eqb} = 1.5 \times [1.6+5+0.45 \times 6.5 \times 4.5\,(1-1/(3 \times 1.11^2))] = 24.3\ kN/m$ For shear , $w_{u.eqs} = 1.5 \times [\,1.6+5+0.45 \times 6.5 \times 4.5\,(1-1/(2 \times 1.11))] = 20.8\ kN/m$	
6.	*Design Moment :* $M_u = 24.3 \times 5^2/10 = 60.75\ kN.m$ at support as well as mid-span.	Sect. 7.3.2(7)
7.	*Main Steel :* *(a) At intermediate support :* $M_u = w_u L^2/10 = 60.75 kN.m = (0.8 \times 24.3^2/8)$ *i.e* 20% redistribution $k_{u.limit} = 0.4$ $M_{ur.limit} = 0.36 \times 20 \times 0.4 \times (1-0.42 \times 0.4) \times 230 \times 340^2 \times 10^{-6}$ $= 63.7\ kN.m > M_u\ (=60.75\ kN.m)$ ∴ The section is singly reinforced $A_{st} = \frac{0.5 \times 20}{415}\left[1-\sqrt{1-\frac{4.6 \times 60.75 \times 10^6}{20 \times 230 \times 340^2}}\right] \times 230 \times 340$ $= 587\ mm^2$ *(b) At Mid-span Section :* $M_u = 60.75\ kN.m$ The beam is designed as L - beam , $L_o = 0.8 \times 5000 = 4000\ mm$ $b_f = 4000/12 + 3 \times 130 + 230 = 953\ mm.$ For $x_u = D_f$, $M_{url} = 0.36 \times 20 \times 953 \times 130\,(340 - 0.42 \times 130) \times 10^{-6}$ $= 254.6\ kN.m > M_u \quad \therefore x_u < D_f$ $A_{st} = \frac{0.5 \times 20}{415}\left[1-\sqrt{1-\frac{4.6 \times 60.75 \times 10^6}{20 \times 953 \times 340^2}}\right] \times 953 \times 340$ $= 512\ mm^2$ *(c) Detailing :*	Eq. 4.3.1 Eq. 4.3.8 Eq. 4.3.1

Required A_{st}	At mid-span	At continuous end
At top	---	587 mm^2
At bottom	512 mm^2	
Provide At top - No. Diam (mm)	2-#12	2-#12+2-#16*
At bottom - No. Diam (mm)	2-#10+1-#16+1#16*(bent)	2-#10+1#16
Provided A_{st} At top- area in mm^2	226	628
At bottom - area in mm^2	559	358

* Bent - up from 1250mm to intermediate support from adjacent spans.

Beams B24 - B25 continued....

Step	Design Calculations	Reference
8.	*Design for Shear :*	
	(a) At Continuous End : $V_{u.max} = 0.6\ w_{us}\ L = 0.6 \times 20.8 \times 5 = 62.4\ kN$	
	$A_{st1} = 628\ mm^2$, $V_{ur.min} = 69.17\ kN$ even for A_{st1} = 2-#16	Table - 7.3.1
	∴ Minimum stirrups are sufficient. Provide ϕ 6 mm at 130 mm c/c	Table - 7.3.1
	(b) At Simply Supported End: $V_{u.max} = 0.45\ w_{us} L = 0.45 \times 20.80 \times 5 = 46.8\ kN.$	
	Since A_{st1} at discontinuous end is also 2-#16+2-#10, just as at continuous end, and since minimum stirrups are sufficient at continuous end, they are sufficient at discontinuous end too.	
9.	*Check for Deflection :*	
	Actual L/d ratio = 5000/340 = 14.7 << basic L/d ratio = 26 ∴ safe	
10.	*Load on Column :* - on column C20 and C12 = 46.8 kN	
	- on column C14 = 62.4 kN	

Miscellaneous Beam B17 - B18

Step	Design Calculations	Reference
	The beam is designed by three alternative methods and results compared	
	Alternative I : Two span continuous beam with beam *B17* size 230 *mm* x 700 *mm* and *B18 = 230 mm x 380 mm*	
	Alternative II : Two span continuous beam with beam *B17* size 230 *mm* x 700 *mm* and *B18 = 230 mm x 700 mm*	
	Alternative III : B17 size 230 *mm* x 700 *mm* and *B18* size 230 *mm* x 380 *mm* *both simply supported at ends*	
1.	*Beam Mark : B17 - B18*	
2.	*End Condition :* Two span continuous beam *ABC* with unequal spans *AB* and *BC*, simply supported at both ends *A* and *C* and continuous over an intermediate support *B*.	
3.	*Spans : AB = 10 m, BC = 2.5 m.*	
4.	*Sections :*	
	Span *AB* : 230mm × 700mm, $d' = 70mm$, $d = 630mm$, $D_f = 130mm.$	
	Span *BC* : 230mm × 380mm, $d' = 40mm$, $d = 340mm$, $D_f = 110mm.$	
5.	*Loads:* $w_{max} = 1.5(DL + LL)$, $w_{min} = DL$	
	Span AB: Self + parapet wall + Slab *S3(UDL)* from one side only.	
	Self weight of beam = 25 × .23 × (0.70 − 0.13) = 3.28 kN/m.	
	Weight of parapet wall = 5 kN/m.	
	Slab *S3* : Dead load = 5.00 kN/m^2, $LL = 1.5\ kN/m^2$, Total load = 6.50 kN/m^2	Sect. 7.2.3(5)
	$\therefore w_{max} = 1.5\,(3.28 + 5 + 0.45 \times 6.5 \times 4)$ = **30.0 kN/m**	
	$w_{min} = (3.28 + 5 + 0.45 \times 5.0 \times 4)$ = **17.3 kN/m**	
	Span BC : Self + parapet wall + cantilever Slab *S1*+ Slab *S2* (*Tri*) from other side.	
	Slab S1 : Dead load = 4.25 kN/m^2, $LL = 0.75\ kN/m^2$, Total load = 5.00 kN/m^2	Sect. 7.2.1(5)
	Slab S2 : Dead load = 4.50 kN/m^2, $LL = 1.50\ kN/m^2$, Total load = 6.00 kN/m^2	Sect. 7.2.2(5)
	The equivalent *UD* load = $wL_x/6$.	Eq. 5.3.1a
	Self weight of beam = 1.6 kN/m	Sect. 7.3.2(5c)
	$w_{max} = 1.5\,(1.6 + 5 + 5.00 \times 1 + 6.0 \times 2.5/6)$ = **21.2 kN/m**	
	$w_{min} = (1.6 + 5 + 4.25 \times 1 + 4.5 \times 2.5/6)$ = **12.7 kN/m**	

Beam B17 - B18 continued....

Step	Design Calculations	Reference
6.	*Design Moment*	

Alternative - I Continuous Beam - B17 (230 mm x 700 mm) , B18 (230 mm x 380 mm)

According to exact method of analysis , three different loading cases will be required to be considered. Actual moment will be calculated by using moment distribution method. For this, first of all, distribution factors will be calculated.

Since both the spans AB and BC act as flanged beams, the stiffness of flanged beam equal to twice the stiffness of rectangular beam can be taken *(see Sect.3.2.5)*.However, since the span AB is very large compared to span BC, the span BC will practically be subjected to negative $B.M.$ and therefore flange in portion BC will not be effective and moment of inertia of beam BC will be taken equal to that of rectangular section.

Beam AB : $K = (2 \times 230 \times 700^3/12)/10000 = 1315 \times 10^3\ mm^2$.

Beam BC : $K = (230 \times 380^3/12)/2500 = 421 \times 10^3\ mm^2$

Dist.Factors : $d_{BA} = 1315/(1315+421) = 0.76$, $d_{BC} = 1-0.76 = 0.24$

Case-I: Maximum Moment at Intermediate Support - B

For this, there shall be maximum load on both spans.

Load on AB , $w_1 = 30.0\ kN/m$, Load on BC, $w_2 = 21.2\ kN/m$

Fixed End Moments :

$$M_{FBA} = 30.0 \times 10^2/8 = 375.0\ kN.m$$

$$M_{FBC} = 21.2 \times 2.5^2/8 = 16.6\ kN.m$$

Moment Distribution :

Joint	B			C
Member	AB	BA	BC	CB
Dist. Factors	0	0.76	0.24	-
FEM	0	375.0	–16.6	0
Distri.		–272.4	–86.0	-
Final Moment	0	102.6	–102.6	0

Shear : Maximum shear occurs at the intermediate support

$$V_{AB} = 30 \times 10/2 - 102.6/10 = 139.74\ kN$$

$$V_{BA} = 30 \times 10/2 + 102.6/10 = 160.26\ kN$$

$$V_{BC} = 21.2 \times 2.5/2 + 102.6/2.5 = 67.54\ kN$$

$$V_{CB} = 21.2 \times 2.5/2 - 102.6/2.5 = -14.54\ kN$$

Case-II : Maximum Span Moment in AB

For this, there shall be maximum load on span AB and minimum load on span BC.

Maximum Load on AB $w_1 = 30.0 kN/m$, Minimum Load on BC $w_2 = 12.7\ kN.m$

Fixed End Moments : $M_{FBA} = 375.0\ kN.m$ as in *Case - I* since load is the same.

$$M_{FBC} = 12.7 \times 2.5^2/8 = 9.9\ kN.m$$

Distribution factors remain unchanged.

Beam B17 - B18 continued....

Step	Design Calculations	Reference

Moment Distribution :

Joint	A	B		C
Member	AB	BA	BC	CB
Dist. Factors	-	0.76	0.24	-
FEM	0	375.0	−9.9	0
Distri.		−277.5	−87.6	-
Final Moment	0	97.5	−97.5	0

$V_{AB} = 30 \times 10/2 - 97.5/10 = 140.25\ kN$

$V_{BA} = 30 \times 10/2 + 97.5/10 = 159.75\ kN$

$V_{BC} = 12.7 \times 2.5/2 + 97.5/2.5 = 54.87\ kN$

$V_{CB} = 12.7 \times 2.5/2 - 97.5/2.5 = -23.12\ kN$

$x_{max} = 140.25/30 = 4.675m,\ L_o = 2 \times 4.675 = 9.35\ m$

Maximum moment in AB $= 140.25 \times 4.675/2 = 327.83\ kN.$

Maximum moment at mid-span of BC $= -23.12 \times 1.25 - 12.7 \times 1.25^2/2 = -38.8\ kN/m$

Case-III : Maximum Span Moment in BC

For this, there shall be minimum load on AB and maximum load on BC.

Load on AB , $w_1 = 17.3\ kN/m$, Load on BC, $w_2 = 21.2\ kN/m$

Fixed End Moments : $M_{FBA} = 17.3 \times 10^2/8 = 216.2\ kN.m$

$M_{FBC} = 21.2 \times 2.5^2/8 = 16.6\ kN.m$

Distribution factors remain unchanged.

Moment Distribution :

Joint	A	B		C
Member	AB	BA	BC	CB
Dist. Factors	-	0.76	0.24	-
Initial FEM	0	216.2	−16.6	0
Distri.Moments	-	−151.7	−47.9	-
Final M. in kN.m	0	64.5	−64.50	0

Shear : Shear at C will be maximum in this case.

$V_{CB} = 21.2 \times 2.5/2 - 64.5/2.5 = 0.7\ kN$

$V_{BC} = 21.2 \times 2.5/2 + 64.5/2.5 = 52.3\ kN$

Mid - span moment $= 0.7 \times 2.5/2 - 21.2 \times 1.25^2/2 = -15.7\ kN.m$

Results:

Max. values of	A	Midspan	B		Midspan	C	
Max. Moment kN.m	0	327.83	−102.6		−38.8	0	
Governing Case	-	II	I		II	-	
Maximum Shear kN	140.25		160.26	67.54		−23.12	0.7
Governing Case	II		I	I		II	III

Beam B17 - B18 continued.....

Step	Design Calculations	Reference
7.	*Main Steel* :	
	(a) At middle of Span AB : M_u = 327.83 kN.m	
	The section shall be designed as flanged (L) section.	
	Maximum moment at midspan has occurred for *Case - II*. Therefore,	
	L_o for this case will be taken for calculation of b_f	
	For Case - II , $L_o = 9.35m = 9350\ mm$.	
	$b_f = 9350/12 + 3 \times 130 + 230 = 1399\ mm$.	
	For $x_u = D_f$, $M_{ur1} = 0.36 \times 20 \times 1399 \times 130 \times (630 - 0.42 \times 130) \times 10^{-6}$	
	$= 753\ kN.m > 327.83\ kN.m \quad \therefore x_u < D_f$	
	$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 327.83 \times 10^6}{20 \times 1399 \times 630^2}}\right] \times 1399 \times 630$	
	$= 1495\ mm^2$	
	(b) (i) At Intermediate Support (Right side) : M_u = 102.6kN.m	
	Since the beam sections on two sides of support are different, the area of steel will be calculated for both sides of the section.	
	(i) For section 230 × 380 mm of span BC, $d = 340\ mm$	
	For this section $M_{ur.max} = 73.4\ kN.m < M_u\ (=102.6\ kN.m)$	Sect.7.3.2(7)
	∴ The section is doubly reinforced , $x_{u.max} = 0.48 \times 340 = 163.2\ mm$	
	$M_{u2} = 102.6 - 73.4 = 29.2\ kN.m$	
	Tension steel :	
	$A_{st1} = \frac{73.4 \times 10^6}{0.87 \times 415 \times (340 - 0.42 \times 163.2)} = 749\ mm^2$	
	$A_{st2} = \frac{29.2 \times 10^6}{0.87 \times 415 \times (340 - 40)} = 270\ mm^2$	
	∴ Total tension steel = 749 + 270 = 1019 mm^2	
	Compression steel :	
	For $d_c/d = 0.12$, $A_{sc} = 1.065 \times 270 = 288\ mm^2$	Table 4.2.3
	Note : *Depending on the diameter of the bar selected it is possible that two rows may be required for top tension bars, which will increase the effective cover. The effect will be marginal decrease in area of tension steel and small increase in area of compression steel.*	
	(i) At intermediate support (Left side) - For section 230mm × 700 mm	
	$M_{ur.max} = 2.76 \times 230 \times 630^2 \times 10^{-6} = 251.9\ kN.m > M_u\ (= 102.6\ kN.m)$	
	∴ The section is singly reinforced	
	$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 102.6 \times 10^6}{20 \times 230 \times 630^2}}\right] \times 230 \times 630$	
	$= 485\ mm^2$	

Beam B17 - B18 continued....

Step	Design Calculations	Reference
	(c) *At middle of Span BC* : Eventhough there is very less sagging moment for worst combination of loading, considerable negative moment persists in the span. Hence, main steel will be designed for maximum negative moment for loading *Case - II*	
	$M_{u.max} = -23.12 \times 2.5/2 - 12.7 \times 2.5^2/8 = -38.8\ kN.m < M_{ur.max} = 73.4\ kN.m$	Sect.7.3.2(7)
	$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 38.8 \times 10^6}{20 \times 230 \times 340^2}}\right] \times 230 \times 340 = 348\ mm^2$	
	(d) *Detailing* :	

Details	A	Span AB Mid-span	Left B	Right	Span BC Mid-span	C
Required A_{st} in mm^2						
- at Top	-	-	485	1019	348	-
- at Bottom	-	1495	-	288	-	-
Provide						
Top - No-Diam.	2#12	2-#12	1+20* +2-#12	1-#20* +1#20+3-#12	3-#12	3-#12
Bottom -No-Diam.	4-#20 +3#12	1-#20* +3-#20+3#12	3-#12	1-#20* +2-#12	2-#12	2-#12
Provided A_{st} in mm^2						
- at Top	226	226	540	967	339	339
- at Bottom	1595	1595	339	540	226	226

Step	Design Calculations	Reference
	* Bars to be discontinued as per curtailment rules	*Cl. 26.2.3
8.	*Design for Shear* :	
	(a) *Left Support* (beam 230*mm* × 700*mm*) , $V_{u.max}$ = 140.25 *kN* for *Case - II.*	
	$A_{stl} = 1595\ mm^2$, $p_t = 100 \times 1595/(230 \times 630) = 1.1\%$	
	$\tau_{uc} = 0.64\ N/mm^2$ by linear interpolation.	Table 4.4.1
	$V_{uc} = 0.64 \times 230 \times 630/1000 = 92.74\ kN$	
	$V_{usv.min} = 0.4 \times 230 \times 630/1000 = 57.96\ kN$	
	$V_{ur.min} = 92.74 + 57.96 = 150.7\ kN. > V_{u.max}\ (=140.25\ kN)$	
	∴ Minimum stirrups are sufficient	
	Assuming #6*mm* - 2 legged stirrups of Fe 250 grade	
	Spacing, $s = 0.87 \times 250/(0.4 \times 230)$ = say 130*mm*	
	∴ Provide φ6 *mm* 2-legged stirrups at 130 *mm* c/c	

Beam B17 - B18 continued....

Step	Design Calculations	Reference
	(b) Intermediate support - Left side (Beam : 230 *mm* x 700 *mm*)	
	$V_{u.max}$ = 160.26 *kN*. A_{st} = 540 mm^2	
	p_t = 100 x 540 / (230 x 630) = 0.37%	Table 4.4.1
	τ_{uc} = 0.42 N/mm^2	
	V_{uc} = 0.42 x 230 x 630/100 = 60.86 *kN*	
	$V_{usv.min}$ = 57.96 *kN*	
	$V_{ur.min}$ = 60.86 + 57.96 = 118.82 *kN* < $V_{u.max}$ (= 160.26 *kN*)	
	Since this is less than $V_{u.max}$, V_{uD} will be calculated	
	V_{uD} = 160.26 – 30 x (0.115 + 0.63) = 137.91 *kN* > $V_{ur.min}$	
	∴ Design stirrups are required	
	V_{us} = $V_{uD} - V_{uc}$ = 137.91 – 60.86 = 77.05 *kN*	
	Using ϕ 6 *mm* Fe250 grade stirrups ,	
	Spacing *s* = 0.87 x 250 x 56.5 x 630 / (77.05 x 1000) = 100 *mm*	
	L_{sl} = $(V_{u.max} - V_{ur.min}) / w_u$ = (160.26 – 118.82)/30 = 1.38 *m*	
	∴ Provide ϕ 6 *mm* 2-legged stirrups at 100 *mm* c/c from support to 1.38 *m* and rest provide ϕ 6 *mm* at 130 *mm* c/c	
	(c) Intermediate Support - Right side (Beam 230*mm* × 380*mm*)	
	$V_{u.max}$ = 67.54 *kN* for *Case - I.*	
	A_{st1} = 967 mm^2, p_t = 100 × 967/ (230 × 340) = 1.23%, τ_{uc} = 0.666 N/mm^2	Table 4.4.1
	V_{uc} = 0.666 × 230 × 340/1000 = 52.1 *kN*	
	$V_{usv.min}$ = 0.4 × 230 × 340/1000 = 31.28 *kN*	
	$V_{ur.min}$ = 52.1 + 31.28 = 83.38 *kN* > $V_{u.max}$ (= 67.54 *kN*)	
	Minimum stirrups are sufficient. Provide ϕ6 *mm* at 130*mm* c/c	
	(d) Right Support C : Since shear at this end is far less than that at intermediate support where minimum stirrups are sufficient, hence the minimum stirrups will be sufficient throughout the span *BC*.	
	Alternative - II : Continuous Beam Both Spans having same Section 230 mm x 700 mm	
	From practical point of view it may be felt that the same section of 230 × 700mm for both spans would be preferable. The detailed analysis for that is given as under :	
	Distribution factors	
	Beam *AB* : *K* = (2 x 230 x 700^3/12)/10000 = 1315 x 10^3 mm^3 (for flanged section)	
	Beam *BC* : *K* = (230 × 700^3/12)/2500 = 2629× 10^3 mm^3 (for rectangular section)	
	Distribution factors : d_{AB} = 1315/(1315 + 2629) = 0.33 , d_{BC} = 1 – 0.33 = 0.67	
	For Beam BC :	
	Self weight of beam *B18* = 3.28 *kN/m* which is the same as beam *B17*	
	w_{max} = 1.5 (3.28 + 5 + 5 × 1 + 6 × 2.5/6) = 23.7 *kN/m*	
	The fixed end moments will be the same for span *AB* as obtained earlier	
	w_{min} = 3.28 + 5 + 4.25 + 4.5 × 2.5/6 = 14.4 *kN/m*	
	Fixed end moments	
	M_{max} = 23.7 x 2.5^2/8 = 18.5 *kN.m* , M_{min} = 14.4 x 2.5^2/8 = 11.25 *kN.m*	

Beam B17 - B18 continued....

Step	Design Calculations	Reference

Case - I Maximum moment at intermediate support

Joint	A	B		C
Member	AB	BA	BC	CB
Dist. Factors	-	0.33	0.67	-
FEM	0	375.0	– 18.5	0
	-	–117.6	–238.9	-
Final Moments	0	257.4	–257.4	0

Shear : V_{AB} = 30 × 10/2 – 257.4/10 = 124.26 kN ,

V_{BA} = 30 × 10/2 + 257.4/10 = 175.74 kN

V_{BC} = 23.7 × 2.5/2 + 257.4/2.5 = 132.58 kN,

V_{CB} = 23.7 × 2.5/2 – 257.4/25 = –73.33 kN

Case - II Maximum span Moment in AB

Joint	A	B		C
Member	AB	BA	BC	CB
Dist. Factors	-	0.33	0.67	-
FEM		375	–11.25	
	-	–120.0	–243.75	-
Final Moment		255.0	–255.0	

V_{AB} = 30 × 10/2 – 255/10 = 124.5 kN,

V_{BA} = 30 × 10/2 + 255/10 = 175.5 kN

V_{BC} = 14.4 × 2.5/2 + 255/2.5 = 120.0 kN,

V_{CB} = 14.4 × 2.5/2 – 255/2.5 = –84.0 kN

x_{max} = 124.5/30 = 4.15 kN.m , L_o = 2 x 4.15 = 8.30 m

M_{max} = 124.5× 4.15/2 = 258.3 kN.m

Mid-span moment in BC = –84 × 2.5/2 – 14.4 × $1.25^2/2$ = –116.25 kN.m

Case - III Maximum span moment in BC

Joint	A	B		C
Member	AB	BA	BC	CB
Dist. Factors	-	0.33	0.67	-
FEM		216.2	–18.5	
	-	–65.2	–132.5	-
Final Moment		–151.0	–151.0	

V_{BC} = 23.7 x 2.5/2 + 151.0 / 2.5 = 90.0 kN

V_{CB} = 23.7 x 2.5/2 – 151.0 / 2.5 = –30.8 kN

Mid - span moment in BC = –30.8 x 2.5/2 – 23.7 x $1.25^2/2$ = – 57.0 kN.m

Beam B17 - B18 continued....

Step	Design Calculations	Reference

Results :

Details	A	Mid-span	B	Mid-span	C
(a) Maximum Moments in kN.m Governing Case	0	258.3 II	−257.4 I	−116.25 II	
(b) Shear in kN Govering Case	124.5 II		175.74 I | 132.58 I	I	−84.0 II

$$b_f = 8300/12 + 3 \times 130 + 230 = 1312\ mm$$

Main Steel : Span *AB*

Mid-span: $A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 258.3 \times 10^6}{20 \times 1312 \times 630^2}}\right] \times 1312 \times 630$

$= 1421\ mm^2$

Support : $M_{ur.max} = 251.9\ kN.m \cong 257.4\ kN.m$

∴ Section is singly reinforced

$$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 257.4 \times 10^6}{20 \times 230 \times 630^2}}\right] \times 230 \times 630$$

$= 1422\ mm^2$

Span BC :

Support : Since the section is uniform $A_{st} = 1422\ mm^2$ as obtained above.

Mid - Span : $M_u = -116.25\ kN.m > M_{ur.max}\ (=256.9\ kN.m)$

∴ Section is singly reinforced

$$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 116.25 \times 10^6}{20 \times 230 \times 630^2}}\right] \times 230 \times 630$$

$= 556\ mm^2$ to be provided at top

Alternative - III : Assuming both beams simply supported at ends

Span AB : Beam 230 *mm* x 700 *mm* , $b_f = 10000/12 + 3 \times 130 + 230 = 1453\ mm$

Maximum $B.M = w_u L^2/8 = 30 \times 10^2/8 = 375\ kN.m$

$$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 375 \times 10^6}{20 \times 1453 \times 630^2}}\right] \times 1453 \times 630$$

$= 1716\ mm^2$

Span BC : Beam 230 x 380 *mm* , $w_u = 21.2\ kN/m$

$M_u = 21.2 \times 2.5^2/8 = 16.6\ kN.m$

$$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 16.6 \times 10^6}{20 \times 230 \times 340^2}}\right] \times 230 \times 340$$

$= 141\ mm^2 < A_{st.min}\ (=160\ mm^2)$

∴ $A_{st} = 160\ mm^2$

Provide 2-#12 , Area provided = 223 mm^2

Beam B17 - B18 continued....

Step	Design Calculations	Reference
	Comparision of Results	
	(a) Bending Moments :	

	A	Mid-span	B	Mid-span	C
Alternative - I Stepped Sect.	0	327.83	−102.6	−38.8	
Alternative - II Uniform Sect.	0	258.3	−257.4	−116.25	0
Alternative - III Stepped Sect. *S.S.*	0	375.00	0	16.6	0

(b) Shear Force :

	A	B		C
Alternative - I	140.25	160.26	67.54	−23.12
Alternative - II	124.5	175.74	132.58	−84.0
Alternative - III	150.00	150.00	26.5	26.5

COMPARISION OF AREA OF STEEL :

	Span AB			*Span BC*		
	A	*Mid-span*	B		*Mid-span*	C
Alternative - I At Top	-	-	485	1019	348	0
At Bottom	-	1495	-	288	-	0
Alternative - II At Top	-	-	1422	1422	556	-
At Bottom	-	1421		-	-	
Alternative - III At Top	-	-	* 572	-	-	-
At Bottom	-	1716	-	-	160	-

Concluding Remarks : *It will be seen from the comparision of results for stepped continuous beam (Alt-1) that beam BC is subjected to negative moments through out the span. But when uniform section is used there is too much increase in negative moment both all support and mid-span of BC to the extent of 2.5 to 3 times the moments for Alttenative - I. Also very large negative reaction gets developed at the end of BC. Even the negative reaction developed is about 47% more than one with stepped section (Alt-1). Designing both beams AB and BC using stepped section simply supported at ends (Alt-3) gives economical situation. However,as the connection is monolithic negative moment will develop at the top of continuous support due to partial fixity resulting in development of tension cracks at support. * Therefore, in addition to 2-#12 mm anchor bars the additional bar of 1#20 mm should be provided extending it in span BC and BA for a length of L_d (i.e. about 1/3 of mid-span steel should be provided to arrest the cracking).*

9. *Load on column :*

Load on Col. *C1* = 26.5 *kN* ,

Load on Col.*C7*, Left = 150 *kN*,

on Right = 26.5 *kN* , Load on Col.*C15* = 150 *kN*

Miscellaneous Beam - B5 : One end Simply Supported and the Other Contineous

Step	Design Calculations	Reference

1. *Beam mark* : *B5*
2. *End Condition*: Continuous at one end and simply supported at the other end.
   - Uniformly distributed load as well as Point Load.
3. *Span* : $L = 4.5$ *m*. = 4500 *mm*.
4. *Section* : Assumed 230*m* × 380 *mm* , $d' = 40$ *mm* , $d = 340$ *mm*.
5. *Loads* :

I - UD Load : It is different over different lengths.

(a) Upto Beam *B27* for a length $= 4.5 - 3.145$ *(see Fig.7.1.2)*
$= 1.355$ *m* say 1.36 *m* from left support

w_{u1} = Self + parapet wall + slab *S2* on 1.36 *m* length
$= 1.5\ (1.6 + 5 + 6 \times 2.5/2) = 21.15$ *kN/m*

(b) From Beam *B27* to right end for a length 3.14 *metres* (= 4.5 − 1.36)

w_2 = self + wall of staircase 3.2 *metres* high above terrace level
+ trapezoidal load from two way cap slab *S5* having $L_y/L_x = 1.3$ — *Sect. 7.2.5*

$$w_{u2} = 1.5\ (1.6 + 5 \times 3.2 + 0.5 \times 5 \times 2.5 \times [1 - 1/(3 \times 1.3^2)]$$
$$= 33.92\ kN/m$$

II - Point Load = End shear from Beam *B27* $W_u = 55$ *kN*
acting at a distance 1.3 *m* from left support — *B27 Step.11*

6. *Design Moments* :

Since the beam is subjected to loads of different intensities over different lengths, the fixed end moments will be obtained.
In general for a beam *AB* partially loaded by a uniformly distributed load of intensity *w* the fixed end moments are given by :

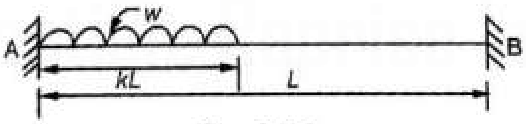

Fig. 7.3.2

$$M_{FAB} = wk^2\ (3k^2 - 8k + 6) \times L^2/12 \quad \text{and} \quad M_{FBA} = wk^3\ (4 - 3k)\ L^2/12$$

Fixed End Moments :

L = 4.5 *metres*;

$w_{u1} = 21.15$ *kN/m*; $w_{u2} = 33.92$ *kN/m* ; W_u = 55 *kN*.

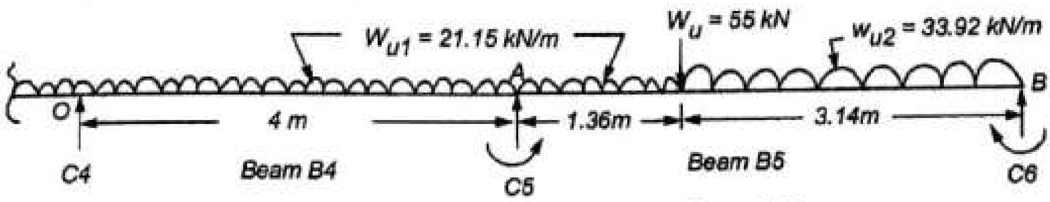

Fig 7.3.3 Loading on Beam B5

$k_1 = 1.36/4.5 = 0.3$, $k_2 = 3.14 / 4.5 = 0.7$, $a = 1.36$ *m* , $b = 3.14$*m*

Beam - B5 continued....

Step	Design Calculations	Reference

M_{FAB} = $w_{u1}\, k_1^2\,(3k_1^2 - 8k_1+6) \times L^2/12 + w_{u2}\, k_2^3\,(4 - 3k_2)L^2/12 + W_u\, ab^2/L^2$

$= 21.15 \times 0.3^2\,(3 \times 0.3^2 - 8 \times 0.3 + 6) \times 4.5^2/12$

$+ 33.92 \times 0.7^3\,(4 - 3 \times 0.7) \times 4.5^2/12 + 55 \times 1.36 \times 3.14^2/4.5^2$

$= 12.43 + 37.3 + 36.42$

$= 86.15$ kN.m

M_{FBA} $= w_{u2}\, k_2^2\,(3\,k_2^2 - 8k_2 + 6)\, L^2/12 + w_{u1}\, k_1^3\,(4-3\,k_1)\, L^2/12 + W_u\, a^2b/L^2$

$= 33.92 \times 0.7^2\,(3 \times 0.7^2 - 8 \times 0.7 + 6) \times 4.5^2/12$

$+ 21.15 \times 0.3^3\,(4 - 3 \times 0.3) \times 4.5^2/12 + 55 \times 1.36^2 \times 3.14/4.5^2$

$= 52.45 + 2.99 + 15.77$

$= 71.21$ kN.m.

Now the beam B5 is continuous towards its left end (i.e. over column C5) along with beam B4 which carries UDL of intensity 21.15 kN/m over a span AO = 4m. The fixed end moment are :

M_{FAO} $= 21.15 \times 4^2/12 = 28.20$ kN.m

The fixed end moments at joint over Col.C5 will be distributed in proportion of beam stiffness.

Distribution factors :

Joint	Member	RSF	SUM	D.F.
A	AO	4EI/4 = EI	1.67EI	0.6
	AB	3EI/4.5 = 0.67EI		0.4

Moment Distribution :

Joint	A			B
Member		AO	AB	BA
D.F	-	0.6	0.4	-
FEM	0	28.20	−86.15	71.21
			−35.60	−71.21
Dist.		56.14	37.41	-
Final Moment		84.34	−84.34	0

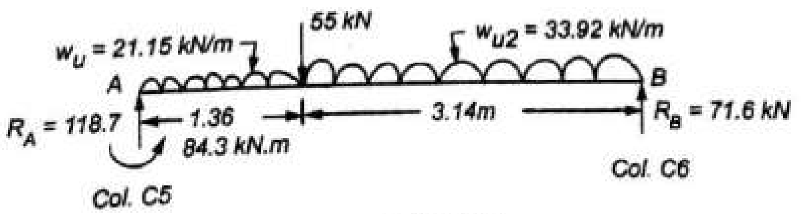

Fig 7.3.4

Beam - B5 continued....

Step	Design Calculations	Reference

$V_{u.BA} = [33.92 \times 3.14 \times (4.5-3.14/2) + 55 \times 1.36 + 21.15 \times 1.36^2/2] - 84.3]4.5$

$= 71.6\ kN$

$V_{u.AB} = (21.15 \times 1.36 + 33.92 \times 3.14 + 55) - 71.6 = 118.7 kN$

Assuming point of maximum span moment to lie between the point load and the right support.

$x_{max} = V_{u.BA} / w_u = 71.6/33.92 = 2.11m$ from $B < 3.14m$ ∴ o.k.

$M_{u.max} = 71.6 \times 2.11/2 - 0 = 75.5\ kN.m$

Main Steel :

(a) *At mid-span :*

$M_u = 75.5\ kN.m \cong M_{ur.max}$

∴ Section is singly using reinforced

$$\text{Required } A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 75.5 \times 10^6}{20 \times 230 \times 340^2}}\right] \times 230 \times 340$$ *Eq. 4.1.6a*

$= 774\ mm^2$

(b) At Support

$M_u = 84.34\ kN.m > M_{ur.max}$

∴ Section is doubly reinforced , $x_{max} = 0.48 \times 340 = 163.2\ mm$

$M_{u2} = 84.34 - 73.4 = 10.94\ kN.m$

$$A_{st1} = \frac{73.4 \times 10^6}{0.87 \times 415 \times (340 - 0.42 \times 163.2)} = 749\ mm^2$$ *Eq. 4.2.3a*

$$A_{st2} = \frac{10.94 \times 10^6}{0.87 \times 415 \times (340 - 40)} = 101\ mm^2$$ *Eq. 4.2.3b*

Total area of tension steel $= A_{st1} + A_{st2} = 749 + 101 = 850\ mm^2$

$d_c/d = 40/340 = 0.12$

$A_{sc} = 1.065 \times 101 = 108\ mm^2$ *Table 4.2.3*

Detailing of Reinforcement :

Details	*Continuous end*	*Mid-span*	*Simple Support*
Required A_{st}			
- at Top	850	-	-
- at Bottom	108	774	-
N-# provided			
- Top	2#12+2#20*(bent)	2#12	2#12
- Bottom	2#12	2#12+2#20(bent)	2#12
Area provided			
- Top	854	226	226
- Bottom	226	854	226

Beam - B5 continued....

Step	Design Calculations	Reference
	(a) At continuous end :	
	$V_{u.max}$ = 118.7 kN $V_{uD} = 118.7 - 21.15 \times (0.115 + 0.34) = 109.1$ kN	
	$p_t\% = 100 \times 854/(230 \times 340) = 1.09\%$	
	$\tau_{uc} = 0.638\ N/mm^2$, $V_{uc} = 0.638 \times 230 \times 340/1000 = 50$ kN	Table 4.4.1
	$V_{usv.min} = 0.4 \times 230 \times 340/1000 = 31.28$ kN	Eq. 4.4.8
	$V_{ur.min} = 50.0 + 31.28 = 81.28\ kN < V_{uD}$	
	∴ Shear reinforcement is required.	Eq. 4.4.9
	$V_{usv} = 109.1 - 50 = 59.1$ kN.	
	Using #8mm two-legged stirrups	
	$s = 0.87 \times 415 \times 100 \times 340/(59.1 \times 1000)$ = say 200 mm c/c	Eq. 4.4.5
	$L_{s1} = (V_{u.max} - V_{ur.min}) / w_u = (118.7 - 81.28)/21.15 = 1.77m > 1.36m$	Eq. 6.3.2a
	$\therefore L_{s1} = 1.36\ m$ only	
	At simply supported End :	
	$V_{u.max}$ = 71.6 kN ,	
	$p_t\% = 226 \times 100/(230 \times 340) = 0.29\%$	
	$\therefore \tau_{uc} = 0.38\ N/mm^2$, $V_{uc} = 0.38 \times 230 \times 340/1000 = 29.7$ kN	Table 4.4.1
	$V_{ur.min} = 29.7 + 31.28 = 60.99$ kN	
	$V_{uD} = 71.6 - 33.92 \times (0.115 + 0.34) = 56.1\ kN < V_{ur.min}$ (= 60.99 kN)	
	∴ minimum stirrups are sufficeint.	
	Provide ϕ6mm at 130 mm c/c	
9.	Check for Deflection.	
	At mid-span Required $A_{st} = 774\ mm^2$, Provided $A_{st} = 854\ mm^2$	
	$f_s = 0.58 \times 415 \times 774/854 = 218\ N/mm^2$	
	Provided $p_t = 100 \times 854/(230 \times 340) = 1.09\%$	
	Modification factor $\alpha_1 = 1.05$	Fig.4.7.1
	Required $d = 4500/(26 \times 1.05) = 165\ mm < 340mm$ ∴ safe	
10.	Load on column : Load on Col. C5 on right = 118.7 kN	
	Load on Col. C6 on left = 71.6 kN	

Miscellaneous Beam - B10 One end simply supported and other Continuous

Step	Design Calculations	Reference
1.	Beam Mark : B10	
2.	End Condition: End condition No.2 but with Point load and different UD loads over different parts of beam. The beam is exactly similar to beam B5 except that there is also a slab S4 on the other side and therefore, there is additional load from S4 and the beam acts as a flanged (L) beam.	
3.	Span : 4.5 m = 4500mm.	
4.	Section : Assumed 230mm × 380mm, $d' = d_c = 40mm$, $d = 340m$, $D_f = 130mm$.	

Beam - B10 continued....

Step	Design Calculations	Reference
5.	*Loads* : For a length of 1.355 say 1.36*m*, from left support load w_{u1} is the same as that for Beam *B5* except that there is no parapet load on this beam.	
	$w_{u1} = 1.5\,(1.6 + 6 \times 2.5/2) = 13.65$ *kN/m*.	
	Load $w_{u2} = 33.92$ *kN/m* same as that for beam *B5*	B27 Step-11
	Point Load $w_u = 74$ *kN*	
	Additional load from two-way slab *S4* is triangular.	
	Equivalent *UD* load $w_{ub3} = w_u L_x/3$.	Sect.7.2.6
	$w_{ub3} = 9.75 \times 4.5/3 = 14.6$ *kN/m*.	
	Since the load from two-way slab is much less than the loads the equivalent *UD* load for shear is assumed to be equal to *UD* load for bending moment.	
	For Bending : Point Load of 74 *kN*, which end reaction of *B27*	
	Total *UD* load on left 1.36 *m* = $w_{us} = 13.65 + 14.6 = 28.25$ *kN/m* ,	
	Total *UD* load on right 3.14 *m* = $w_{u2} = 33.92 + 14.6 = 48.52$ *kN/m*	
6.	*Design Moments* :	

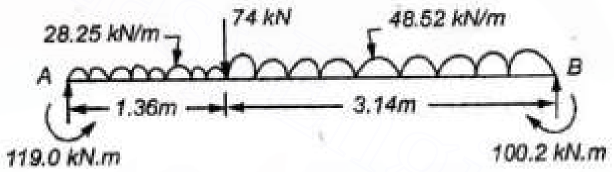

Fig. 7.3.5 Loading on beam B10

$k_1 = 0.3,\ k_2 = 0.7,\ a = 1.36\ m$ and $b = 3.14\ m$ as obtained in Beam *B5*

Fixed End Moments :

$$M_{FAB} = 28.25 \times 0.3^2 \times (3 \times 0.3^2 - 8 \times 0.3 + 6) \times 4.5^2/12 + 48.52 \times 0.7^3\,(4 - 3 \times 0.7) \times 4.5^2/12 + 74 \times 1.36 \times 3.14^2/4.5^2 = 119.0\,kN.m$$

$$M_{FBA} = 48.52 \times 0.7^2\,(3 \times 0.7^2 - 8 \times 0.7 + 6) \times 4.5^2/12 + 28.25 \times 0.3^3\,(4 - 3 \times 0.3) \times 4.5^2/12 + 74 \times 1.36^2 \times 3.14/4.5^2 = 100.2\ kN.m$$

To the left of *A*, beam is *B9* of span 4.0 *metres* is continuous at the other end and carrying a *UD* load $w_u = 20.15$ *kN/m*.

$$M_{FAO} = 20.15 \times 4^2/12 = 26.9\ kN.m$$

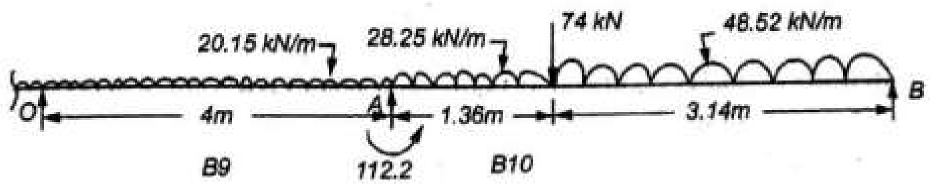

Fig. 7.3.6

Distribution factor will be the same as obtained for Beam *B5* viz. 0.6 and 0.4 — Moment

Beam - B10 continued....

Step	Design Calculations	Reference
	Moment Distribution	

Joint	A			B
Member		AO	AB	BA
D.F-	-	0.6	0.4	-
FEM	0	26.9	−119.0	100.2
			−50.1	−100.2
		85.3	56.9	0
Final Moment	0	112.2	−112.2	0

V_{uAB} $= [28.25 \times 1.36\,(1.36/2 + 3.14) + 48.52 \times 3.14^2/2 + 74 \times 3.14 + 112.2]\,/4.5$
$= 162.3\ kN$

V_{uBA} $= (74 + 28.25 \times 1.36 + 48.52 \times 3.14 - 162.3 = 102.5\ kN$

x_{max} $= 102.5/48.52 = 2.11m < 3.14m$ $\therefore$ o.k.

$M_{u.max} = 102.5 \times 2.11 - 48.52 \times 2.11^2/2 = 108.3\ kN.m$

7. *Main Steel :*

(a) *At Continuous End :*

Allowing 20%redistribution of moment $M_{uAB} = 0.8 \times 112.2 = 89.8\ kN.m.$

$k_{u.limit} = 0.6 - 0.2 = 0.4$, $x_{u.limit} = 0.4 \times 340 = 136mm$

$M_{ur.limit} = 0.36 \times 20 \times 0.4 \times (1 - 0.42 \times 0.4) \times 230 \times 340^2 \times 10^{-6}$
$= 63.7\ kN.m$

The section will be required to be Doubly reinforced.

$M_{u2} = 89.8 - 63.7 = 26.1\ kN.m$

Tension Steel :

$$A_{st1} = \frac{63.7 \times 10^6}{0.87 \times 415 \times (340 - 0.42 \times 136)} = 624\ mm^2$$ Eq. 4.2.3a

$$A_{st2} = \frac{26.1 \times 10^6}{0.87 \times 415 \times (340 - 40)} = 241\ mm^2$$ Eq. 4.2.3b

$$A_{st} = A_{st1} + A_{st2} = 624 + 241 = 865\ mm^2$$

Compression Steel :

For M20 - Fe415, $d'_c/d = 40/340 = 0.12$

$A_{sc} = 1.065 \times 241 = 257\ mm^2$ Table 4.2.3

(b) *At Midspan :*

$M_{u.max} = 108.3\ kN.m$

The beam is designed as L-beam

Assuming $L_o = 0.7L = 0.7 \times 4500 = 3150mm$

The beam acts as L-beam since there is slab $S4$ only of thickness 130mm on one side.

$b_f = 3150/12 + 3 \times 130 + 230 = 882\ mm.$

$(M_{ur1})_{x_u = D_f} = 0.36 \times 20 \times 882 \times 130 \times (340 - 0.42 \times 130) \times 10^{-6}$ Eq.4.3.8
$= 235\ kN.m > M_{u.max}\ \therefore x_u < D_f$

IV(d) Beam - B10 continued....

Step	Design Calculations	Reference
	Required $A_{st} = \frac{0.5 \times 20}{415}\left[1-\sqrt{1-\frac{4.6 \times 108.3 \times 10^6}{20 \times 882 \times 340^2}}\right] \times 882 \times 340 = 945 mm^2$	
	(c) Detailing :	

Required A_{st}	Continuous End	Midspan	Simple Support
- at top	865	---	---
- at bottom	257	945	---
Provide at top - No. Di(mm)	2#12 +2-#20 bent* from mid-span	2-#12	2#12
at bottom - No.Di(mm)	3-#12	3-#12+2-#20(bent)*	3#12+2#20
Provided area - at top	854	226	226
- at bottom	339	967	339

8. **Design for Shear :**

(a) At Continuous End :

$V_{u.max} = 162.3\ kN$

$A_{st1} = 854\ mm^2$. $p_t = 100 \times 854/(230 \times 340) = 1.09\%$

$\tau_{uc} = 0.638\ N/mm^2$ by linear interpolation. — *Table 4.4.1*

$V_{uc} = 0.638 \times 230 \times 340/1000 = 50.0\ kN$

$V_{usv.min} = 0.4 \times 230 \times 340/1000 = 31.28\ kN.$

$V_{ur.min} = 50.0 + 31.28 = 81.28\ kN < V_{u.max} (= 162.3\ kN)$

Design shear will be calculated at critical section.

$V_{uD} = V_{u.max} - w_u(b_s/2+d) = 162.3 - 28.25 \times (0.115 + 0.34)$

$= 149.4\ kN > V_{ur.min}$

∴ Stirrups will be required to be designed.

$V_{us} = V_{uD} - V_{uc} = 149.4 - 50.00 = 99.4\ kN.$

Assuming # 8mm - 2 legged Fe415 grade stirrups ($A_{sv} = 100\ mm^2$)

Spacing $s = 0.87 \times 415 \times 100 \times 340/(99.4 \times 1000) = 120\ mm.$

Length of Shear Zone = $L_{s1} = (V_{u.max} - V_{ur.min})/w_u$

$= (162.3 - 81.28)/28.25 > 2.87\ m$

Shear to the right of point load $= 162.3 - 28.25 \times 1.36 - 74 = 49.9\ kN < V_{ur.min}$

$\therefore L_{s1} = 1.36$ only.

(b) *At Simply Supported End :*

$V_{u.max} = 102.5kN.$

$A_{st1} = 3\text{-}\#12 = 339\ mm^2$, $p_t = 100 \times 339/(230 \times 340) = 0.43\%$

$\tau_{uc} = 0.446N/mm^2$ $\therefore V_{uc} = 0.446 \times 230 \times 340/1000 = 34.88\ kN$ — *Table 4.4.1*

$\therefore V_{ur.min} = 34.88 + 31.28 = 66.16\ kN < V_{u.max}$

Beam - B10 continued....

Step	Design Calculations	Reference
	$V_{uD} = 102.5 - 48.52 \times (.23/2+.340) = 80.4\ kN > V_{ur.min}\ (= 66.16 kN)$ ∴ Stirrups will be required to be designed. $V_{usv} = V_{uD} - V_{uc} = 80.4 - 34.88 = 45.52\ kN$ Using φ6*mm* 2-legged stirrups of Fe250 $s = \frac{0.87 \times 250 \times 57 \times 340}{45.52 \times 1000}$ = say 90 *mm* $L_{s1} = (102.5 - 66.16) / 48.52 = 0.75\ m$ ∴ Provide φ 6 *mm* at 90 *mm* c/c upto 750 *mm* and then φ 6 *mm* at 130 *mm* c/c.	
9.	*Load on Column :* - on right column *C11* = 162.3 *kN* - on left column *C12* = 102.5 *kN*	

Schedule of Beams

Beam No.	Overall Size B x D	N-# Steel at			Stirrups
		Left Top	Mid-span Bottom	Right Left Top	
B1	230x380	2#10	2#12+1#12*	2#10 + 1#12*+1#12**	φ6@130
B2,B3,B4	230x380	2#10+1#12*+1#12**	2#12+1#12*	2#10+1#12*+1#12**	φ 6@130
B5	230x380	2#12+2#20*	2#12 +2#20*	2#12	#8@100(8)Left rest φ6@130
B6	230x380	2#10	2#12+1#12*	2#10+1#12**+1#12/2.7	φ 6@130
B7,B8,B9	230x380	2#10+1#12/2.7	2#12	2#10+1#12/2.7	φ6@130
B13,14,15					
B10		2#12+2#20*	3#12+2#20*	2#12	#8120(14left) φ 6@90(10 Right) rest φ 6@130
B11		2#10	4#16	2#10	φ6 @130
B12		2#10	2#12	2#10+1#12(2.7)	φ6 @ 130
B16		2#10+1#12*+1#16/2.7	3#12+1#12*	2#10	φ6 @130
B17	230x700	Top 2#12	2#12	2#12+1#20*	φ6 @100(15 Right)
		Bottom 4#20+3#12	3#12+3#20+1#20*	3#12	Rest φ6 @ 130
B18	230x380	Top 3#12+1#20+1#20*	3#12	3#12	φ6 @130
		Bottom 1#20+2#12	2#12	2#12	
B19,20,21	230x700	2#12	4#25+2#25(7*m*)	2#12	#8@190(24), φ6@130(24) φ6 @300 (8)
B22	230x380	2#10	3#20	2#10+3#20	#8@250(5) R rest φ 6@130
B23	230x380	2#10+3#20	3#20	2#10	#8@250(5) L rest φ 6@130
B24	230x380	2#12	2#10 +1#16+1#16*	2#12 +1#16+1#16*	φ6@130
B25	230x380	2#12+1#16+1#16*	2#10 +1#16+1#16*	2#12	φ6@130
B26	230x380	2#10+1#12*	2#12+1#12*	2#10+1#12*	φ6 @130
B27	230x380	2#10+1#12*	3#12+1#12*	2#10+1#12*	φ6 @130

Note: 1). * Bar bent from mid-span 2) ** Bar obtained from adjacent spans 3) For reinforcement to the right of B4 see B5 4) For reinforcement to the right of B9 see B10 5) For reinforcement to the right of B15 see B16.

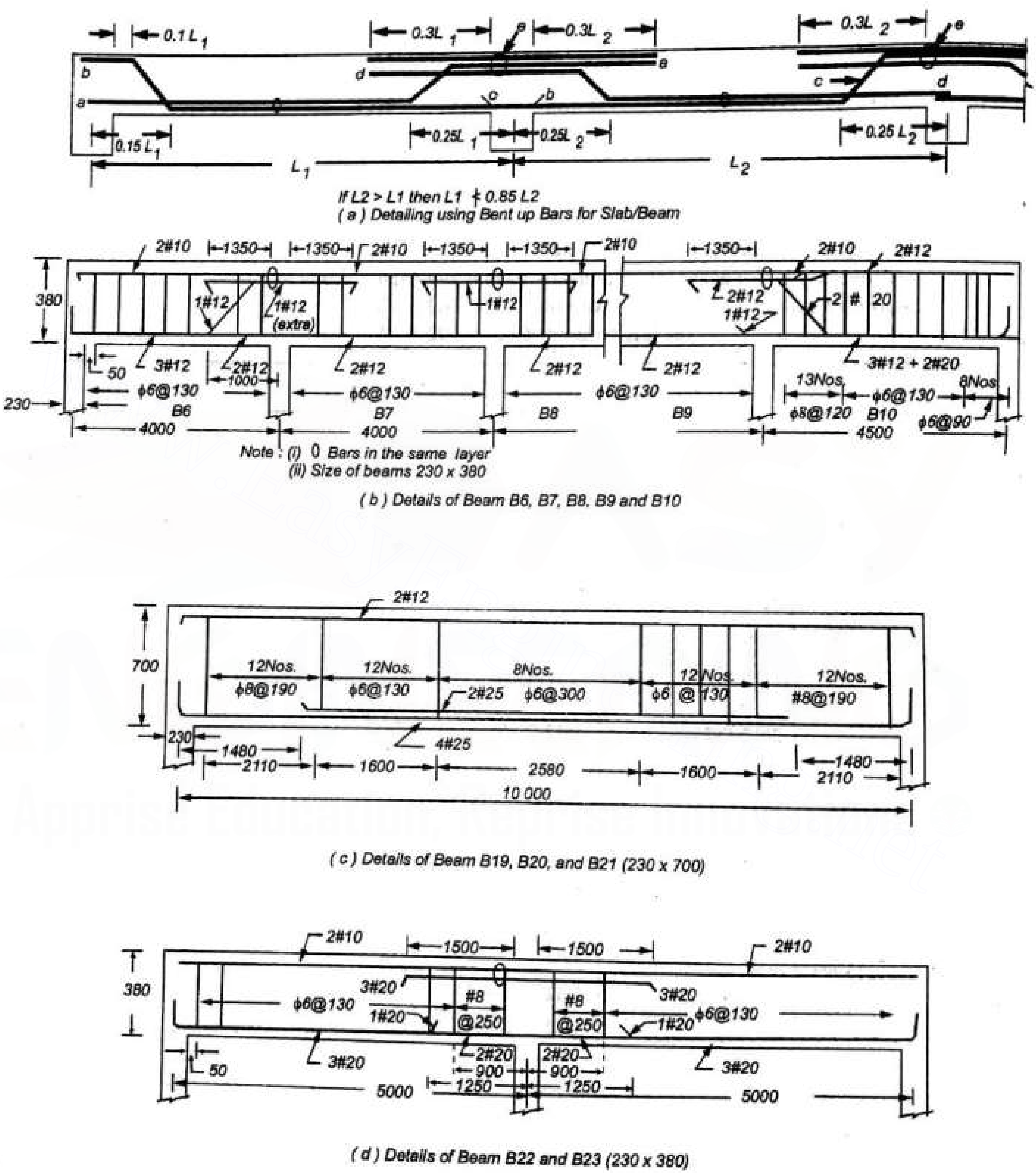

Fig. 7.3.7 Reinforcement Detailing for Beams

7.4 DESIGN OF COLUMNS

7.4.1 Categorization of Columns

Category-I : Axially Loaded Columns

Columns with beams only on opposite sides : *C2, C3, C4.*

Category-II : Columns subjected to Axial Load and Uniaxial Bending

(a) Columns with beams either only in one direction *or* in three directions. Beams in opposite directions are required to be of the same span and load : *C6, C8, C9, C10, C13, C14, C16, C17, C18.*

(b) Columns having beams on opposite sides only but having unequal spans and / *or* loads : *C5.*

Category-III : Columns under Axial Load and Biaxial Bending.

(a) Columns with beams in two adjacent perpendicular directions : *C1, C15, C20.*

(b) Columns with beams on three sides but beams on opposite sides have unequal spans and / *or* loads : *C7, C11, C19.*

(c) Columns with beams on three sides but beams on opposite sides are not at the same level : *C12.*

7.4.2 Assessment of Floor Loads on Columns and Grouping

(a) Load Assessment : With a view to arrive at an approximate column section, it is first necessary to assess the floor loads on columns (excluding self weight and allowances for bending and for slenderness). This can be done by summing up the end shears from beams meeting at the column. The directions in which beam end shears are taken serially as per *Fig. 7.4.1.* are shown in *Table 7.4.1.* The beam end shears are shown in *Fig. 7.4.2.*

∴ The self weight of the column = 1.5 x 25 x 0.23 x 0.23 x 3.84 = 8*kN*

The axial load on the column for design is shown in *Table 7.4.1*

The unsupported length of the column = 3200 + 600 + 720 − 300 (footing depth) − 380 Minimum beam depth = 3840 *mm*

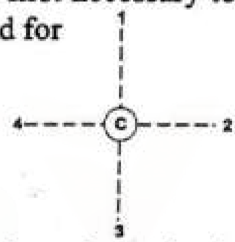

Fig. 7.4.1

Table 7.4.1 : Loads on Columns

Category	*Column Mark*	*Beam End Shears in kN from direction* 1	2	3	4	*Total Rounded kN*	*Self wt. kN*	*Total axial Load kN*
I	*C2*	-	42.3	-	50.8	93	8	101
	C3	-	42.3	-	42.3	85	8	93
	C4	-	42.3	-	42.3	85	8	93
II	*C5*	-	118.7	-	42.3	161	8	167
	C6	-	-	52.4	71.6	124	8	132
	C8	-	27.3	250.0	32.8	310	8	318
	C9	-	27.3	250.0	27.3	305	8	313
	C10	-	27.3	250.0	27.3	305	8	313
	C13	114.9	54.7	114.9	-	285	8	293
	C14	62.4	-	62.4	54.7	180	8	188
	C16	250.0	19.8	-	23.8	294	8	302
	C17	250.0	19.8	-	19.8	290	8	298
	C18	250.0	19.8	-	19.8	290	8	298
III	*C1*	-	38.1	26.5	-	65	8	73
	C7	26.5	24.6	150.0	-	201	8	209
	C11	-	162.3	86.2	27.3	276	8	284
	C12	52.4	-	46.8	102.5	202	8	210
	C15	150.0	17.8	-	-	168	8	176
	C19	86.2	56.4	-	19.8	163	8	171
	C20	46.8	-	-	42.3	90	8	98

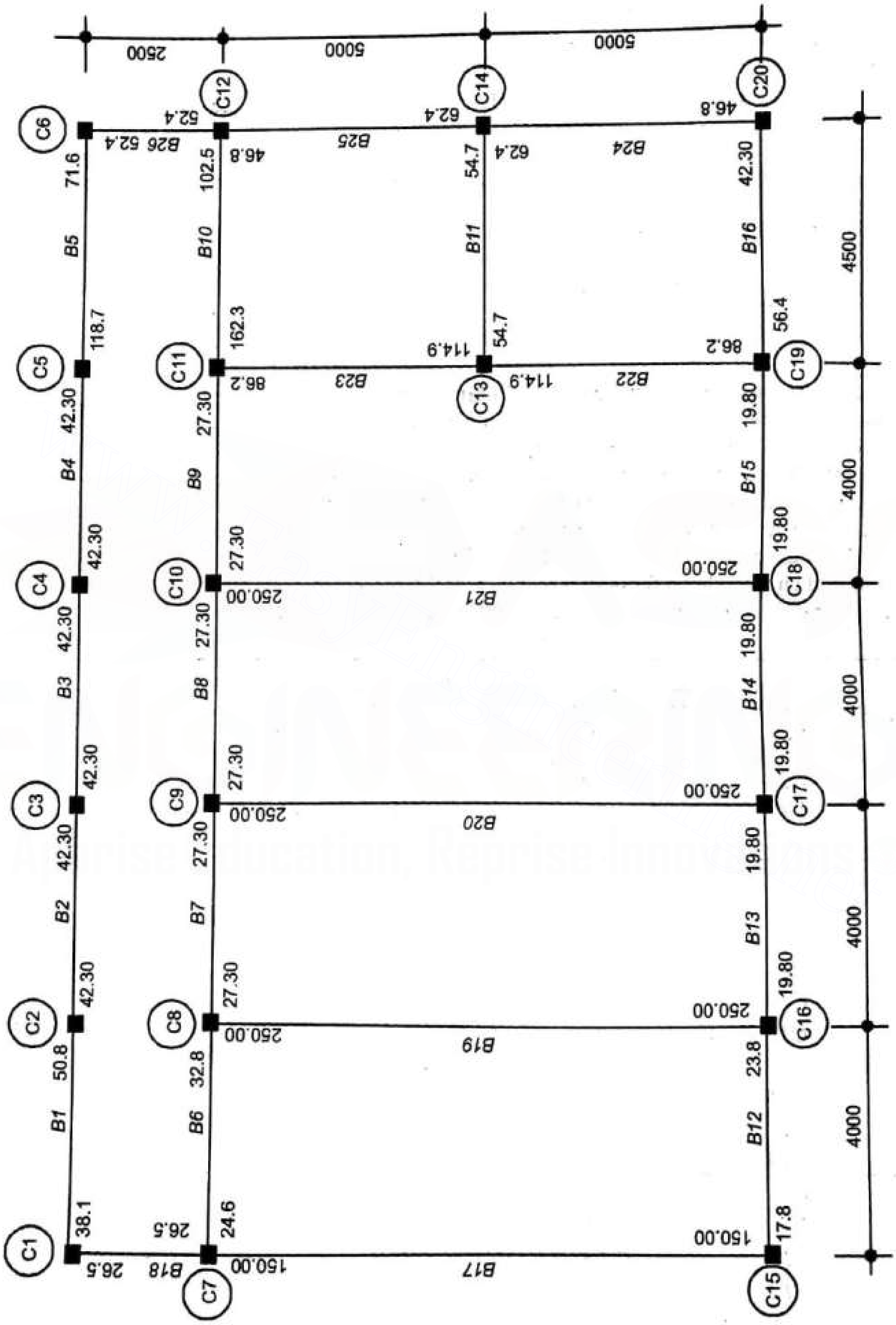

Fig. 7.4.2 Beam End Shears on Columns

(b) Grouping of Columns

The columns under each category having loads within a variation of 10% to 20% have been grouped together *(Table 7.4.2)* to reduce calculations. There is single design calculation for each group for maximum load.

Table 7.4.2 Grouping of Columns

Category	Group	Column Marks	$P_{u.max}$ kN	$P_{u.min}$ kN
I	-	C2, (C3, C4)	101	93
II	(a)	C8,(C9, C10, C16, C17, C18)	318	298
	(b)	C13	293	
	(c)	C14, (C5, C6)	188	132
III	(a)	C11, (C7)	284	
	(b)	C12, C7, (C15, C19,C20, C1)	210	73*

*Note : * Though variation between maximum and minimum load is large, the maximum load itself is very small in relation to the load carrying capacity of 400kN for the smallest section 230 x 230mm with 4-#12 proposed for this design. Therefore, all columns below 50% of this load carrying capacity have been grouped together since the smallest section would be adequate for all, keeping margin of 50% for allowances for bending (fixity) and for slenderness.*

7.4.3 Determination of Effective Length and Slenderness

Since even the heaviest load on the column being 310 *kN* which is less than the axial load carrying capacity of minimum size of 230*mm* x 230*mm* with minimum steel (4-#12) for such public buildings, it is proposed to use this single section for all columns. The change in reinforcement , if required, will cater for the requirements of bending moment and slenderness.

It is given that the ground is level and depth of foundation is constant. Therefore, the lengths of all columns are equal, and hence single calculation for effective length and slenderness is sufficient for all columns. When the depths of foundation for different columns are different, again each group of columns under each category is required to be subdivided into subgroup based on their lengths.

Calculations given below are common for all columns :

Floor to floor height	=	3200 *mm*.
Height of plinth above *G.L.*	=	600 *mm*.
Depth of foundation below *G.L.*	=	720 *mm*.
Assuming depth of footing	=	300 *mm*
Total height of column *above top of footing*	=	4220 *mm*.
Depth of shallowest beam	=	380 *mm*.
Unsupported length of column L = 4220 – 380	=	3840 *mm*.

Assuming effective length $L_{eff} = L$ since all columns are supported by beams in both the directions and there are longitudinal and transverse external walls. Actual effective length is therefore, likely to be less than L if exact calculations are done as explained in *Sect. 4.8.3 (ii)*

Effective length of column L_{eff}	=	3840 *mm*.	
Assumed Section b x D (in *mm*)	=	230 *mm* x 230 *mm*.	
Slenderness ratio $L_{eff}/b = 3840/230$	=	$16.7 > 12$	
∴ The column is slender			
Allowance for slenderness	=	$(1/C_r - 1) \times 100\%$	*(Eq.7.4.1)*
where, $C_r = 1.25 - L_{eff}/(48b)$	=	$1.25 - 16.7/48 = 0.9$	(7.4.1a)
∴ Allowance for slenderness	=	$(1/0.9 - 1) \times 100 = 10.9\%$	say 11%
Factored self weight of column	=	1.5x25x.23x.23x3.84 = 7.6 *kN* say 8 *kN*.	

7.4.4 Calculation of Equivalent Axial Design Load for Short Column and Design of Reinforcement.

The equivalent axial design loads are calculated by adding to floor loads the allowances for bending due to effect of full/partial fixity between column and beam as explained in *Sect. 5.4.2* and allowance for slenderness as explained in *Sect. 6.4.9(b). For a slender column under biaxial bending, allowance for slenderness shall be taken twice since the moment due to minimum eccentricity occurs in both the directions simultaneously.*

For design of Reinforcement, it will be appropriate to calculate load carrying capacities of column section 230 *mm* x 230 *mm* with different number diameter combinations of bars.
Strength in axial compression is obtained by using following equations :

$$P_u = (P_{uc} + P_{us}) = \lambda\,[0.4 f_{ck} bD + (0.67 f_y - 0.4 f_{ck})\, A_{sc}] \qquad (7.4.2)$$

where, $P_{uc} = \lambda\,(0.4 f_{ck}\, bD)$ where, $\lambda = 0.9$ for $b = 230\ mm$.

$$= 0.9\,(0.4 \times 20 \times 230) \times 230/1000 = 381\ kN.$$

$$P_{us} = \lambda\,(0.67 f_y - 0.4 f_{ck})\, A_{sc} = 0.9\,(0.67 \times 415 - 0.4 \times 20)\, A_{sc}/1000 = 0.243\, A_{sc}$$

Similarly, values of P_{uz} and P_{ub} are required in calculation of additional moment in the case of slender column.

$$P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc} = 0.45 f_{ck} A_g + (0.75 f_y - 0.45 f_{ck}) A_{sc} \qquad (Eq.\ 4.8.6a)$$

$$= [0.45 \times 20 \times 230 \times 230 + (0.75 \times 415 - 0.45 \times 20)\, A_{sc}]\,/\,1000\ kN$$

$$= (476.1 + 0.302\, A_{sc})\ kN$$

Ignoring the contribution of steel to strength in axial compression, P_{ub} can be obtained using following equation assuming diameter of link equal to 6 *mm* :

$$P_{ub} = 0.36 f_{ck}\ b(7/11)\ d = 0.36 \times 20 \times 230 \times (7/11) \times [230 - (40 + \phi/2 + 6)]/1000$$

$$= 1.05\,(184 - \phi/2)$$

$$= 187\ kN \text{ for } \phi = 12\ mm$$

and $P_{ub} = 185\ kN$ for $\phi = 16\ mm$.

The values of P_{uc}, P_u, P_{uz} and P_{ub} are calculated for different *N*-ϕ. Combinations using the above relations, and the same are presented in *Table 7.4.3.*

Table 7.4.3. : Load carrying capacities P_u, P_{uz} and P_{ub} in *kN* of Axially Loaded Columns
Concrete Grade - M20 : Steel Grade Fe 415

Section	P_{uc}	Number Diameter Combinations of Main Reinforcement									Reference
b x *D*	*kN*	No.-Di(*mm*)	4-#12	6-#12	4-#16	8-#12	4#16+2#12	6-#16	4#16+4#12	8-#16	
mm mm		A_{sc} mm^2	452	678	804	904	1030	1206	1256	1608	
		P_{us}	110	165	195	220	250	293	305	391	
230x230	381	P_u(Approx)	491	546	576	601	631	674	686	772	*Eq. 7.4.2*
		P_{uz}	613	681	719	749	787	840	856	962	*Eq. 4.8.6a*
		P_{ub}	187	187	185	187	185	185	185	185	

Note : *Exact value of P_u can be obtained corresponding to minimum eccentricity of 20 mm by using the theory of eccentrically loaded columns[7.4]. The values of P_{ub} given above are also approximate. The exact values should be obtained from theory of eccentrically loaded columns corresponding to $x_u = (7/11)d$. Alternative procedure for calculating P_{ub} has been shown in specimen calculations in Sect. 7.4.5.1 in design of C2.*

The percentage allowance for uniaxial bending and bi-axial bending is taken as 15% and 33% respectively. The allowance for slenderness has been worked out as 11%. Based on this after adding allowance for slenderness are given in *Table 7.4.4.*

Table 7.4.4 Calculation of Equivalent axial load and Design load.

Cat No.	Group	Column Mark	Section b x D	Floor Load	Self Wt	Total Load	Pa.f 15% /33%	Eq. axial Load	Pa.s (11%)	Design Load	Steel N - ϕ
I		C2,C3,C4	230x230	93	8	101	- -	101	12	113	4-#12
II	(a)	C8,C9,C10, C16,C17,C18	230x230	310	8	318	48	366	41	407	4-#12
	(b)	C13	230x230	285	8	293	44	337	37	374	4-#12
	(c)	C14,C5,C6	230x230	180	8	188	29	217	24	241	4-#12
III	(a)	C11	230x230	276	8	284	94	378	42x2	462	4-#12
	(b)	C12,C7,C15, C19,C20,C1	230x230	202	8	210	167	269	30x2	329	4-#12

Notes : (1) All loads are in kN. (2) Allowance for fixity is based on % of floor load while allowance for slenderness is % of Total equivalent axial load of short column.

Design of lateral ties :

Though theoretically, minimum diameter of lateral tie is 1/4 the diameter of main bar *or 5mm* whichever is less, in practice, minimum diameter adopted is 6*mm*. Therefore, provide 6*mm* diameter ties of Grade Fe250 at a spacing equal to *least of the following (See Sect. 5.4.3B)*

(i) Least lateral dimension b = 230 *mm*,
(ii) 16 times diameter of main bar = 16 x 12 = 192*mm* for ϕ = 12*mm*, and
= 16 x 16 = 256*mm* for ϕ = 16 *mm*.
(iii) Minimum pitch = 300 *mm*.

i.e. s = 190 *mm* for main bar of 12 *mm* diameter, and
s = 230 *mm* for main bar of 16 *mm* diameter

7.4.5 Check for Effect of Bending and Slenderness

Now all the columns will be checked for the effect of bending due to full or partial fixity between beam and column according to the procedure detailed in *Sect. 6.4.8*

7.4.5.1 *Category - 1 : Columns under Axial Loads only – Slender Columns*

Design Calculations	*Reference*
Columns C2, (C3, C4) Axial Load P_u = 101 *kN* excluding allowances for bending and slenderness since the actual effects of both are now separately considered theoretically. Assumed Section : 230*mm* x 230*mm* with main steel 4-#12 (A_{sc} = 452 mm^2). Effective Length L_{effy} = 3840*mm* Slenderness Ratio L_{effy}/b = 3840/230 = 16.7 > 12. ∴ The column is slender. Smaller End Moment M_{u1} = 0 at the footing end since the footing is designed as rotation free *i.e.* for axial load only. Larger End Moment M_{u2} = 0 at the top of column because there is no unbalanced moment from beams meeting at the column on opposite faces. ∴ Initial Moment M_i = 0 due to external moments.	

Category - I : Columns under Axial Loads only : Slender Columns continued....

Design Calculations	*Reference*
Minimum Eccentricity $e_{min.y}$ $= L/500 + b/30 = 3840/500 + 230/30 < 20\ mm$	
$\therefore\ e_{miny} = 20mm.$	
Minimum moment $M_{u.miny} = P_u \times e_{min.y} = 101 x 20/1000 = 2.02\ kN.m$	
Revised Initial Moment $M_i = M_{u.miny} = 2.02\ kN.m$	
The above steps calculate the design moment for the column. The steps that follow calculate the additional moments due to the effect of slenderness.	
For this value P_{uz} and P_{ub} are required to be calculated to begin with.	
Ideal Axial Strength $P_{uz} = 613\ kN$	*Table 7.4.3*
Axial load (P_{ub}) corresponding to maximum compressive stain of 0.0035 in concrete and tensile strain of 0.002 in outer most layer of steel is obtained as under :	*Eq. 4.8.13d*
I - From first principles :	
Thus $e_{max} = .0035$ and $e_{min} = -.002$ at tension steel level	
$x_u = [.0035/(.0035 + .002)]d = (7/11)\ d$	
$d' = 40 + \phi/2 + 6 = 40 + 12/2 + 6 = 52mm,\ d = 230 - 52 = 178\ mm.$	
$x_u = (7/11) \times 178 = 113\ mm$	
$P_{ub} = P_{uc} + P_{us}$	
$P_{uc} = 0.36 f_{ck}\ bx_u = 0.36 \times 20 \times 230 \times 113 / 1000 = 187.12\ kN$	
$P_{us} = (f_{s1} - f_{c1}) A_{s1} + f_{s2} \times A_{s2}$, f_{s1} and f_{s2} depend upon strains ε_1 and ε_2.	
$\varepsilon_1 = [(113 - 52)/113] \times 0.0035 = .00190,\ \ \varepsilon_2 = -0.002.$	
$f_{s1} = 323.55\ N/mm^2$ and $f_{s2} = -327.74\ N/mm^2$	*Table 2.14.2*
$f_{c1} = 0.45 \times 20 = 9.0\ N/mm^2$	
$A_{s1} = 452/2 = 226\ mm^2 = A_{s2}.$	
$P_{us} = [\ (323.55 - 9) \times 226 + (-327.74) \times 226]/1000 = -2.98\ kN.$	
$P_{ub} = 187.12 - 2.98 = 184.14\ kN$ *say* $184\ kN$	
II - Using Design Aids :	
$P_{ub} = (k_1 + k_2.p/f_{ck}) f_{ck}\ bD.$	
$f_{ck}\ bD = 20 \times 230 \times 230/1000 = 1058\ kN$	
$p/f_{ck} = 100 \times A_{sc} / (f_{ck}\ bD) = (100 \times 452) / (1058 \times 1000) = 0.043$	
$d'/D = 52/230 = 0.226$	
$k_1 = 0.178$ by extrapolation	
$k_2 = -0.057$ by extrapolation	
$P_{ub} = (0.178 - 0.057 \times 0.043) \times 1058 = 185.7\ kN$	
III - Using Approximate Method	
$P_{ub} = 187\ kN$ as calculated earlier in *Table 7.4.3* above.	
It may be observed that results given by approximate method are slightly on the higher side can be accepted.	
Reduction factor for additional moment due to slenderness	
$k = (P_{uz} - P_u)/(P_{uz} - P_{ub}) >$	*(Eq. 4.8.13c)*
$= (613 - 101)/(613 - 185.7) > 1\ \ \therefore k = 1$	

Category - I : Columns under Axial Loads only : Slender Columns continued....

Design Calculations	*Reference*
Additional Moment $M_{ay} = (P_u\, b/2000)\,(L_{effy}\,/\,b)^2\,.\,k$	*(Eq. 4.8.13b)*
M_{ay} = (101 x 0.23/2000) x 16.7 ^{2} x 1 = 3.24 *kN.m*	
Total Moment on Equivalent Short Column ,	
$M_{uT} = M_i + M_{ay}$ = 2.02 + 3.24 = 5.26 *kN.m*	*(Eq. 4.8.15)*
Check for Combined effect of P_u = 101 *kN* and M_u = 5.26 *kN.m*	
I. By Use of Chart (Appendix G)	
d'/b = 52/230 = 0.226 , $P_u\,/\,(f_{ck}bD)$ = 101/1058 = 0.095	
p/f_{ck} = 100 x 452/(230 x 230 x 20) = 0.043	

d'/D	$P_u/f_{ck}\,bD$	p/f_{ck}	$M_u/f_{ck}\,b^2D$	Chart No.
0.2	0.095	0.043	0.085	4G
0.25	0.095	0.043	0.077	5G
0.226	0.095	0.043	0.081 by interpolation	

Moment of resistance provided by the section

= $(M_{ur})_{provided}$ = 0.081 x 1058 x 0.23 = 19.7 *kN.m* > 5.26 *kN.m*

II. Results from the generalised column design software prepared by the author[7.4]

$(M_{ur})_{provided}$ = 21.8 *kN.m*, $(P_u)_{provided}$ = 178.5 *kN*

The section is safe under combined effect of P_u and M_u.

Note : *No separate calculations are necessary for bending about x - axis since the column is axially loaded and the section is square.*

7.4.5.2 *Category - II : Columns under Axial Load and Uniaxial Bending (Group (a))*

Typical detailed calculations are shown only for column in group (a). For other groups only brief calculations are given.

Design Calculations	*Reference*
(a) **Columns *C8, (C9, C10, C16, C17, C18)* :**	
(i) Bending about (major axis of bending) x - axis :	
Axial Load : P_u = 318 *kN* (actual axial load excluding allowances).	
Section : 230 *mm* x 230 *mm* with 4-#12. A_{sc} = 452 mm^2.	
Effective Length $L_{eff.x}$ = 3840	
Slenderness Ratio $L_{eff.x}/D$ = 3840/230 = 16.7 >12. ∴Column is slender.	
Smaller End Moment M_{u1} = 0 at the base as it is designed as rotation free.	
Larger End Moment = Moment in the column at top due to partial fixity between the beam and the column is calculated as under :	
Column :	
I_c = 230 x 230^3/12 = 233.2 x 10^6 mm^4 , L_c = 3840 *mm*,	
k_c = 0.75 x 233.2 x 10^6/3840 = 45.55 x 10^3 mm^3.	
The stiffness of .75*I/L* is taken since the lower end of the column is rotation free.	

Category - II : Columns under Axial Load and Uniaxial Bending Group (a) continued....

Design Calculations	Reference
Beam :	
Section 230 *mm* x 750*mm*. L = 10000*mm*, beam - flanged, $w_u = 50$ *kN/m*.	
$I_b = 230 \times 700^3/12 = 6574 \times 10^6\ mm^4$ for rectangular section.	
$I_b = 2 \times 6574 \times 10^6\ mm^4$ for flanged section, *(see sect 3.2.5)*.	
$K_b = 2 \times 6574 \times 10^6 / 10000 = 1314.8 \times 10^3\ mm^3$	
Distribution Factor	
$d_{col} = k_c/(k_c + k_b/2) = 45.55/(45.55 + 1314.8/2) = 0.0648$	
Fixed End Moment from beam due to partial fixity $M_F = w_u L^2/24$.	
$M_F = 50 \times 10^2/24 = 208.3$ *kN.m*.	
$M_{col} = d_{col} \times M_F = 0.0648 \times 208.3 = 13.5$ *kN.m*	
$M_{u2} = 13.5$ *kN.m*	
Initial Moment $M_i = 0.6\ M_{u2} + 0.4 M_{u1} = 0.6 \times 13.5 + 0.4 \times 0 = 8.1$ *kN.m*	*Eq.4.8.14a*
Minimum Eccentricity e_{min} = 20 *mm* as calculated earlier.	
Minimum Moment $M_{u.min} = P_u \times e_{min} = 318 \times 20/1000 = 6.36\ kN.m < M_i$.	
Modified Initial Moment M_i = 8.1 *kN.m*	
Ideal Axial Strength P_{uz} = 613 *kN* from *Table 7.4.3*.	
Axial Strength for balanced section P_{ub} = 187 *kN* from *Table 7.4.3*.	
Reduction Factor for additional Moment due to slenderness	
$k = (613 - 318) / (613 - 187) = 0.69$	*Eq. 4.8.13c*
Additional Moment	
$M_{ax} = \frac{P_u D}{2000} \times \left(\frac{L_{effx}}{D}\right)^2 k = \frac{318 \times 0.23}{2000} \times 16.7^2 \times 0.69$	
$= 7.04$ *kN.m*	
Total design Moment for equivalent short column $M_{ux} = M_i + M_{ax}$	
$M_{ux} = 8.1 + 7.04 = 15.14\ kN.m < M_{u2}\ (= 13.5\ kN.m)$	*Eq. 4.8.15*
Moment of Resistance	
$d'/D = 52/230 = 0.226$,	
$f_{ck} bD = 20 \times 230 \times 0.23 = 1058\ kN$, $P_u/f_{ck}\ bD = 318/1058 = 0.3$	
$p/f_{ck} = 100 \times 452/(230 \times 230 \times 20) = 0.043$	
I. By using charts	

d'/D	P_u/f_{ck}	P/f_{ck}	$M_u/f_{ck}\ bD^2$	Chart No.
0.2	0.3	0.043	0.084	4G
0.25	0.3	0.043	0.075	5G
0.226	0.3	0.043	0.079 by interpolation	

$(M_{ur}) = 0.079 \times 1058 \times 0.23 = 19.2\ kN.m > 15.14\ kN.m$ ∴ safe

II. Results from the generalised column design software[7.5] prepared by the author

$(M_{ur})_{provided} = 18.9\ kN.m$, $(P_{ur})_{provided} = 318.02\ kN$

Category - II : Columns under Axial Load and Uniaxial Bending Group (a) continued....

Design Calculations	*Reference*
(ii) Bending @ y - axis :	
In this case, external moment may be taken nearly equal to zero because beams are on opposite sides and they are of equal spans and carry equal loads. However, even if it is thought necessary to verify for the difference in moments on opposite sides due to different end conditions of beams *B6* and *B7* meeting at column *C8*, the calculations are given below :	
Unbalanced moment = $w_u L^2/10$ from beam *B6*-$w_u L^2/12$ from beam *B7*.	
For beams *B6* and *B7* , w_u = 20.15 *kN/m* , L = 4m = 4000 *mm*.	
M_F = 20.15 x 4²/10 – 20.15 x 4²/12 = 5.37 *kN.m*	
Beam B6 : 230 x 380 *mm*,	
k_{bL} = 0.75* (230 x 380³/12)/4000 = 197.20 x 10³ *mm*³	
Beam B7 : 230 x 380 *mm*,	
k_{bR} = (230 x 380³/12) / 4000 = 262.93 x 10³ *mm*³	
Column C8 : 230 x 230 *mm*,	
k_{col} = 0.75 (230 x 230³ / 12) / 3840 = 45.55 x 10³ *mm*³	
d_{col} = 45.55 / [45.55 + (197.20 + 262.93)/2] = 0.165	
M_{col} = 0.165 x 5.37 = 0.88 *kN.m* = M_{u2}	
Initial moment = M_i = 0.6 M_{u2} + 0.4 M_{u1} = 0.6 x 0.88 = 0.53 *kN.m*	
$M_{u.min}$ = P_u x ε_{min} = 318 x 20/1000 = 6.36 *kN.m* > M_i	
∴ Modified initial moment = 6.36 *kN.m*	
Additional moment = 7.04 *kN.m* (as obtained earlier)	
∴ Total design moment = 6.36 + 7.04	
∴ M_{uy} = 13.4 *kN.m*	
Since design moment M_{ux} (=15.14 *kN.m*) > M_{uy} (=13.4 *kN.m*) , the bending about *x - axis* governs.	
* k = 0.75 I / L since opposite end is taken rotation free.	

(b) Category - II : Column Under Axial load and Uniaxial bending - Group II (b) and II (c)

Group Column Mark		II (b) C13	II (c) C14, (C5, C6)
P_u in kN		293	188
Section b x D		230 x 230	230 x 230
N - ϕ		4-#12	4-#12
A_{sc}	mm^2	452	452
(i) Bending about x - axis			
$L_{eff.\,x}$		3840	3840
$L_{eff.\,x}/D$		16.7	16.7
M_{u1}	kN.m	0	0
M_{u2} (a) *Beam (230 mm x 380 mm)(Sect. 7.3.3.1)*		Right B11	Left B11
Type		T	T
(b) w_u	kN/m	31.65	31.65
(c) L	m	4.5	4.5
(d) $M_F = w_u L^2/24$	kN.m	26.7	26.7
(e) k_b x 10^3		2 x 233.7	467.4
(f) k_c x 10^3		45.55	45.55
(g) $d_{col} = k_c/(k_c + \Sigma k_b/2) = 45.55/(45.55+467.4/2)$		0.163	0.163
(h) $M_{col} = M_{u2} = M_F$ x d_{col} = 26.7 x 0.163		4.36	4.36
$M_i = 0.6\,M_{u2} + 0.4\,M_{u1} = 0.6$ x 4.36	kN.m	2.61	2.61
e_{min}	mm	20	20
$M_{u.min} = P_u$ x e_{min}	kN.m	5.86	3.76
Revised M_i (greater of M_i and $M_{u.min}$)	kN.m	5.86	3.76
P_{uz} *(Eq. 4.8.6a)*	kN	613	613
P_{ub} *(Eq. 4.8.13b)*	kN	187	187
k *(Eq. 4.8.13c)*		0.75	0.99
M_a *(Eq. 4.8.13)*	kN.m	7.02	5.98
$M_{ux} = M_i + M_a$		12.88	9.74
M_{urx}	kN.m	19.68*	21.95*
M_{urx} (=19.68 kN.m) > M_{ux} (12.88 kN.m)		∴ safe	∴ safe
(ii) Bending about y - axis			
M_i	kN.m	5.86	3.76
M_{ay}	kN.m	7.02	5.98
M_{uy}	kN.m	12.88	9.74
Since $M_{ux} = M_{uy}$, and bending about x - axis has been worked out to be safe ∴ o.k.			

***Note : Details obtained as per software*[7.5]**

Category - III, Column under axial load and Bi-axial bending - Group (a) and Group (b)

Column Group Column Mark		---(a)--- C11		---(b)--- C12, (C7,C15, C19,C20, C1)
P_u	kN	284		210
Section $b \times D$	mm	230 x 230		230 x 230
N - #	mm	6-#12**		4-#12
A_{sc}	mm^2	452		452
(i) Bending about x - axis				
$L_{eff.x}$	mm	3840		3840
$L_{eff.x}/D$		16.7		16.7
M_{ux1}	kN.m			
(a) Beam from		*Left*	*Right*	*Left*
Beam Mark *(Sect.7.3.3.2)*		*B9*	*B10*	*B10*
b mm x *D mm*		230x380	230x380	230x380
Beam Type		Rect	Flanged	Flanged
L	mm	4000	4500	4500
w_{u1}	kN/m	20.15	*UDL+PTL*	*UDL + PTL*
M_{F1}	kN.m	89.8@		44.9(=89.8/2)@@
$k_b \times 10^3$	mm^3	263	467	467
(b) $k_{col} \times 10^3$ $k_{col}+(\Sigma k_b/2)$	mm^3	45.55		45.55
		410.55		279.05
(c) $d_{col} = k_{col}/(k_{col} + \Sigma k_b/2)$		0.11		0.163
$M_{col} = M_{ux2} = d_{col} \times M_{F1}$		9.88		7.32
$M_{i.x} = 0.6\, M_{u2} + 0.4\, M_{u1}$		5.93		4.4
$e_{x.min}$	mm	20		20
$M_{ux.min} = P_u \times 0.02$	kN.m	5.68		4.20
Revised $M_{i.x}$	kN.m	5.93		4.4
P_{uz}	kN	681		613
P_{ubx}	kN	183.5		185.04
k		0.8		0.94
M_{ax}	kN.m	7.26		6.34
$M_{ux} = M_{ix} + M_{ax}$	kN.m	13.19		10.74
M_{urx}	kN.m	23.59		21.85

Column under axial load and Bi-axial bending continued ....

Column Group Column Mark		---(a)--- C11, (C7).	---(b)--- C12, (C15, C19, C6, C20, C1)
(ii) *Bending about y - axis*			
$L_{eff.y}$	mm	**3840**	**3840**
$L_{eff.y}/b$		16.7	16.7
M_{uy1}	kN.m	0	0
$M_{uy2} = M_{col}$	kN.m		
(a) *Beam from*		*Left*	*Left*
Beam Mark		*B23*	*B25*
b mm x *D*	mm	230 x 380	230x380
Beam Type		Flanged	Flanged
L	mm	5000	5000
w_u	kN/m	43	24.3
$M_F = w_u L^2/24$	kN.m	44.8	25.3
$k_{b1} \times 10^3$	mm^3	420	420
$k_{col} \times 10^3$	mm^3	45.55	45.55
$k_{col} + (\Sigma k_b/2)$		255.55	255.55
(c) $d_{col} = k_{col} = k_{col}/(k_{col} + \Sigma k_b/2)$		0.178	0.178
(d) $M_{col} = M_{uy2} = d_{col} \times M_F$	kN.m	**7.97**	**4.5**
$M_{iy} = 0.6\ M_{u2} + 0.4\ M_{u1}$	kN.m	4.8	2.7
$e_{y.min}$	mm	20	20
$M_{uy.min}$	kN.m	5.68	4.2
Revised $M_{i.y}$	kN.m	5.68	4.2
P_{uz}	kN	613	613
P_{uby}	kN	183	185.04
k		0.8	0.94
M_{ay}	kN.m	7.25	6.34
M_{uy}	kN.m	12.93	10.54
M_{ury}	kN.m	20.69	21.85
L.H.S. value		0.981 < 1	0.82 <1
Remarks		∴ safe	∴ safe

(1) ** Columns works out to be unsafe for #4-12*mm* hence steel revised to 6#12
(2) @ Final moment obtained after redistribution see design of beam *B10* item 7(a)
(3) @@ M_F is taken equal to 1/2 of moment at continuous end.

Table 7.5.2 Schedule of Column

Category No.	*Group*	*Column Mark*	*Concrete Grade*	*Section b (mm) x D(mm)*	*Main Steel No.- Di. (mm)*	*LateralTies Di. (mm) - s (mm)*
I		C2, C3, C4	M20	230 x 230	4 - #12	φ 6 - 190
II	(a)	C8, 9, 10, 16, 17, 18	M20	230 x 230	4 - #12	φ 6 - 190
	(b)	C13,	M20	230 x 230	4 - #12	φ 6 - 190
	(c)	C14, C5, C6	M20	230 x 230	4 - #12	φ 6 - 190
III	(a)	C11	M20	230 x 230	6 - #12	φ 6 - 190
	(b)	C12, 7, 15, 19, 20, 1	M20	230 x 230	4 - #12	φ 6 - 190

7.5 DESIGN OF COLUMN FOOTINGS

7.5.1 Categorisation of Footings

This is done exactly on the same lines of categorisation of columns.

Category - I : Axially Loaded Footing
Category - II : Eccentrically Loaded Footing - Uniaxial Bending
Category - III : Eccentrically Loaded Footing - Biaxial Bending
Category - IV : Combined Footing
Category - V : Raft or File Footing

It may be remembered that though basis of categorisation is the same for columns and footings, it is not necessary that the categories of a particular column and corresponding footing be same. For example, in the project under consideration, columns come under three different categories while footings for all columns are designed for axial loads only assuming rotation free condition at the base because of low bearing capacity of soil. *(See Sect. 3.2.4c). Thus, all column footings are under Category - I only.* For details of eccentric footings and combined footing see *Ref.* [7.6.]

7.5.2 Grouping of Footings

Since the size of footing depends on the load acting on the column base and the size of column, the columns with axial loads within ± 20% range and having the same cross-section can be grouped together to reduce the computational efforts and labour during the execution. The footing is designed for largest load in that group.

It may be noted that for design of footing, only the actual axial load at the column base, excluding the allowances for bending and slenderness shall be considered and the grouping of footings shall be done considering these loads. Accordingly, the footings for the building under consideration are grouped as follows:

Table. 7.5.1 Design Load for Axially Loaded Column footings

Group No.	Column Mark	Axial load kN	Max.load kN	Col.self wt kN	Design Load kN	Working Load kN
I	C8,C9,C10,C11, C13,C16,C17,C18	310, 305, 305, 276 285, 294, 290, 290	310	8	318	212
II	C5,C7,C12,C14, C15,C19	161, 201, 202, 180 168, 163	202	8	210	140
III	C6	124	124	8	132	88
IV	C1,C2,C3,C4,C20	65,93, 85, 85, 90	93	8	101	68

For details of loads see Table 7.4.1
* *Working Load = Design Load / 1.5*

7.5.3 Design of Footings

Illustrative design calculations according to the procedure explained in *Sect. 6.5* have been presented here only for footing under *Group - I i.e.* for column *C8* for design load of 318 *kN* (working load of 212 *kN*). The calculations for remaining footings are presented in a tabular form as per the details obtained from the software program[7.6] . Reader can very easily check the calculations.

7.5.4 Illustrated Design Calculation for Footing

Common Data:

Concrete grade, (Mild environment)			M20
Steel grade			Fe 415
Design Constant	$R_{u.max}$	=	2.76 N/mm^2
Column section	b	=	230 mm
	D	=	230 mm
Bearing capacity of soil	f_b	=	150 kN/m^2
Minimum depth of footing	$D_{f.min}$	=	150 mm
Offset at footing level	e	=	75 mm

Group - I Footing Design for Columns : *C8, C9, C10, C11, C13, C16, C17, C18*

Step	Design Calculations	Reference
(I)	***Data :*** Maximum Design ultimate column load P_u = 318 kN Working load $P = P_u/1.5 = 318/1.5 = 212$ kN	
(II)	***Proportioning of Base size*** Area of footing required $= A_f = 1.1\ P/f_b = 1.1 \times 212/150 = 1.55\ m^2$	Eq. 6.5.1
	Required Length of Footing L_f for equal projections : $L_f = (D-b)/2 + \sqrt{(D-b)^2/4 + A_f} = \sqrt{A_f}$ since $b = D$	Eq. 6.5.2a
	$\therefore L_f = \sqrt{1.55 \times 100}$ = 1245 mm Provide length of footing L_f = 1250 mm	
	Projection $C_x = C_y = (L_f - D)/2 = (1250 - 230)/2$ = 510 mm	Eq.6.5.2b
	Provide Width of footing $B_f = b + 2 \times C_y = 230 + 2 \times 510$ = 1250 mm Area of footing provided $A_f = L_f \times B_f = 1.25 \times 1.25$ = 1.5625 m^2 Upward factored soil pressure (due to P_u)	
	$w_u = P_u / A_f = 318/1.5625$ = 203.52 kN/m^2	Eq.6.5.3
(III)	***Depth of Footing from Bending Moment considerations :*** Breadth of footing at top $b_1 = b + 2e = 230 + 2 \times 75$ = 380 mm Bending Moment at column face parallel to *x-axis*	
	$M_{ux} = w_u B_f C_x^2/2 = 203.52 \times 1.25 \times 0.510^2/2$ = 33.1 kN.m	Eq.6.5.4a
	Length of footing at top $D_1 = D + 2e = 230 + 150$ = 380 mm Bending Moment at column face parallel to *y - axis*,	
	$M_{uy} = w_u L_f C_y^2/2 = 203.52 \times 1.25 \times 0.51^2/2$ = 33.1 kN.m	Eq.6.5.4b
	(a) Required effective depth for bending about *x - axis*, $d_x = \sqrt{\dfrac{M_{ux}}{R_{u.max} \times b_1}} = \sqrt{\dfrac{33.1 \times 10^6}{2.76 \times 380}}$ = 178 mm	Eq.6.5.5a
	$d_y = \sqrt{\dfrac{M_{uy}}{R_{u.max} \times D_1}} = \sqrt{\dfrac{33.1 \times 10^6}{2.76 \times 380}}$ = 178 mm	Eq.6.5.5b

Footing Design for Columns : *C8, C9, C10, C11, C13, C16, C17, C18 continued....*

Step	Design Calculations	Reference
	Assuming diameter of bar = 8 *mm*	
	Effective cover for bottom bars d'_x = 50 + 8/2 = 54 *mm*	
	Effective cover for top bars d'_y = 54 + 8 = 62 *mm*	
	Required depth of footing = 178 + 62 = 240 *mm*	
	Assume total depth D_f = 240 *mm*	
	Effective depth for bottom bars $= d_x = 240 - 54 = 186$ *mm*	
	Effective depth for top bars $= d_y = 240 - 62 = 178$ *mm*	
(IV)	***Check Depth of footing from Two - way shear Considerations***	
	Perimeter at critical section $= B2 = 2(b + D + 2d_y)$	
	$\therefore$ $= 2(230 + 230 + 2 \times 178) = 1632$ *mm*	*Eq.6.5.6a*
	Effective depth at peripheral section $= D_2 = D_f - y_2 - d'_y$	*Eq.6.5.6b*
	where , $y_2 = (d_y/2 - e)\, y_f/x_f = (178/2 - 75) \times 90/(510 - 75) = 2.89$ *mm*	
	$\therefore D_2 = 240 - 2.89 - 62 = 175.1$ *mm*	
	Area resisting shear $= A_2 =$ perimeter x $D_2 = 1632 \times 175.1 = 285763$ *mm*	*Eq.6.5.6c*
	Shear resisted by concrete $V_{uc2} = \tau_{uc2} \times A_2$	*Eq.6.5.6d*
	$\tau_{uc2} = k_s\, \tau'_{uc}$, $k_s = (0.5 + 230/230) \ngtr 1$ $\therefore k_s = 1$	
	$\tau'_{uc} = 0.25\sqrt{f_{ck}} = 0.25\sqrt{20} = 1.118\ N/mm^2$	
	$\therefore \tau_{uc2} = 1.118 \times 1 = 1.118\ N/mm^2$	
	$\therefore V_{uc2} = \tau_{uc2} \times A_2$ $1.118 \times 285763/1000 = 319.5\ kN$	
	Design shear $V_{uD2} = w_u\,[L_f B_f - (D + d_y)(b + d_y)]$	*Eq.6.5.6e*
	$= 203.52\,[1250 \times 1250 - (230+178)(230+178)] \times 10^{-6} = 317.8\ kN$	
	V_{uc2} $(=319.5\ kN) > V_{uD2}\ (= 317.8\ kN)$ $\therefore$ safe	
	Area of Steel and Check for Development Length	
	(a) $A_{st.min} = 0.85\, b_1\, d_x/f_y = 0.85 \times 380 \times 186/415 = 145\ mm^2$	
	$A_{stx} = \dfrac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \dfrac{4.6 \times 33.1 \times 10^6}{20 \times 380 \times 186^2}}\right] \times 380 \times 186 = 598\ mm^2$	*Eq.6.5.7a*
	Provide 12 Nos # 8 *mm*, Area provided $A_{st} = 600\ mm^2$	
	(b) $A_{sty.min} = 0.85\ D_f\ d_y/f_y = 0.85 \times 380 \times 178/415 = 139\ mm^2$	
	$A_{sty} = \dfrac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \dfrac{4.6 \times 33.1 \times 10^6}{20 \times 380 \times 178^2}}\right] \times 380 \times 178 = 642\ mm^2$	*Eq.6.5.7b*
	Provide 13 Nos # 8 *mm*, Area provided $A_{sty} = 650\ mm^2$	
	$(L_d)_{reqd} = (0.87 f_y / 4\tau_{bd})\, \phi = [0.87 \times 415/(4 \times 1.2 \times 1.6)] \times 8$	
	$= 376mm < (L_d)_{prov}\ (= 510 - 60 = 450\ mm)$	

Footing Design for Columns : C8, C9, C10, C11, C13, C16, C17, C18 continued....

Step	Design Calculations	Reference
(V)	***Check for One-way shear for bending about y - axis***	
	Critical section for one-way shear about y - *axis* is taken at a distance d_y from the face of column.	
	Depth of footing above rectangular portion at critical section.	
	$D_1 = y_1 - [(d_y - e)\, y_1/x_1]$	Eq.6.5.8a
	$y_1 = D_f - D_{f.min} = 240 - 150 = 90\ mm$	
	$x_1 = C_y - e = 510 - 75 = 435\ mm$	
	$\therefore D_1 = 90 - [(178 - 75) \times 90/435] = 68.7\ mm$	
	Width of footing at critical section $= B_2 = D + 2d_y = 230 + 2 \times 178 = 586\ mm$	Eq.6.5.8b
	Area of footing at critical section $= A_y = (B_2 + L_f)\, D_1/2 + (D_{f.min} - d'_y)\, L_f$	Eq.6.5.8c
	$\therefore A_y = (586 + 1250)\, 68.7/2 + (150 - 62) \times 1250 = 173067\ mm^2$	
	Percentage of steel $= p_{ty} = 100 \times 650/173067 = 0.37\%$,	Eq.4.4.3
	$\tau_{ucy} = 0.417\ N/mm^2$	
	Shear resisted by concrete $= V_{ucy} = \tau_{ucy} \times A_y = 0.417 \times 173067/1000 = 72.1\ kN$	Eq.6.5.8d
	Shear to concrete footing is subjected $= V_{uDy} = w_u L_f (C_y - d_y)$	Eq.6.5.8e
	$= 203.52 \times 1250\, (510 - 178) \times 10^{-6} = 84.4\ kN$	
	$V_{ucy}\ (= 72.1) < V_{uDy}\ (= 84.4\ kN)$ $\therefore$ ***unsafe***	
	Since the difference between V_{ucy} and V_{uDy} is large increase the depth of the footing.	
	Try depth of footing $D_f = 300\ mm$	
	Effective depth for bottom bars $d_x = 300 - 54 = 246\ mm$	
	Effective depth for top bars $d_y = 300 - 62 = 238\ mm$	
	$$\text{Required } A_{stx} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 33.1 \times 10^6}{20 \times 380 \times 246^2}}\right] \times 380 \times 246$$	
	$= 410\ mm^2$	
	Provide 9 No # 8*mm* bars, Area provided $A_{stx} = 450\ mm^2$	
	$$\text{Required } A_{sty} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 33.1 \times 10^6}{20 \times 380 \times 238^2}}\right] \times 380 \times 238$$	
	$= 428\ mm^2$	
	Provide 9 No # 8*mm* bars, Area provided $A_{sty} = 450\ mm^2$	
	Check for one-way shear for bending about y-axis for revised $D_f = 300$ mm :	
	Depth of footing above rectangular portion $D_1 = y_1 - [(d_y - e)\, y_1/x_1]$	
	$y_1 = D_f - D_{f.min} = 300 - 150 = 150\ mm$, $x_1 = C_y - e = 510 - 75 = 435\ mm$	
	$\therefore D_1 = 150 - [(238 - 75) \times 150/435] = 94\ mm$	
	Width of footing at critical section $B_2 = D + 2d_y = 230 + 2 \times 238 = 706\ mm$	

Footing for Design Columns : C8, C9, C10, C11, C13, C16, C17, C18 continued....

Step	Design Calculations	Reference
	Area of footing at critical section $A_y = (B_2 + L_f)\, D_1/2 + (D_{f.min} - d'_y)\, L_f$ $= (706 + 1250)\, 94/2 + (150 - 62) \times 1250 = 201932\ mm^2$	
	Percentage of steel $p_{ty} = 100 \times 450/201932 = 0.222\%$, $\tau_{uc} = 0.3376\ N/mm^2$	Eq.4.4.3
	Shear resisted by concrete $V_{ucy} = \tau_{ucy} \times A_y = 0.3376 \times 201932/1000 = 68.17\ kN$ Shear to which footing is subjected $V_{uDy} = w_u\, L_f (C_y - d_y)$ $= 203.52 \times 1250 \times (510 - 238)^{-6} \times 10$ $= 69.2\ kN$ $V_{ucy}\ (= 68.17\ kN) < V_{uDy}\ (= 69.2\ kN)$ ∴ ***unsafe*** Since difference between V_{ucy} and V_{uDy} is small increase the number of bars by 1 *i.e.* provide 10No. # 8 *mm* bars. Area provided $A_{sty} = 500\ mm^2$ $p_{ty} = 100 \times 500/201932 = 0.25\%$, $\tau_{uc} = 0.36\ N/mm^2$ Shear resisted by concrete $V_{ucy} = 0.36 \times 201932/1000 = 72.7\ kN > V_{uDy}$ ∴ ***safe***	
(VI)	***Check for one-way shear for bending about x - axis :*** Provide 9 Nos - # 8 *mm* bars. $A_{stx} = 450\ mm^2$, $d_x = 300 - 8/2 = 246\ mm^2$	
	$D_3 = y_1 (d_x - e)\, y_1/x_1 = 150 - (246 - 75) \times 150/435$ $= 91\ mm$	Eq. 6.5.9a
	Width at top of footing $= B_3 = b + 2d_x = 230 + 2 \times 246 = 722\ mm$ Area of footing at critical section $= A_x = (B_3 + B_f)\, D_3/2 + (D_{f.min} - d'_x)\, B_f$ $= (722 + 1250)\, 91/2 + (150 - 54) \times 1250$ $= 209726\ mm^2$	Eq.6.5.9b
	Percentage steel $p_{tx} = 100\, A_{stx}/A_x = 100 \times 450/209726 = 0.215\%$ $\tau_{uc} = 0.332\ N/mm^2$	Eq.4.4.3
	Shear resisted by concrete $V_{ucx} = 0.332 \times 209726 / 1000 = 69.6\ kN$ Shear to which footing is subjected $V_{uDx} = w_u \times B_f (C_x - d_x)$ $= 203.52 \times 1250\, (510 - 246) \times 10^{-6}$ $= 67.1\ kN$ $V_{ucx}\ (= 69.6\ kN) > V_{uDx}\ (= 67.1 kN)$ ∴ safe	Eq.6.5.9c
(VII)	***Check for bearing pressure at column Base*** $P_u/(bD) < 0.45 f_{ck} \times \sqrt{A_1/A_2}$ where $\sqrt{A_1/A_2} \not> 2$ $A_1 = L_f \times B_f$ *or* $(b + 4D_f)\ (D + 4D_f)$ whichever is less $= 1250 \times 1250$ *or* $(230 + 4 \times 300)^2$ $= 1562500\ mm^2$ *or* $2044900\ mm^2$ $\therefore A_1 = 1562500\ mm^2$ $A_2 = b \times D = 230 \times 230 = 52900\ mm^2$ $\sqrt{A_1/A_2} = \sqrt{1562500/52900} = 5.4 > 2$ $\therefore \sqrt{A_1/A_2} = 2$	Eq. 6.5.10

Footing Design for Columns : C8, C9, C10, C11, C13, C16, C17, C18 continued....

Step	Design Calculations	Reference
	Actual bearing pressure $= P_u/bD = 318 \times 1000/(230 \times 230) = 6.01\ N/mm^2$	
	Permissible bearing pressure $= 0.45\ f_{ck}\ \sqrt{A_1/A_2} = 0.45 \times 20 \times 2$ $= 18\ N/mm^2 > 6.01\ N/mm^2$ ∴ safe	
(VIII)	**RESULTS :**	
	Size of footing : 1250 *mm* x 1250 *mm*	
	Total depth of footing : 300 *mm*	
	Minimum depth of footing : 150 *mm*	
	No - Dia of bars along long direction : 9-#8	
	No- Dia. of bars along short direction : 10-#8	
	Clear distance between bars along long direction : 133 *mm*	
	Clear distance between bars along short direction : 118 *mm*	

7.5.5 Design of remaining footing in Tabular form

The results of the following footings have been obtained from the generalised software for design of footing prepared by the author [7.7]. The load on column *C1* is very small hence it has been included in group *IV* only.

Group - II Columns *C5, C7, C12, C14, C15, C19,*
Maximum ultimate load $= P_u = 210\ kN,$

Group - III Column *C6*
Maximum ultimate Column load $P_u = 132\ kN$

Group - IV Columns *C1, C2, C3, C4, C20*
Maximum ultimate column load $P_u = 101\ kN$

Footing Design for Columns Under Group II, III and IV

Step	Design Calculations	Group II	Group III	Group IV	Reference
(I)	**Data :**				
	Maximum column load P_u	210 *kN*	132 *kN*	101 *kN*	
	Design working load $= P = P_u/1.5$	140 *kN*	88 *kN*	68 *kN*	
	Column Section *mm* x *mm*	230 x 230	230 x 230	230 x 230	
	Bearing capacity of soil	150kN/m^2	150kN/m^2	150kN/m^2	
	Materials used	M20, Fe415	M20, Fe415	M20, Fe415	
(II)	***Proportioning of Base size***				
	Area of footing required m^2	1.0267	0.6453	0.499	*Eq.6.5.1*
	Area of footing provided m^2	1.0404	0.7396	0.7396	
	Length of footing provided *mm*	1020	810/860*	710/860*	*Eq.6.5.2a*
	Breadth of footing provided *mm*	1020	810/860*	710/860*	
	Projection from column face *mm*	395	315	315	
	$w_u = P \times 1.5$/Area of footing kN/m^2	201.84	178.47	137.9	*(Eq.6.5.3)*

* Length and Breath of Footing increased to satisfy development requirements

Step	Design Calculations	Group II	Group III	Group IV	Reference
(III)	***Depth of footing required from B.M. Considerations :***				
	M_{ux} kN.m	16.06	7.61	5.88	
	M_{uy} kN.m	16.06	7.61	5.88	
	Depth for B.M mm	180	140	120	Eq. 6.5.5
(IV)	***Depth of footing required from Two-way shear consideration***				
	Perimeter at critical Sect. B_2	1432	1272	1272	
	Depth at peripheral Sect. D_2	128	88	88	
	Area resisting shear A_2	183296	111936	111936	
	Shear resisted by concrete V_{uc2}	204.93	125.15	125.15	
	Design shear V_{uD2}	184.13	113.95	81.54	
	Depth for 2 - way shear $(D_f)_{req}$	190	150	150	Eq. 6.5.6
(V)	***Depth increased to satisfy one-way shear requirements $(D_f)_{prov}$***	200@	160@	150	
	Revised calculations for two-way shear				
	B_2 mm	1472	1312	1272	
	D_2 mm	138	98	88	
	A_2 mm^2	203136	128576	111936	
	V_{uc2} kN	227.11	143.75	125.1	
	V_{uD2} kN	182.66	112.8	88.05	
	Area of Steel				
	Area of steel provided about y-axis mm^2	502.6(10-#8)	301.6(6#8)	201.1(4#8)	
	Area of steel provided about x-axis mm^2	452.4(9-#8)	251.3(5#8)	201.1(4#8)	
(V)	***Check for One-way Shear for Bending about y-axis***				
	A_y mm^2	120399	84280	75680	
	p_{ty}	0.417	0.358	0.26	
	τ_{ucy} N/mm^2	0.445	0.417	0.368	
	V_{ucy} kN	53.57	35.20	27.89	Eq. 6.5.8
	V_{uDy} kN	52.91	33.31	26.92	
(VI)	***Check for One-way shear for bending about x - axis :***				
	A_x mm^2	127917	91160	82560	
	p_{tx}	0.353	0.276	0.24	
	τ_{ucx} N/mm^2	0.416	0.374	0.355	
	V_{ucx} kN	53.17	34.13	29.32	
	V_{uDx} kN	51.26	32.08	25.97	

Step	Design Calculations	Group II	Group III	Group IV	Reference
(VII)	**Check for bearing pressure at column base :**				
	Actual Bearing pressure	18.0	18.0	18.0	Eq. 6.5.10
	Permissible pressure	3.97	2.49	1.93	
(VIII)	**Results**				
	Length of footing L_f	1020 mm	860	860	
	Breadth of footing B_f	1020 mm	860	860	
	Total depth of footing D_f	200 mm	160	150	
	Mini. depth of footing $D_{f.min}$	150 mm	150	150	
	N-# of bars along long direction N_x - #	9-#8 mm	5-#8 mm	4-#8 mm	
	N-# of bars along short direction N_y #	10-#8 mm	6-#8 mm	4-#8 mm	
	Clear dist. of bars along long direction C_{LX}	105 mm	178 mm	242 mm	
	Clear dist. of bars along short direction C_{LY}	93 mm	142 mm	242 mm	

SCHEDULE OF FOOTING

Column Marks	b x d	L_f	B_f	Offset at top	D_f	D_f min.	N_x -#	S_x	N_y #	S_y
C8,C9,C10,C11, C13,C16,C17,C18	230x230	1250	1250	75	300	150	9-#8	133	10-#8	118
C5,C7,C12,C14, C15,C19	230x230	1020	1020	75	200	150	9-#8	105	10-#8	93
C6	230x230	860	860	75	160	150	5-#8	178	6-#8	142
C1,C2,C3,C4,C20	230x230	860	860	-	150	150	4-#8	242	4-#8	242

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7.2 Shah, V.L. and Karve, S.R., "Handbook of reinforcement concrete design", Structures Publications, Pune,411009

7.3 Purushothaman,P. "Reinforced Concrete Structural Element ", Tata McGraw-Hill, New Delhi, ,1984, Chapter-8 , Page.424

7.4 SP 34 : 1987 "Handbook of concrete reinforcement and detailing", BIS, New Delhi, 1987

7.5 Shah,V.L., "Generalised program for design of axially loaded column, column subjected to axial load and uniaxial bending and biaxial bending including slender column , " Structures Publications, Pune,411009, *Software packages No.3.*

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7.7 Shah,V.L. "Design of Pad *or* Sloped footing for axially loaded column", Structures Publications, Pune, 2009 , *Software package No. 3.*

CHAPTER - 8

PROJECT - II : DESIGN OF MULTI-STOREYED COMMERCIAL BUILDING

8.1 INTRODUCTION

In the first project, the detailed design of a single storey public building was presented. Since, the object of that project was to illustrate the design of all types of members of a R.C. building from first principles, less emphasis was given on the analysis, and the members (slabs, beams and columns) were assumed to be simply connected. This assumption can be considered to be valid essentially for a single storeyed structure. In the case of a multistoreyed structure, with the increase in height, the effect of horizontal loads requires consideration. Therefore, such structures are provided with rigid frames having rigid joints. If a multistoreyed structure is assumed to have simple connections, it is likely to collapse under the action of horizontal loads (in absence of walls) due to lack of rigid connections between the component members. In a rigid frame, forces get distributed between the component members due to rigidity of connection and hence, analysis of the structures as a whole becomes necessary. Therefore, a four storeyed office building having a regular layout and which can be divided into a number of similar vertical plane frames has been considered in this project to illustrate the analysis and design of a rigid jointed plane frame. Since the degree of accuracy required in analysis for such a R.C. building is not very high, the substitute frame method has been used. The method is suitable for hand computation for non-sway structures.

The analysis of one intermediate floor frame has been illustrated giving detailed calculations for all the three different types of substitute frames (*viz.* Floor frame, Bay frame, Beam-column Systems) used in the analysis. The results obtained by the three methods have been compared to examine the relative merits and demerits of each in regards to simplicity and degree of accuracy.

The purpose of this project is not to design the entire building but to illustrate the simplified frame analysis method and design the connected members. The detailed analysis of residential building has been given in *Project - 3 Chapter - 9.*

8.2 SALIENT FEATURES

A floor plan of a typical commercial building is shown in *Fig.8.2.1.*

Following are some of the salient features of the plan which are worth noting :

(1) The plan is regular in nature in the sense that it has all columns equispaced in longitudinal direction. Thus, the entire building space frame can be divided into a number of vertical plane frames.

(2) According to the requirements of an architect, there has to be only one line of internal columns along one side of the corridor. The transverse beams are, therefore, two span continuous beams with unequal spans. The span of beams have been taken equal to centre to centre distance between the columns.

(3) The longitudinal wall on the other side of the corridor is required to be supported by a longitudinal beam continuous over series of transverse beams of the frames. Thus, the two span continuous transverse beams are not only subjected to uniformly distributed load due to floor load but also a point load from the supported longitudinal beam.

(4) According to requirements of the Code, various loading arrangements (alternate span loaded, adjacent span loaded etc.) are required to be considered when live load (*LL*) exceeds 0.75 times the dead load (*DL*). For only beams carrying walls, this condition will not arise if wall load is taken as dead load*. Therefore, the frame along columns 22-13-3 has been taken for analysis on which there are no walls, so that *LL* is greater than 0.75*DL*. If the probability of construction of walls on every frame in future is to be considered, it would be safe to design the frame with wall loads but considering wall load as live load. However, in that case, the design will be very conservative and uneconomical. In this design, this probability has not been considered.

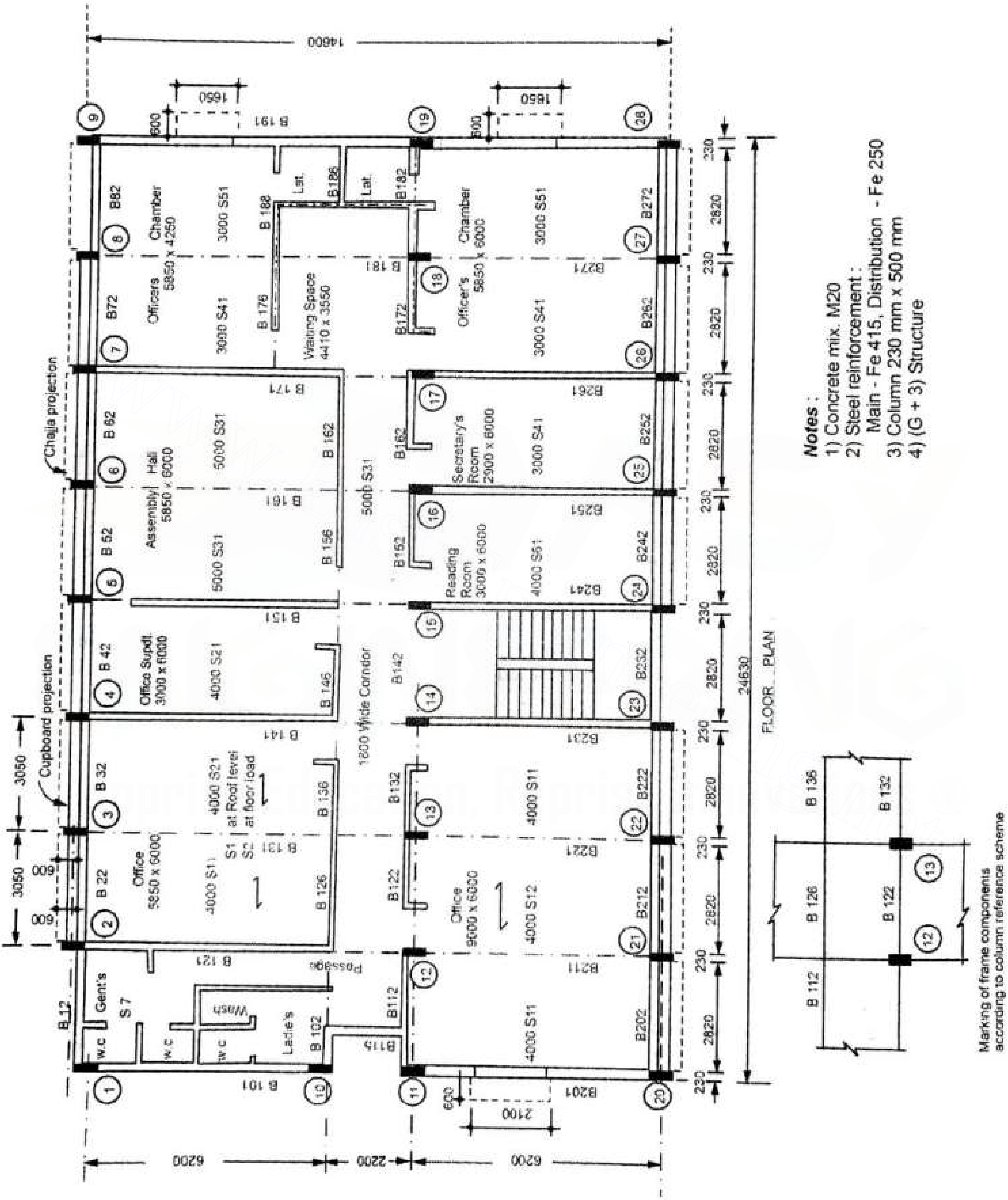


Fig. 8.2.1 Floor Plan of Typical Commercial Building

Remarks : **In the opinion of the authors, the load of the internal walls, which are liable to be removed to convert two rooms into a hall or vice-versa, should be considered as live load and not dead load. This is because either such existing walls are likely to be demolished or new walls constructed in future according to the requirements of the user.*

(5) According to another requirement of the architect, all the columns are required to be of equal size 230*mm* x 500*mm* right from bottom to top, and R.C.C. floor shall also be provided at the plinth level.

(6) The various rooms are used for different purposes. Therefore, the live loads for all rooms are not the same. (*For example*, Office Chamber - 3kN/m^2, Office 4kN/m^2, Stairs-Passages and Assembly Hall - 5 kN/m^2, *etc*). Therefore, though practically all frames are geometrically similar, they are subjected to different loads. This can be seen from the following :

Frame	*Left (Short) Span*	*Right (Long) Span*
21-12-2	No wall	Internal Wall Only *UD* load.
22-13-3	No wall.	No wall.
23-14-4	Internal wall with office floor load on one side.	Internal wall with office floor load on both sides.
24-15-5	Internal wall with reading room floor load from one side	Internal wall with office floor load on one side and assembly hall load on the other.
25-16-6	Internal wall	No wall
26-17-7	Internal Wall	Internal wall with two point (beam) loads at different locations
27-18-8	No wall	No wall but beam (point) load at different location
28-19-9	External wall	External wall.
This requires analysis to be done separately, practically, for all frames.		

8.3 DATA

1. Type of Structure : Multistorey rigid jointed frame
2. Layout : As shown in *Fig. 8.2.1*
3. Number of Storeys : Four. (Ground + 3)
4. Floor to floor height : 3.35 *metres*.
5. Height of plinth : 1.2 *metre*. above G.L.
6. Depth of foundation : 2.1 *metres* below G.L.
7. External walls : 250 *mm* thick including plaster.
8. Internal walls : 150 *mm* thick including plaster, 2.2 *m* high.
9. Impose Load : As per *Table A-2*
10. Exposure conditions : Mild Environment *(Table C-1)*
11. Bearing Capacity of soil : 200 kN/m^2
12. Materials : *Concrete* : M20, *Steel* : Main Fe415, Secondary Fe250.
13. Design Philosophy : Limit State Method conforming to *IS:456-2000*[8.1]
14. Design Assumptions : All members of the main frames are rigid jointed. Simple connections in case of other components.

8.4 LOADS

(a) *Dead Load :*

(i) *Finishes.*

(a) Terrace waterproofing 2.5 kN/m^2 (b) Floor finish 1.0 kN/m^2

(ii) *Slab :* 25D kN/m^2 where, D is depth of slab in *metres*.

(iii) *Walls :* External 250 *mm* thick : 20 x 0.25 = 5 *kN/m/metre height.*
Internal 150 *mm* thick : 20 x 0.15 = 3 *kN/m/metre height.*

(b) *Imposed Load* [8.2].

(i) Roof : 1.5 kN/m^2, (ii) Office floors : 4kN/m^2,
(iii) Reading room : 4kN/m^2, (iv) Officers Chamber : 3 kN/m^2,
(v) Assembly Hall : 5kN/m^2, (vi) Stairs, Corridors : 5 kN/m^2
(vii) Sanitary Blocks : Public 3kN/m^2,

8.5 DESIGN OF FRAME

The design of frame 3-13-22 consists of the following :

(a) Design of slab , (b) Analysis of Frame , (c) Design of Transverse Beam , (d) Design of Longitudinal Beams, (e) Design of Columns , and (f) Design of Footings

8.6 DESIGN OF MEMBERS

Slab Marks		*Roof Slabs S1*	*Floor Slabs S2*	*Reference*
Type		One-way Continuous	One-way Continuous	
End Condition No.		2	2	
Design moment Coefficient α		1/10	1/10	
Span L	*metres*	3.05	3.05	
Load : FF	kN/m^2	2.50	1.00	
LL	kN/m^2	1.50	4.00	
Total Working load w_t	kN/m^2	4.00	5.00	
Depth :				
For deflection[8.3, 8.4],	*D mm*	110	110	Sect.6.2.2(3)
p_t = 0.4%, α_1 = 1.33, r_b = 26,				*Fig. 4.7.1*
Effective depth=3050/(26 x 1.33)	*d mm*	90	90	
Loads :				
Self weight = 25*D* = 25 x 0.11	kN/m^2	2.75	2.75	
Dead Load w_d = Self + *FF*	kN/m^2	5.25	3.75	
Live Load w_L = *LL*	kN/m^2	1.50	4.00	
Total working Load $w = w_d + w_L$	kN/m^2	6.75	7.75	
Ultimate Load w_u = 1.5 *w*	kN/m^2	10.125	11.625	
Design Moment : $M_{u.max} = w_u L^2/10$	*kN.m*	9.42	10.81	
Main Steel **(Fe415)**	Area mm^2	313	364	*Eq. 6.2.3*
Diameter and spacing	*mm*	#8@160	#8@130	*TableH-3*
Distribution Steel **(Fe250)**	Area mm^2	165	165	
Diameter and Spacing	ϕ-s *mm*	ϕ6@170	ϕ6@170	*TableH-3*
End Shear :				
Long Edge w_u *L/2*	*kN/m*	15.44	17.73	
Short Edge w_u *L/6*	*kN/m*	5.15	5.91	
End Shear due to factored live load				
load on long edge = $1.5 w_L x L/2$	*kN/m*	3.43	9.15	

8.7 ANALYSIS OF FRAME : *See Fig. 8.7.1*

8.7.1 Member Data :

Member Details		4-5	5-6	Note No.
(a) Beams				
(1) Size : $b \times D$	*mm*	230x500	230x500	(1)
(2) Length L	*mm*	6200	8400	
(3) Moment of Inertia	mm^4			
(i) Rectangular section $I = bD^3/12$		2396×10^6	2396×10^6	
(ii) Multiplying factor for flanged section		2	2	(2)
(iii) I of flanged Section	mm^4	4792×10^6	4792×10^6	
(4) *Stiffness* $k_b = I / L$	mm^3	***0.77x10^6***	***0.57x10^6***	
(b)Columns :		*Upper* (4-1, 5-2, 6-3)	*Lower* (4-7, 5-8, 6-9)	
(1) Size $b \times D$	*mm*	230x500	230x500	
(2) Length L	*mm*	3350	3350	(3)
(3) Moment of Inertia I - $bD^3/12$	mm^4	2396×10^6	2396×10^6	
(4) ***Stiffness*** $k_c = I/L$	mm^3	***0.715 x 10^6***	***0.715 x 10^6***	

Explanatory Notes :

(1) Even though span lengths of two beams are different, the depth has been taken the same for both the spans from practical considerations.

(2) Since, both the beams are flanged, the moment of inertia of flanged section is taken equal to 2 times that of a rectangular beam. See *Sect.3.2.5.*

(3) Length of column for the purpose of calculation of stiffness is to be taken equal to centre to centre distance *i.e.* floor to floor height (See *Sect. 5.2.3d or Clause 22.2(d)*).

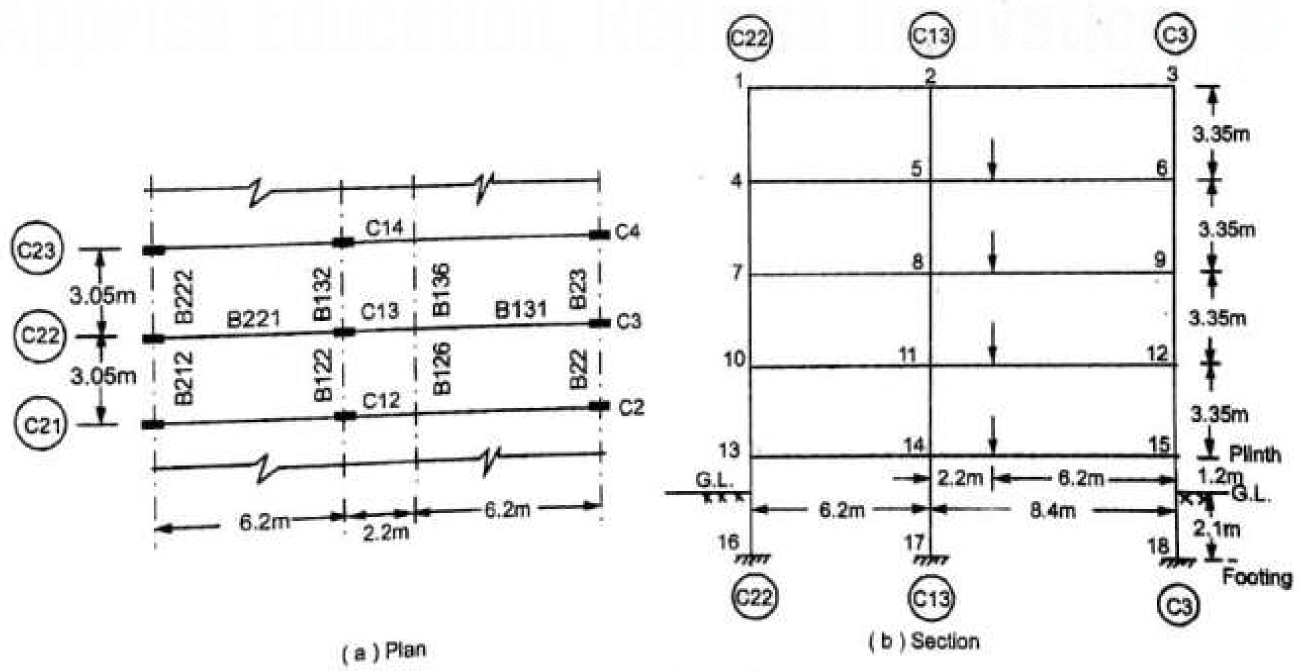


Fig. 8.7.1 Cross - section of plane frame

8.8 LOAD DATA

8.8.1 Roof Level :

Slab : 110mm thick : Dead Load : w_d = Self Wt.+ FF
= 25 x 0.11 + 2.5 = 5.25 kN/m^2

Live Load : w_L = = **1.50** kN/m^2

Total Working Load : w_t = 5.25 + 1.50 = 6.75 kN/m^2

Total Ultimate Load : w_u = 1.5 x 6.75 = **10.125** kN/m^2

Parapet Wall :250mm thick x 1m high : w_w = 20x .25x 1 = **5.00** kN/m

(1) Longitudinal Beams : (see Fig. 8.7.1a)

(a) External beams B22, B23, B212, B222 : Assuming Section 230mm x 300mm

DL : Self : 25 x 0.23 x (0.3 – 0.11) = 1.10 kN/m
Parapet Wall : = 5.00 kN/m
Slab : w_d x L_x/6 = 5.25 x 3.05/6 = **2.67** kN/m*
Total dead load w_d = 8.77 kN/m

LL : Slab : w_L x L_x/6 = 1.50 x 3.05/6 = 0.76 kN/m*
Total Working Load w = (DL + LL) = 9.53 kN/m

Maximum Ultimate Load for beam design :
w_{ub} = 1.5 x 9.53 = 14.30 kN/m

Maximum Dead load for column design excluding triangular slab load
= 1.5(Self + wall) = 1.5(1.1 +5.0) = 9.15 kN/m

Beam end shear as column load = 9.15 x 3.05/2 = **13.96** kN (from each side)

∴ Total load on column = 2 x 13.96 = **27.92 kN**

Note:*This is a triangular load transferred to the longitudinal beam as explained in Sect.5.3.2. It is taken for beam design only and not for column design because it is already considered while transferring the load to the transverse beams. Therefore, to avoid duplication it is excluded while computing the load on the column.*

(b) Internal beams : B122, B132, B126, B136 : Assuming section 230mm x 300mm

DL : Self : (Same as external) = 1.10 kN/m
Slab: 2 (w_d x L_x/6)=2 x 2.67 = 5.34 kN/m
Total dead load w_d = 6.44 kN/m

LL : Slab :2 (w_L x L_x/6) =2x 0.76 = 1.52 kN/m
Total working load, w = DL + LL = 7.96 kN/m

Maximum ultimate load for beam design w_u = 1.5 (DL+LL)=1.5 x 7.96 = 11.94 kN/m

Maximum ultimate load for column design excluding triangular slab load
= 1.5 x self = 1.5 x 1.1 = 1.65 kN/m only.

Beam End Shear as column load = 1.65 x3.05/2 = **2.52** kN (from each side)

∴ Total load on column = 2 x 2.52 = **5.04 kN**

(2) Transverse Beam : B221, B131 : Section 230mm x 500mm

DL : Self : 25x0.23 x (0.5–0.11) = 2.24 kN/m
Slab : (25x0.11+2.5) x 3.05 = 16.01 kN/m
Total dead load w_d = 18.25 kN/m

LL : Slab : 1.50 x 3.05 w_L = 4.58 kN/m
Total working load w = w_d + w_l = 22.83 kN/m

Maximum load w_{max} = 1.5(DL + LL) = 1.5 x 22.83 = 34.24 kN/m **say 35 kN/m**

Minimum load w_{min} = 18.25 = 18.25 kN/m **say 18 kN/m**

Ratio LL/Total Load = 4.58/22.83 = **0.2**

8.8.2 Floor Level :

Slab : Dead Load : Self weight + floor finish

w_d = 25x0.11 + 1 = 3.75 kN/m^2

Live Load : w_L = 4.00 kN/m^2

Total working load w = $w_d + w_L$ = 7.75 kN/m^2

Total ultimate load w_u = 1.5 x w = **11.625** kN/m^2

Wall : *Internal* : 150*mm* thick (including plaster) , 2.2*m* high

w_{w1} = 20 x 0.15 x 2.2 = 6.60 kN/m

External : 250*mm* thick (including plaster)

Assuming external longitudinal beam 450*mm* deep

Wall height = 3.35 – 0.45 = 2.90*m*.

w_{w2} = 20 x 0.25 x 2.9= 14.50 kN/m

Since the internal walls are not at all likely to be demolished as they form the sides of a passage, the load of the internal wall just as the load of external walls, is taken as dead load and not live load.

(1) *Longitudinal Beams :*

(a) *Internal* : Section 230*mm* x 300*mm*

Dead Load :	Self weight = 25 x 0.23 x (0.3 – 0.11)	= 1.10	kN/m
	Wall : w_{w1} = 20 x 0.15 x 2.2	= 6.60	kN/m
	Slab* : 2 x (3.75 x 3.05/6) = 2 x 1.91	= **3.82**	kN/m
	Total dead load	= 11.52	kN/m
Live Load :	: 2 x (4 x 3.05/6) = 2 x 2.03	= **4.06**	kN/m
Total working load		= 15.58	kN/m
Total ultimate load	w_u = 1.5 x 15.58	= **23.37**	kN/m

(b) *External* : Section 230 *mm* x 450 *mm*

Dead Load :	Self w_t = 25 x 0.23 x (0.45 – 0.11)	= 1.96	kN/m
	Wall = w_{w2} = 20 x 0.25 x 2.9	= 14.50	kN/m
	Slab* = w_d L_x /6 = 3.75 x 3.05/6	= **1.91**	kN/m
	Total dead load	= 18.37	kN/m
Live Load :	= w_L L_x /6 = 4.00 x 3.05/6	= **2.03**	kN/m
Total working load	w	= 20.40	kN/m
Total ultimate load	w_u = 1.5 x 20.4	= **30.6**	kN/m

(2) *Main Transverse Beam* : *B221* in *Fig. 8.2.1* (*or* 4-5 in *Fig. 8.7.1*)

Size - 230 *mm* x 500 *mm* , L = 6.2 *m*

(a) *Uniformly Distributed Load :*

Dead load :	Self weight : 25 x 0.23 x (0.5 – 0.11)	= 2.24	kN/m
	Slab : (25 x 0.11 +1) x 3.05	= 11.44	kN/m
	Total dead load	= 13.68	kN/m
Live load :	Floor : 4 x 3.05	= 12.20	kN/m
	Total Working Load w = $DL+LL$ = 13.68 + 12.20	= 25.88	kN/m
Maximum Load :	w_{max} = w_1 = 1.5 (DL + LL)		
	= 1.5 (13.68 + 12.2)	= 38.82	kN/m **say 39 kN/m**
Minimum Load :	w_{min} = DL = 13.68	= 13.68	kN/m **say 13 kN/m**
Ratio *LL*/Total load	= 12.20/25.88	= **0.47**	

(b) *Point Load* : Nil

(3) Main Transverse Beam : *B131 in Fig. 8.2.1 (or* Beam 5-6 in *Fig. 8.7.1)*

(a) *Uniformly Distributed Load* : Same as that for *Beam* 4-5.

$w_{max} = w_1 = \quad = 39$ kN/m

$w_{min} = w_2 = \quad = 13$ kN/m

(b) Point Load : Shear from Internal Longitudinal Beams *B126* and *B136*

Dead Load = 11.52 x 3.05 = 35.14 kN

Live load = 4.06 x 3.05 = 12.38 kN

Maximum Load : P_{max} = P1 = 1.5 (DL + LL)

P_{max} = 1.5 (35.14 + 12.38) = 71.30 kN **say 72 kN**

Minimum Load : P_{min} = P2 = DL = 35.14 kN **say 35 kN**

Note : *Though the slabs S1-S2 are spanning one way across transverse main beams, part of slab load over a triangular area with central ordinate ($wL_x/4$) is transferred to longitudinal beam as explained in Sect.5.3.2.*

8.8.3 Plinth Level :

(1) Longitudinal Beams : (Provided at G.L)

External : Section 230mm x 450mm,

Self : 25 x 0.23 x 0.45 = 2.6 kN/m

Wall : 250mm thick,(3.35 + 1.2 − 0.45) = 4.1 m

w_w = 20 x 0.25 x 4.1 = 20.5 kN/m

Total working load, w = 23.1 kN/m

Total Ultimate load w_u=1.5w =1.5x23.1 = 34.7 kN

Load on each column = 2 x 34.7 x 3.05/2 = say **106** kN

Internal : Section 230mm x 300mm. *(provided at G.L and supported by column)*

Self : 25 x 0.23 x 0.3 = 1.73 kN/m

Wall : 150mm thick, (2.2 + 1.2) = 3.4 high

w_w = 20 x 0.15 x 3.4 = 10.2 kN/m

Total working load w = 1.73 + 10.2 = 11.93 kN/m

Total ultimate load w_u =1.5 x 11.93 = say 18 kN/m

Load on each column = 2 x 18 x 3.05/2 = say **55** kN

(2) Transverse Beams :

(1) As specified by the architect the transverse beam with floor slab will be provided at plinth level.

(2) The internal longitudinal wall supported by transverse beam will be 2.2m high and point load transferred to the transverse beam will be the P_{max} = 72 kN and P_{min} = 35 kN as obtained earlier.

Comments: *Normally, concrete floor only is provided at the plinth level. But if treacherous soil, like black cotton soil, exists for a very large depth it is either required to be removed or if the flooring is provided it cracks badly due its peculiar nature of soil swelling in wet season and cracking in dry season. In such cases R.C.C. slab is provided at the plinth level also along with the transverse beam. The transverse beam also helps in reducing the effective length of a column and avoids column becoming slender.*

8.8.4 Fixed End Moments : for Transverse floor beam *(see Fig. 4.7.1)*

(a) *Span 4-5* : L = 6.2 metres

Maximum : $M_{F45} = w_1 L^2/12 = 39 \times 6.2^2/12$ = 124.93 kN/m = M_{F54}

Minimum : $M_{F45} = w_2 L^2/12 = 13 \times 6.2^2/12$ = 41.64 kN.m = M_{F54}

(b) *Span 5-6* : L = 8.4 m , Point Load at 2.2 metres from support 5.

Maximum : $M_{F56} = 39 \times 8.4^2/12 + 72 \times 2.2 \times 6.2^2/8.4^2$ = 315.61 kN.m

$M_{F65} = 39 \times 8.4^2/12 + 72 \times 2.2^2 \times 6.2/8.4^2$ = 259.94 kN.m

Minimum : $M_{F56} = 13 \times 8.4^2/12 + 35 \times 2.2 \times 6.2^2/8.4^2$ = 118.39 kN.m

$M_{F65} = 13 \times 8.4^2/12 + 35 \times 2.2^2 \times 6.2/8.4^2$ = 91.32 kN.m

8.9 SUBSTITUTE FRAME - I (S.F.I) : FLOOR FRAME

The substitute frame-I *or* floor frame having second degree of approximation *(see Sect. 3.2.2a)* is used for analysis using Moment Distribution Method. The process of distribution has been carried out only for 3 cycles working up to two places of decimal. This much accuracy is considered adequate for analysis of a R.C. building frame.

Distribution Factors :

Joint	Member	RSF*	SUM	DF = RSF/SUM
4	4-1 4-5 4-7	0.715 0.77 0.715	2.2	0.325 0.35 0.325
5	5-2 5-6 5-8 5-4	0.715 0.570 0.715 0.770	2.77	0.26 0.20 0.26 0.28
6	3-6 6-9 6-5	0.715 0.715 0.570	2.00	0.36 0.36 0.28

***RSF = Rotational Stiffness Factor = I/L (see Sect. 8.7.1)*

Fig.8.9.1 shows values of the members.

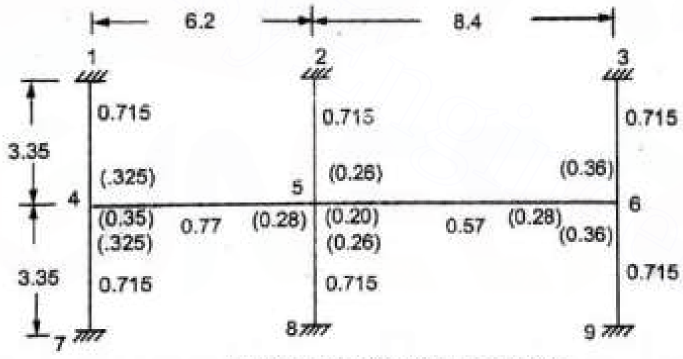


Fig.8.9.1

Since live load (4 kN/m^2) > 0.75 times Dead load (.75 x 3.75 kN/m^2), all loading arrangements will have to be considered to obtain the design moments and shears.

Loading Case - I Maximum Span moment in span 4-5,

For this, there shall be maximum load on span 4-5, and minimum load on span 5-6 as shown in *Fig.8.9.2*

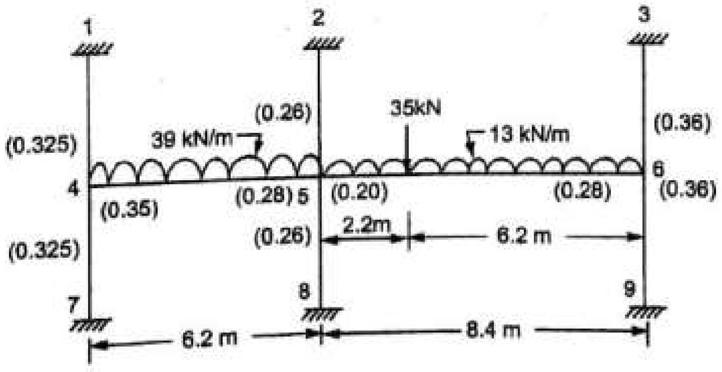


Fig. 8.9.2 Loading Case - I : Maximum Span Moment in 4-5

Moment Distribution

Joint	4		5			6	
Members	Cols	4-5	5-4	Cols	5-6	6-5	Cols
Distribution factors	0.65	0.35	0.28	0.52	0.20	0.28	0.72
FEM	-	−124.93	124.93	–	−118.39	91.32	–
Balance	81.20	43.73	−1.83	−3.40	−1.31	−25.57	−65.75
C.O.	-	−0.92	21.86	–	−12.78	−0.66	–
Balance	0.60	0.32	−2.54	−4.72	−1.82	0.18	0.48
C.O.	-	−1.27	0.16	–	0.09	−0.91	
Balance	0.83	0.44	−0.07	−0.13	−0.05	0.25	0.66
Final Moment in *kN.m*	82.63	−82.63	142.51	−8.25	−134.26	64.61	−64.61
Moment in each column	41.32			−4.1			−32.30

Shear :

$V_{4\text{-}5}$ = 39 x 6.2/2 − (142.51 − 82.63)/6.2 = 111.24 *kN*

$V_{5\text{-}4}$ = 39 x 6.2/2 + (142.51 − 82.63)/6.2 = 130.56 *kN*

$V_{5\text{-}6}$ = 13 x 8.4/2 + 35 x 6.2/8.4 +(134.26 − 64.61)/8.4 = 88.72 *kN*

$V_{6\text{-}5}$ = 13 x 8.4 + 35 − 88.72 = 55.48 *kN*

Maximum Span Moment :

Span 4-5 : x_{max} = 111.24/39 = 2.852 *m* from joint 4 *(Eq.2.6.1)*

M_{max} = 111.24 x 2.852/2−82.63 = 76.0 *kN.m* *(Eq.2.6.2)*

Span 5-6 : x_{max} = 55.48/13 = 4.27*m* < 6.2*m* from joint 6 ∴ o.k.

M_{max} = 55.48 x 4.27/2 − 64.61 = 53.84 *kN.m*

Loading Case - II : Maximum Span Moment in Span 5-6

For this, there shall be maximum load on span 5-6, and minimum load on span 4-5 as shown in *Fig. 8.9.3.*

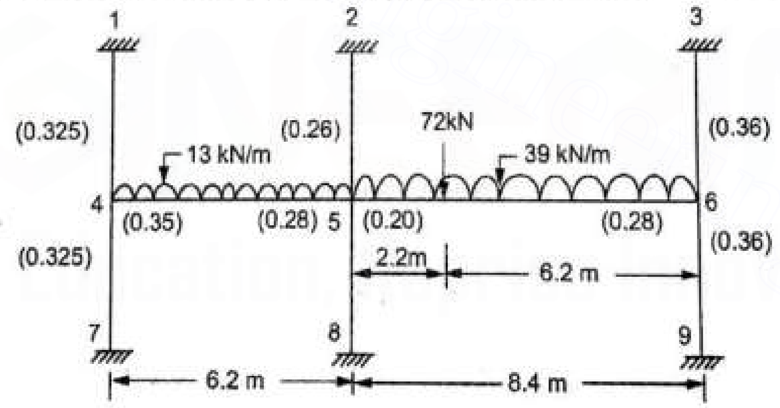


Fig. 8.9.3 Loading Case - II : Maximum Span Moment in 5-6

Moment Distribution

Joint	4		5			6	
Members	Cols	4-5	5-4	Cols	5-6	6-5	Cols
Distribution factors	0.65	0.35	0.28	0.52	0.20	0.28	0.72
FEM	-	−41.64	41.64	–	−315.61	259.94	–
Balance	27.07	14.57	76.71	142.46	54.80	−72.80	−187.14
C.O.	-	38.36	7.28	–	−36.40	27.40	–
Balance	−24.93	−13.43	8.15	15.14	5.82	−7.67	−19.73
C.O.	-	4.08	−6.72	–	−3.84	2.91	
Balance	−2.65	−1.43	2.96	5.49	2.11	−0.81	−2.10
Final Moment in *kN.m*	−0.51	0.51	130.02	163.09	−293.12	208.97	−208.97
Moment in each column	−0.25			81.54			−104.48

Shear : $V_{4\text{-}5}$ = 13 x 6.2/2 − (130.02 + 0.51)/6.2 = 19.24 *kN*

$V_{5\text{-}4}$ = 13 x 6.2/2 + (130.02 + 0.51)/6.2 = 61.35 *kN*

$V_{5\text{-}6}$ = 39 x 8.4/2 + 72 x 6.2/8.4 + (293.12 − 208.97)/8.4 = 226.96*kN*

$V_{6\text{-}5}$ = 39 x 8.4/2 + 72 x 2.2/8.4 − (293.12 − 208.97)/8.4 = 172.64*kN*

Maximum Span Moments :

Span 4-5 : x_{max} = 19.24/13 = 1.426*m* from joint 4

M_{max} = 19.24 x 1.48/2 + 0.51 = 14.75 *kN.m*

Span 5-6 : x_{max} = 172.64/39 = 4.427*m* (from joint 6) < 6.2*m* ∴ o.k.

M_{max} = 172.64 x 4.427/2 − 208.97 = 173.17 *kN.m*

Loading Case - III : Maximum support moment at 5

The corresponding loading case shall be both spans carry maximum loads as shown in *Fig.8.9.4*

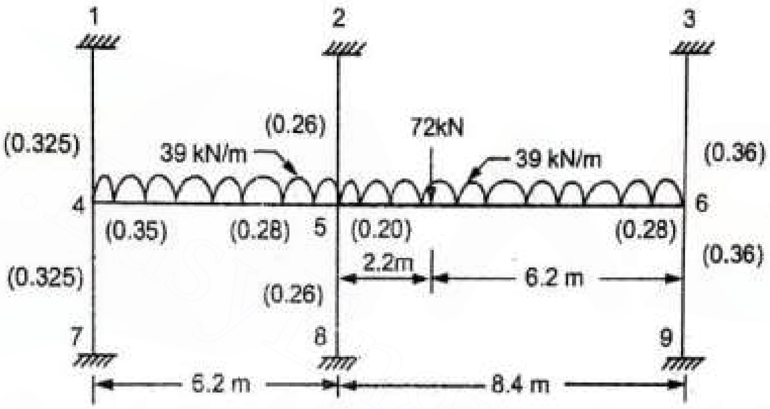


Fig. 8.9.4 Loading Case - III : Maximum Support Moment at joint 5

Moment Distribution

Joint	4		5			6	
Members	Cols	4-5	5-4	Cols	5-6	6-5	Cols
Distribution factors	0.65	0.35	0.28	0.52	0.20	0.28	0.72
FEM	-	−124.93	124.93	−	−315.61	259.94	−
Balance	81.20	43.73	53.39	99.15	38.14	−72.80	187.14
C.O.	-	26.70	21.86	−	−36.40	19.07	−
Balance	−17.36	−9.34	4.07	7.56	2.91	−5.34	−13.73
C.O.	-	2.04	−4.67	−	−2.67	1.45	
Balance	−1.33	−0.71	2.05	3.82	1.47	−0.41	−1.04
Final Moment in *kN.m*	62.51	−62.51	201.63	110.53	−312.16	201.91	−201.91
Moment in each column	31.25			55.26			−100.95

Shear :

$V_{4\text{-}5}$ = 39 x 6.2/2 − (201.63 − 62.51)/6.2 = 98.46 *kN*

$V_{5\text{-}4}$ = 39 x 6.2/2 + (201.63 − 62.51)/6.2 = 143.34 *kN*

x_{max} = 98.46/39 = 2.52*m* from joint 5

M_{max} = 98.46 x 2.52/2 − 62.51 = 61.55 *kN.m*

$V_{5\text{-}6}$ = 39 x 8.4/2 + 72 x 6.2/8.4 + (312.16 − 201.91)/8.4 = 230.06 *kN*

$V_{6\text{-}5}$ = 39 x 8.4/2 + 72 x 2.2/8.4 − (312.16 − 201.91)/8.4 = 169.54 *kN*

x_{max} = 169.54/39 = 4.347*m* from joint 6 < 6.2*m* ∴ o.k.

M_{max} = 169.54 x 4.347/2 − 201.91 = 166.58 *kN.m*

Results of Substitute Frame - I :Bending Moment in kN.m

Loading	Jt. 4		5						6
CaseNo.	Col.	4-5	Span mmt	5-4	Cols	5-6	Span mmt	6-5	Col
I	41.32	82.63	76.0	142.51	4.10	134.26	53.84	64.61	32.30
II	0.25	0.51	14.75	130.02	81.54	293.12	173.17	208.97	104.48
III	31.25	62.51	61.55	201.63	55.26	312.16	166.58	201.91	100.95
Max.mmt	41.32	82.63	76.00	201.63	81.54	312.16	173.17	208.97	104.48
Loading Case		I	I	III	II	III	II	II	II

Results of Substitute Frame - I : Shear in kN

Loading Case No.	Jt 4		5	6
I	111.24	130.56	88.72	55.48
II	18.54	61.35	226.96	172.64
III	98.46	143.34	230.06	169.54
Maximum shear	111.24	143.34	230.06	172.64
Loading case	I	III	III	II
Maximum column load	111.24	373.40		172.64

8.10 SUBSTITUTE FRAME - II : BAY FRAME

In this method having third degree of approximation, separate bay frames will be considered for span *4-5* and span *5-6* as given in *Sect. 3.2.2(b)*. Besides, both the both spans will be considered ignoring outer columns to determine maximum support moment at intermediate support

Loading Case - I :Maximum Span Moment in Span 4-5, and Maximum Support Moment at 4.

The substitute frame will be as shown in *Fig. 8.10.1*. In this case, span 4-5 shall carry maximum load and span 5-6 minimum load just as in *Case-I* of Substitute frame - I *(Sect 8.9.2)*. Since the effect of columns at joint 6 is ignored here, the stiffness of beam 5-6 is reduced to half [8.5, 8.6, 8.7].

The distribution factors are calculated as under.

Distribution Factors :

Joint	Member	R.S.F.	SUM	D.F
4	4-1	0.715	2.2	0.325
	4-5	0.770		0.350
	4-7	0.715		0.325
5	5-2	0.715	2.485	0.288
	5-6	0.57/2		0.114
	5-8	0.715		0.288
	5-4	0.77		0.310

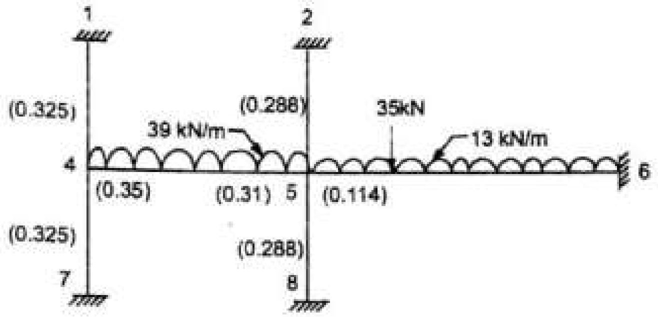


Fig. 8.10.1 Loading Case - I : Maximum Span Moment in 4-5

Fixed end moments will be the same as calculated in *Sect. 8.8.4*.

Joint	4		5		
Members	Cols	4-5	5-4	Cols	5-6
Distribution factors	0.65	0.35	0.31	0.576	0.114
FEM	–	−124.93	+124.93	-	−118.39
Balance	81.20	43.73	−2.03	−3.77	−0.740
C.O.	–	−1.01	21.86	-	-
Balance	0.66	0.35	−6.78	−12.59	−2.49
C.O.	–	−3.39	0.17	-	-
Balance	2.20	1.19	−0.05	−0.10	−0.02
Final Moment	84.06	−84.06	138.10	−16.46	−121.64
Col. moment	42.03	-	-	−8.23	-

Shear :

$$V_{4\text{-}5} = 39 \times 6.2/2 - (138.10 - 84.06)/6.2 = 112.18\ kN$$

$$V_{5\text{-}4} = 39 \times 6.2/2 + (138.10 - 84.06)/6.2 = 129.61\ kN$$

Maximum span moment :

$$x_{max} = 112.18/39 = 2.876\ m$$

$$M_{max} = 112.18 \times 2.876/2 - 84.06 = 72.25\ kN.m$$

Loading Case - II : Maximum Mid-span Moment in Span 5-6, Maximum Support Moment at 6.

The substitute bay frame for this case will be as shown in Fig. 8.10.2. In this case, span 4-5 shall carry maximum load as in Case - II of Substitute Frame - I *(Sect. 8.9.3)*. Since effect of column at 4 is ignored here the stiffness of beam 4-5 is reduced to half. The distribution factors are calculated as under. The distribution factor at jt. 6 remains same as that in Substitute Frame - I. However, it changes at joint 5.

Distribution factors :

Joint	*Member*	*R.S.F.*	*SUM*	*D.F.*
5	5-2	0.715	2.385	0.300
	5-6	0.570		0.239
	5-8	0.715		0.300
	5-4	0.77/2		0.161
6	6-3	0.715	2.0	0.36
	6-9	0.715		0.36
	6-5	0.570		0.28

Fixed end moments will be the same as calculated in *Sect.8.8.4.*

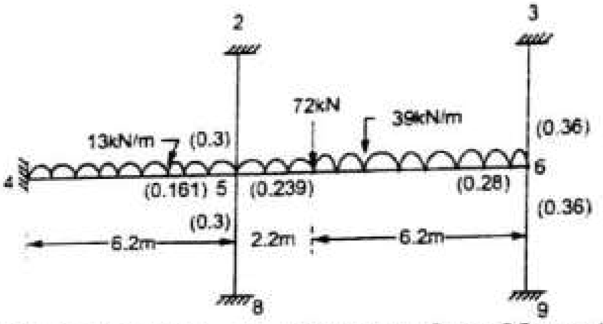


Fig. 8.10.2 Loading Case - II : Maximum Span Moment in 5-6

Joint	5			6	
Members	5-4	Cols	5-6	6-5	Cols
Distribution factors	0.161	0.6	0.239	0.28	0.72
FEM	41.64	-	−315.61	259.94	-
Balance	44.11	164.38	65.48	−72.78	−187.16
C.O.	-	-	−36.40	32.74	-
Balance	5.86	21.84	8.70	−9.17	−23.57
C.O.	-	-	−4.58	4.35	-
Balance	0.74	2.75	1.09	−1.22	−3.13
Final Moment	92.35	188.97	−281.32	213.86	−213.86
Column moment	-	94.48	-	-	−107.0

Shear : $V_{6-5} = 39 \times 8.4/2 + 72 \times 2.2/8.4 - (281.32 - 213.86)/8.4 = 174.62\ kN$

$V_{5-6} = 39 \times 8.4/2 + 72 \times 6.2/8.4 + (281.32 - 213.86)/8.4 = 224.97\ kN$

$x_{max} = 174.62/39 = 4.477m$ from joint 6 < 6.2 m

$M_{max} = 174.62 \times 4.477/2 - 213.86 = 177.03\ kN.m$

Loading Case - III : Maximum Support Moment at 5.

The substitute frame for this case will be as shown in *Fig. 8.10.3* which will have both spans 4-5 and 5-6 fixed at their for ends. In this case, both spans will carry max. load just as in *Case - III* of Substitute *Frame -I (Sect. 8.9.4)*. The distribution factors at Joint 5 are given as under:

Distribution Factors :

Joint	Member	R.S.F.	SUM	D.F
5	5-2	0.715	2.1	0.340
	5-6	0.57/2		0.136
	5-8	0.715		0.340
	5-4	0.77/2		0.184

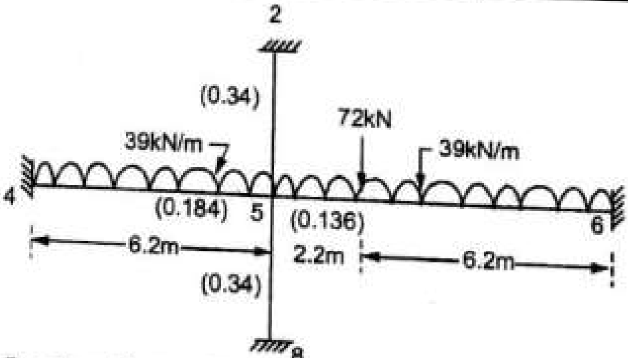


Fig. 8.10.3 Loading Case - III : Maximum Support Moment at joint 5

Joint	4	5			6
Members	4-5	5-4	Col	5-6	6-5
D.F	–	0.184	0.68	0.136	–
F.E.M	–124.93	124.93	–	–315.61	259.94
Balance	–	35.09	129.66	25.93	–
C.O.	17.54	–	–	–	12.96
Final Moment	–107.39	160.02	129.66	–289.68	272.90
Col. moment	–	–	64.83	–	–

Shear : $V_{5\text{-}4}$ = 39 x 6.2/2 + (160.02 – 107.39)/6.2 = 129.39 *kN*

$V_{5\text{-}6}$ = 39 x 8.4/2 + 72 x 6.2/8.4 + (289.68 – 275.90)/8.4 = 218.58 *kN*

Results of Substitute frame - II : Bending Moments in kN.m

Case: No.	Beam Moments in *kN.m*						Moment in each column		
	4-5	Span mmt.	5-4	5 5-6	Span mmt.	6-5	4-1and 4-7	5-2and5-8	6-3and6-9
I	84.06	72.25	138.10	121.64	–	–	42.03	8.23	–
II	–	–	92.35	281.32	177.03	213.86	–	94.48	107.0
III	–	–	160.02	289.68	–	–	–	64.83	–
Max.mmt	84.06	72.25	160.02	289.68	177.03	213.86	42.03	94.48	107.0
Loading Case	*I*	*I*	*III*	*III*	*II*	*II*			

Results of Substitute Frame II : Shear in kN

Loading Case	*Joint 4*	5		6
I	112.18	129.61	–	–
II	–	–	224.97	174.62
III	–	129.39	218.58	–
Maximum shear	112.18	129.61	224.97	174.62
Maximum Column Load	112.18	354.58		174.62

8.11 SUBSTITUTE FRAME METHOD - III : BEAM-COLUMNS SYSTEM

8.11.1 Beam System

As detailed in *Sect. 3.2.2c*, in this approach, moments in beams are obtained by considering floor beam as a continuous beam simply supported over column neglecting totally the fixity offered by the columns. All possible loading arrangements are considered.

Joint	*Member*	*R.S.F.*	*SUM*	*D.F*
5	5-4	0.77	1.34	0.575
	5-6	0.57		0.425

Initial fixed end moments will be the same as those calculated for Substitute Frame Method -I *(Sect. 8.8.4.)*

(a) Loading Case - I : Maximum Span Moment in Span 4-5

In this case, span 4-5 will carry maximum load while there will be minimum load on span 5-6. *See Fig. 8.11.1.* The Distribution factors are calculated as under.

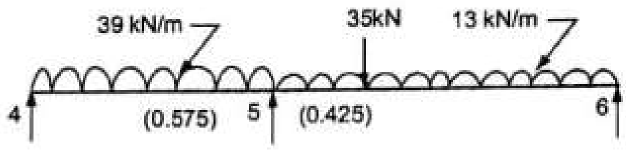


Fig. 8.11.1 Loading Case - I : Maximum Span Moment in 4-5

Moment Distribution :

Joint	4	5		6
Member	4-5	5-4	5-6	6-5
D.F.	–	0.575	0.425	–
F.E.M.	−124.93	124.93	−118.39	91.32
	+124.93	62.46	−45.66	−91.32
Balance		−13.42	−9.92	
Final Moments	0	173.97	−173.97	0

Shear :

V_{4-5} = 39 x 6.2/2 − 173.97/6.2 = 92.84 *kN*
V_{5-4} = 39 x 6.2/2 + 173.97/6.2 = 148.96 *kN*
V_{5-6} = 13 x 8.4/2 + 35 x 6.2/8.4 + 173.97/8.4 = 101.14 *kN*
V_{6-5} = 13 x 8.4/2 + 35 x 2.2/8.4 − 173.97/8.4 = 43.05 *kN*

Maximum span Moment :

Span 4-5 : x_{max} = 92.84/39 = 2.38 *m*
M_{max} = 92.84 x 2.38/2 − 0 = 110.48 *kN.m*

(b) Loading Case - II : Maximum Span Moment in Span 5-6

In this case, maximum load shall occur on Span *5-6* and minimum load on Span *4-5* as shown in *Fig. 8.11.2.* Distribution factors remain unchanged.

Moment Distribution :

Joint	4	5		6
Member	4-5	5-4	5-6	6-5
D.F.		0.575	0.425	
F.E.M.	−41.64	41.64	−315.61	259.94
	41.64	20.82	−129.97	−259.94
Balance		220.30	162.83	
Final Moment	0	282.76	−282.76	0

Shear :

$V_{4\text{-}5}$ = 13 x 6.2/2 − 282.76/6.2 = −5.31 *kN*

$V_{5\text{-}4}$ = 13 x 6.2/2 + 282.76/6.2 = 85.91 *kN*

$V_{5\text{-}6}$ = 39 x 8.4/2 + 72 x 6.2/8.4 + 282.77/8.4 = 250.61 *kN*

$V_{6\text{-}5}$ = 39 x 8.4/2 + 72 x 2.2/8.4 − 282.77/8.4 = 148.99 *kN*

Maximum span moment :

Span 5-6

x_{max} = 148.99/39 = 3.82*m* from joint 6

M_{max} = 148.99 x 3.82/2 − 0 = 284.57 *kN.m*

(c) *Loading Case - III : Maximum Support Moment at 5.*

In this case, maximum load shall occur on both the spans *4-5* and *5-6* as shown in *Fig. 8.11.3.* Distribution factors remain unchanged.

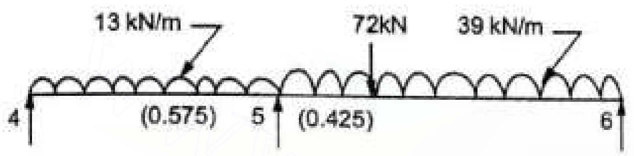


Fig. 8.11.2 Loading Case - II : Maximum Span Moment in 5-6

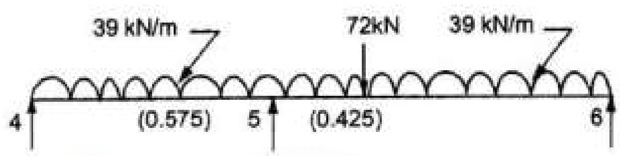


Fig. 8.11.3 Loading Case - III : Maximum Support Moment at joint 5

Moment Distribution :

Joint	4	5		6
Member	4-5	5-4	5-6	6-5
D.F.		0.575	0.425	
F.E.M.	−124.93	124.93	−315.61	259.94
Bals	124.93	62.46	−129.97	−259.94
Balance	–	148.46	109.73	–
Final Moment	0	335.85	-335.85	0

Shear :

$V_{4\text{-}5}$ = 39 x 6.2/2 − 335.85/6.2 = 66.73 *kN*

$V_{5\text{-}4}$ = 39 x 6.2/2 + 335.85/6.2 = 175.07 *kN*

$V_{5\text{-}6}$ = 39 x 8.4/2 + 72 x 6.2/8.4 + 335.85/8.4 = 256.93 *kN*

$V_{6\text{-}5}$ = 39 x 8.4/2 + 72 x 2.2/8.4 − 335.85/8.4 = 142.67 *kN*

Maximum Span Moment

Span 4-5 : x_{max} = 66.73/39 = 1.711*m*

M_{max} = 66.73 x 1.711/2 − 0 = 57.08 *kN.m*

Span 5-6 : x_{max} = 142.67 /39 = 3.658*m* < 6.2 *m*

M_{max} = 142.67 x 3.658/2 − 0 = 260.94 *kN.m*

Note : *The continuous beam sub-frame approximation ignores the fixity that may be imparted by the end columns at joint 4 and joint 6. If this approximation is used it is obviously necessary to provide some reinforcement at the top of the beam at end supports at joints 4 and 6 to account for partial fixity. This value may be taken equal to 30% to 40% of the initial fixed end moment or the beam moments and shear may be modified taking into account column moments.*

Results of Substitute Frame - III Beam Moment and Shear

	Beam Moment					*Shear*			
Case No.	Jt.4	Span mmt.	5	Span mmt.	6	4	5	5	6
I	0	110.48	−173.97	−	0	92.84	148.96	101.14	43.05
II	0	−	−282.76	284.57	0	−5.31	85.91	250.61	148.99
III	0	57.08	−335.85	260.94	0	66.73	175.07	256.93	142.67
Maximum	0	110.48	−335.85	284.57	0	92.84	175.07	256.93	148.99

8.11.2 Column System

Moments in columns are obtained by considering substitute column frame which consists of only the relevant column together with connected beams fixed at their far ends. The stiffness of beams is reduced to half. This sub-frame can be used to find column moments only. The relevant column systems with loadings for maximum moments in columns are shown below :

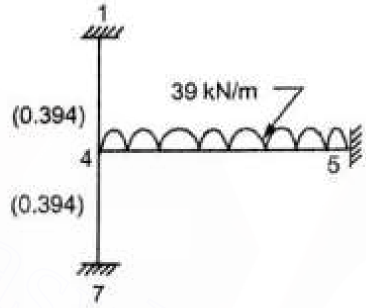


Fig. 8.11.4 Loading Case for Maximum Column Moment at joint 4

Loading Case - I : Maximum Moment at joint 4

ΣK = 0.715 + 0.77/2 + 0.715 = 1.815

Distribution factor for column : D_{col} = 0.715/1.815 = 0.394

M_{col} = 0.394 x 39x 6.2^2/12 = 49.22 *kN.m*

Loading Case - II : Maximum Moment at joint 6

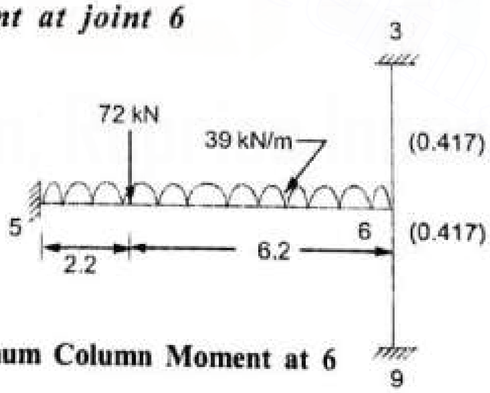


Fig. 8.11.5 Loading Case II for Maximum Column Moment at 6

ΣK = 0.715 + 0.715 + 0.57/2 = 1.715

D_{col} = 0.715/1.715 = 0.417

M_{6-5} = 72 x 2.2^2 x $6.2/8.4^2$ + 39 x 8.4^2/12 = 259.94 *kN.m*

M_{col} = 0.417 x 259.94 = 108.4 *kN.m*

Loading Case - III : Maximum Moment at joint 5

ΣK = 0.715 + 0.77/2 + 0.715 + 0.57/2 = 2.1

D_{col} = 0.715/2.1 = 0.34

Unbalanced moment at the joint

M_e = 72 x 2.2 x $6.2^2/8.4^2$ + 39 x 8.4^2/12 − 13 x 6.2^2/12 = 273.97 *kN.m*

M_{col} = 0.34 x 273.97 = 93.15 *kN.m*

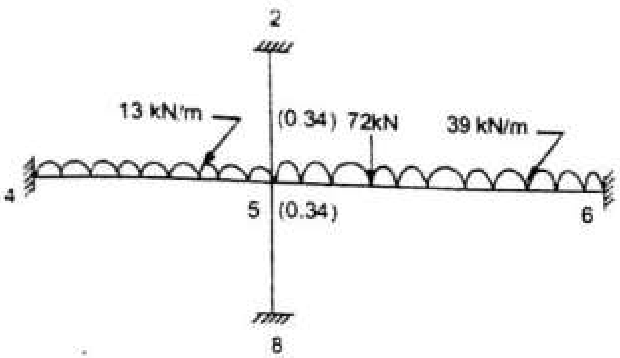


Fig. 8.11.6 Loading Case for Maximum Column Moment at 5

Results :

Column Moments kN.m			
Joint	4	5	6
	49.22	93.15	108.4

8.12 COMPARISON OF RESULTS OF THREE METHODS *viz.* SUBSTITUTE FRAME - I, II AND III

Comparison of Beam and Column Moments in *kN.m*

Method	Joint 4				5				6
	Col	4-5	Span mmt.	5-4	Col.	5-6	Span mmt.	6-5	Col.
Substitute Frame - I	41.32	82.63	76.00	201.63	81.54	312.16	173.17	208.97	104.48
Substitute Frame - II	42.03	84.06	72.25	160.02	94.48	289.68	177.03	213.86	107.00
Substitute Frame - III	49.22	0	110.48	335.85	93.15	335.85	284.57	0	108.4

Comparison of Shear in kN

Method	Jt. 4	5		6
Substitute Frame - I	111.24	143.34	230.06	172.64
Substitute Frame - II	112.18	129.61	224.97	174.62
Substitute Frame - III	92.84	175.07	256.93	148.99

Comparison of Column Moments

Method	Jt. 4	5	6
Substitute Frame - I	41.32	81.54	104.48
Substitute Frame - II	42.03	94.48	107.00
Substitute Frame - III	49.22	93.15	108.4

Remarks

Methods namely substitute frame - I and substitute frame - II give practically the same results except that moments at intermediate support for *Method - II* are lower by about 10%. However, since design moment is allowed to be lower by maximum of 30% when redistribution of moments is done, these values by 10% could be acceptable.

Unmodified values of *Method - III* are far in excess of values of *Method-I* by more than 40% to 60% at Mid-span and about 70% at intermediate support. This is because of assumption of simple support at outer columns. This method of disregarding the effect of columns is, thus, most uneconomical and therefore, should be avoided. The above *Method-III* is also not recommended by

IS code vide SP:24[8.8]. As mentioned earlier since some fixity will be imparted to the beam at its ends (which have been assumed simply supported) the reinforcement corresponding to *30% to 40% of initial fixed end moment* may be provided to avoid cracking.

Observations for Beam Shear, Column Moments and Concluding Remarks :

The values of beam shears and column moments obtained by substitute frame-*II* do not deviate much from those of substitute frame-*I* except that the beam shear at intermediate support are lower by about 5% and the moments in intermediate columns are higher by about 20%.Substitute frame - II does not save much of labour and time.Substitute frame-*III* is a very conservative alternative and should definitely be avoided and only to be used when fast results are required to be obtained.

Therefore, *it can concluded that the Substitute Frame Method-I should always be preferred to other two methods.*

The Top and Bottom frames in the subsequent sections have been analysed using Substitute frame-*I*.

8.13 TOP STOREY FRAME (ROOF LEVEL)

The top storey frame at roof level will be analysed using *substitute frame - I*. Since there will not be any longitudinal beam supported by transverse beam,the slab will be subjected to only uniformly distributed load having maximum value w_{max}=35 *kN/m* and minimum of w_{min}=18 *kN/m* (*See Sect. 8.8.1(2)*)

Loads : w_{max} = 35 *kN/m* , w_{min} = 18 *kN/m*, No point load.

Fixed End Moments :

Span 1-2 : L = 6.2 *m*

Maximum : $M_{F1-2} = 35 \times 6.2^2/12 = 112.12 kN.m = M_{F21}$

Minimum : $M_{F1-2} = 18 \times 6.2^2/12 = 57.66 kN.m = M_{F21}$

Span 2-3 : L = 8.4 *m*

Maximum : $M_{F23} = 35 \times 8.4^2/12 = 205.8 kN.m = M_{F32}$

Minimum : $M_{F23} = 18 \times 8.4^2/12 = 105.84 kN.m = M_{F32}$

Distribution Factors :

Joint	Member	RSF	SUM	D.F
1	1-2	0.770	1.485	0.52
	1-4	0.715		0.48
2	2-1	0.77	2.055	0.37
	2-3	0.57		0.28
	2-5	0.715		0.35
3	3-2	0.57	1.285	0.44
	3-6	0.715		0.56

Loading Case - I : *Maximum Span moment in span 1-2*

In this case maximum load shall be on span 1-2 and minimum on 2-3 as shown in *Fig. 8.13.1*

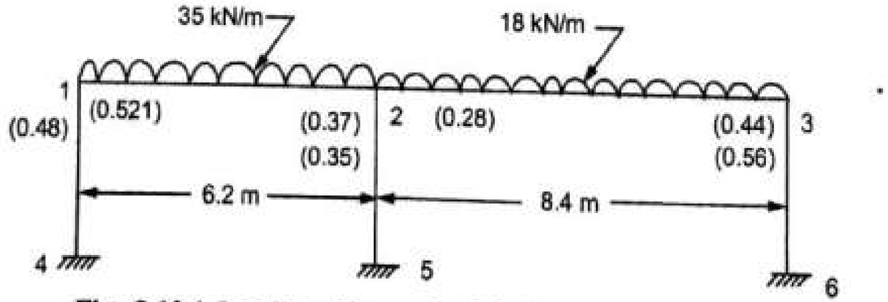


Fig. 8.13.1 Loading Case - I : Maximum Span Moment in 1-2

Joint	1		2			3	
Member	1-4	1-2	2-1	2-5	2-3	3-2	3-6
DF	0.48	0.52	0.37	0.35	0.28	0.44	0.56
FEM	–	−112.12	112.12	–	−105.84	105.84	–
Balance	53.82	58.30	−2.32	−2.20	−1.76	−46.57	−59.27
C.O	–	−1.16	29.15	-	−23.28	−0.88	–
Balance	0.56	0.60	−2.17	−2.06	−1.64	0.39	0.49
C.O.	–	−1.08	0.30	-	0.19	−0.82	–
Balance	0.52	0.56	−0.18	−0.17	−0.14	0.36	0.46
Final Moments	54.90	−54.90	136.90	−4.43	−132.47	58.32	−58.32

Shear :

$V_{1-2} = 35 \times 6.2/2 - (136.9 - 54.9)/6.2 = 95.28\ kN$

$V_{2-1} = 35 \times 6.2/2 + (136.9 - 54.9)/6.2 = 121.72\ kN$

$V_{2-3} = 18 \times 8.4/2 + (132.47 - 58.32)/8.4 = 84.43\ kN$

$V_{3-2} = 18 \times 8.4/2 - (132.47 - 58.32)/8.4 = 66.77\ kN$

Maximum Span moments :

Span 1-2 : $x_{max} = 95.28/35 = 2.722\ m$

$M_{max} = 95.28 \times 2.722/2 - 54.9 = 74.78\ kN.m$

Span 2-3 : $x_{max} = 66.77/18 = 3.71\ m$

$M_{max} = 66.77 \times 3.71/2 - 58.32 = 65.54\ kN.m$

Loading Case - II *Maximum span moment in 2-3*

In this case maximum load shall be on span 2-3 and minimum on 1-2 as shown in *Fig. 8.13.2*

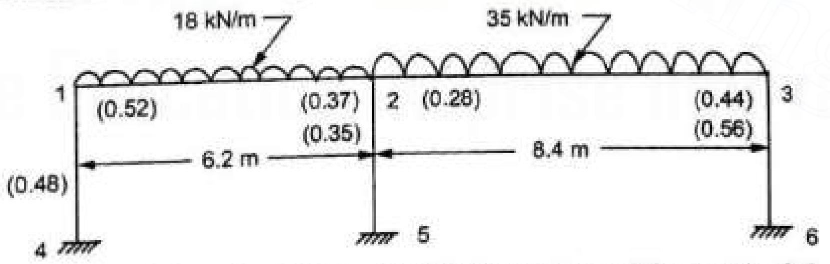


Fig. 8.13.2 Loading Case - 2 : Maximum Span Moment in 2-3

Moment Distribution :

Joint	1		2			3	
Member	1-4	1-2	2-1	2-5	2-3	3-2	3-6
DF	0.48	0.52	0.37	0.35	0.28	0.44	0.56
FEM	–	−57.66	57.66	–	−205.8	205.8	–
Balance	27.68	29.98	54.81	51.85	41.48	−90.55	−115.25
C.O	–	27.40	14.99	–	−45.28	20.74	–
Balance	−13.15	−14.25	11.21	10.60	8.48	−9.12	−11.61
C.O.	–	5.60	−7.12	–	−4.56	4.24	–
Balance	−2.69	−2.91	4.32	4.09	3.27	−1.87	−2.38
Final mmts	11.84	−11.84	135.87	66.54	−202.41	129.24	−129.24

Shear :

V_{1-2} = 18 x 6.2/2 − (135.87 − 11.84)/6.2 = 35.80 kN
V_{2-1} = 18 x 6.2/2 + (135.87 − 11.84)/6.2 = 75.80 kN
V_{2-3} = 35 x 8.4/2 + (202.41 − 129.24)/8.4 = 155.71 kN
V_{3-2} = 35 x 8.4/2 − (202.41 − 129.24)/8.4 = 138.29 kN

Maximum Span Moments :

Span 1-2 x_{max} = 35.8/18 = 1.99 *m*
M_{max} = 35.8 x 1.99/2 − 11.84 = 23.78 *kN.m*
Span 2-3 x_{max} = 138.29/35 = 3.951 *m*
M_{max} = 138.29 x 3.951/2 − 129.24 = 143.95 *kN.m*

Loading Case - III: *Maximum Support moment at joint 2*
In this case maximum load shall be on span 1-2 and 2-3 as shown in *Fig. 8.13.3*

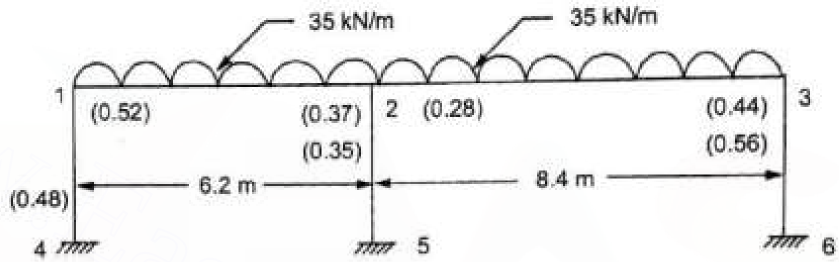


Fig. 8.13.3 Loading Case - III : Maximum Support Moment at joint - 5

Moment Distribution

Joint	1		2			3	
Member	1-4	1-2	2-1	2-5	2-3	3-2	3-6
DF	0.48	0.52	0.37	0.35	0.28	0.44	0.56
FEM		−112.12	112.12	−	−205.8	205.8	−
Balance	53.82	58.30	34.66	32.79	26.23	−90.55	−115.25
C.O.	−	17.33	29.15	−	−45.28	13.11	−
Balance	−8.32	−9.01	5.97	5.64	4.52	−5.77	−7.34
C.O.	−	2.98	−4.5	−	−2.88	2.26	−
Balance	−1.43	−1.55	2.73	2.58	2.07	−0.99	−1.27
Final mmts	44.07	−44.07	180.13	41.01	−221.14	123.86	−123.86

Shear :

V_{1-2} = 35 x 6.2/2 − (180.13 − 44.07)/6.2 = 86.55 kN
V_{2-1} = 35 x 6.2/2 + (180.13 − 44.07)/6.2 = 130.44 kN
V_{2-3} = 35 x 8.4/2 + (221.14 − 123.86)/8.4 = 158.58 kN
V_{3-2} = 35 x 8.4/2 − (221.14 − 123.86)/8.4 = 135.42 kN

Maximum Span moments :

Span 1-2 : x_{max} = 86.55/35 = 2.473 *m*
M_{max} = 86.55 x 2.473/2 − 44.07 = 62.95 *kN.m*
Span 2-3 : x_{max} = 135.42/35 = 3.869 *m*
M_{max} = 135.42 x 3.869/2 − 123.86 = 138.11 *kN.m*

8.13.1 Results of Top Storey Frame: Bending Moment in *kN.m*

Loading Case No.	Col C22	1-2	Span mmt	2-1	Col C13	2-3	Span mmt	3-2	Col C3
I II III	54.90 11.84 44.07	54.90 11.84 44.07	74.78 23.78 62.95	136.90 135.87 180.13	4.43 66.54 20.05	132.47 202.41 221.14	65.54 143.95 138.11	58.32 129.24 123.86	58.32 129.24 123.86
Max.B.M	54.90	54.90	74.78	180.13	66.54	221.14	143.95	129.24	129.24
Case No.	I	I	I	III	II	III	II	II	II

Results of Top Storey Frame: Shear in *kN*

Loading Case	Jt. 1	2		3
I II III	95.28 35.80 86.55	121.72 75.80 130.44	84.43 155.71 158.58	66.77 138.29 135.42
Maximum Shear	95.28	130.44	158.58	138.29
Max. Col. Load	95.28	289.02		138.29

8.14 BOTTOM STOREY FRAME

Since R.C.C. slab has been provided at the plinth level the loadings remain the same. The analysis of the bottom storey frame will be carried out.

The floor to floor height is 3.35

Height of plinth above *G.L.* = 1.2 *m*

Depth of foundation below *G.L* = 2.1 m

∴Height of floor frame from foundation level = 1.2 + 2.1 = 3.3 *m*

The bottom end of the column is assumed to be hinged while top end will be fixed. The values of *I/L* are shown in *Fig. 8.14.1.*

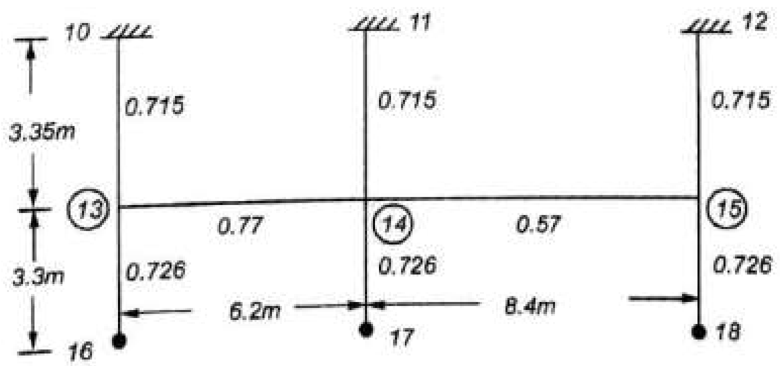


Fig. 8.14.1 Values of Moment of Inertia/Span *(I/L)*

8.14.1 Distribution factors :

Joint	*Member*	*RSF*	*SUM*	*D.F*
13	13-10 13-14 13-16	0.715 0.77 0.75 x 0.726	2.03	0.352 0.380 0.268
14	14-11 14-15 14-17 14-13	0.715 0.57 0.75 x 0.726 0.77	2.6	0.275 0.22 0.21 0.295
15	15-12 15-18 15-14	0.715 0.75 x 0.726 0.57	1.829	0.39 0.30 0.31

The fixed end moments will remain the same *(see Sect. 8.8.4)*

Loading Case-I : Maximum Span Moment in 13-14

For this, Maximum load shall be in 13-14 and minimum on 14-15 as shown in Fig.8.14.2

Moment Distribution :

Joint	13			14				15	Cols	
Member	13-10	13-16	13-14	14-13	14-11	14-17	14-15	15-14	15-12	15.18
DF	0.352	0.268	0.38	0.295	0.275	0.21	0.22	0.31	0.39	0.30
FEM	–		−124.93	124.93	–	–	−118.39	91.32	–	–
Balance	43.98	33.48	47.47	−1.93	−1.8	−1.37	−1.44	−28.31	−35.61	−27.4
C.O	–	–	−0.96	23.73	–	–	−14.2	−0.72	–	–
Balance	0.34	0.26	0.36	−2.81	−2.62	−2.0	−2.1	0.22	0.28	0.22
Final mmts	44.32	33.74	−78.06	143.92	−4.42	−3.37	−136.13	62.51	−35.33	−27.18

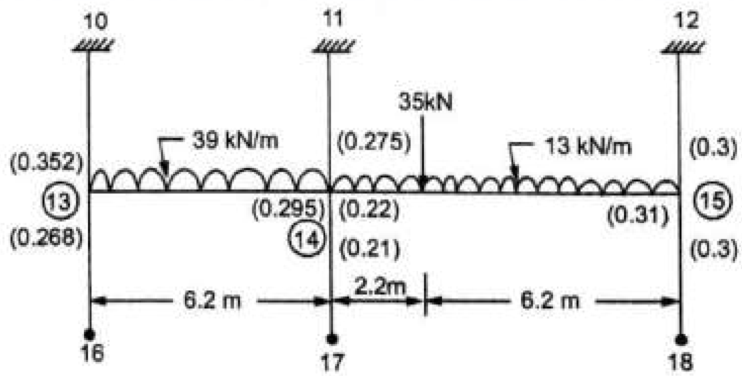


Fig. 8.14.2 Loading Case I : Maximum Span Moment in 13-14

Shear : V_{13-14} = 39 x6.2/2 – (143.92 – 78.06)/6.2 = 110.27 *kN*

V_{14-13} = 39 x 6.2/2 + (143.92 – 78.06)/6.2 = 131.52 *kN*

V_{14-15} = 13 x 8.4/2 + 35 x 6.2/8.4 + (136.13 – 62.51)/8.4 = 89.20 *kN*

V_{15-14} = 13 x 8.4/2 + 35 x 2.2/8.4 – (136.13 – 62.51)/8.4 = 55.0 *kN*

Maximum Span Moments :

Span 13-14 x_{max} = 110.27/39 = 2.827*m* from joint 13

M_{max} = 110.27 x 2.827/2 – 78.06 = 77.8 *kN.m*

Span 14-15 x_{max} = 55/13 = 4.23 *m* < 6.2 *m* from joint 15

M_{max} = 55 x 4.23/2 – 62.51 = 53.81 *kN.m*

Loading Case II : Maximum Span Moment in 14-15

For this there shall be maximum load on 14-15 and minimum on 13-14 as shown in *Fig. 8.14.3*

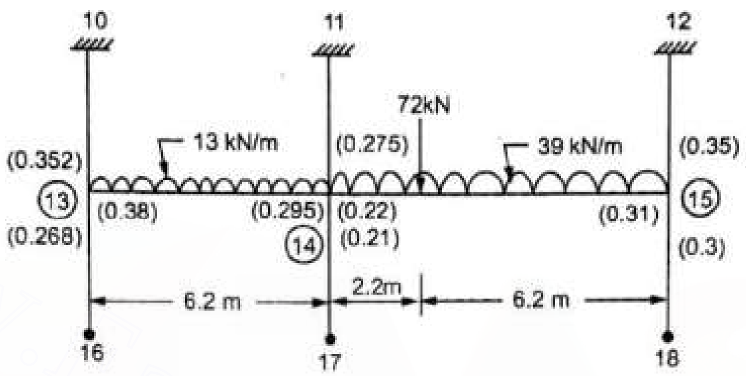


Fig. 8.14.3 Loading Case II : Maximum Span Moment in 14-15

Moment Distribution :

Joint	*13*			*14*				*15*		
Member	13-10	13-16	13-14	14-13	14-11	14-17	14-15	15-14	15-12	15.18
DF	0.352	0.268	0.38	0.295	0.275	0.21	0.22	0.31	0.39	0.30
FEM	–	–	–41.64	41.64	–	–	–315.61	259.94	–	–
Balance	14.66	11.16	15.82	80.82	75.34	57.53	60.27	–80.58	–101.37	–77.99
C.O	–	–	40.4	7.91	–	–	–40.29	30.13	–	–
Balance	–14.22	–10.83	–15.35	9.55	8.90	6.8	7.13	–9.34	–11.75	–9.04
C.O.	–	–	4.77	–7.67	–	–	–4.67	3.56	–	–
Balance	–1.69	–1.28	–1.81	3.64	3.39	2.59	2.72	–1.10	–1.39	–1.07
Final mmts	–1.25	–0.95	2.19	135.90	87.63	66.92	–290.45	202.61	–114.51	–88.1

Shear : V_{13-14} = 13 x6.2/2 – (135.90 + 2.19)/6.2 = 18.03 *kN*

V_{14-13} = 13 x 6.2/2 + (135.90 + 2.19/6.2 = 62.57 *kN*

V_{14-15} = 39 x 8.4/2 + 72 x 6.2/8.4 + (290.45 – 202.61)/8.4 = 227.4 *kN*

V_{15-14} = 39 x 8.4/2 + 72 x 2.2/8.4 – (290.45 – 202.61)/8.4 = 172.2 *kN*

Maximum Span Moment :

Span 13-14 x_{max} = 18.03/13 = 1.387 *m*

M_{max} = 18.03 x 1.387/2 + 2.19 = 14.69 *kN.m*

Span 14-15 x_{max} = 172.2/39 = 4.415 *m* from joint 15

M_{max} = 172.2 x 4.415/2 - 202.61 = 177.52 *kN.m*

Loading Case - III : Maximum Support Moment at joint 14

For this there shall be maximum load on span 13-14 and span 14-15

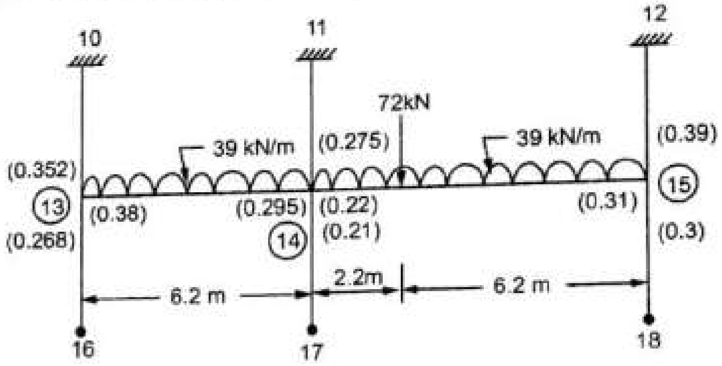


Fig. 8.14.4 Loading Case III : Maximum Support Moment at joint 14

Moment Distribution :

Joint	13			14				15		
Member	13-10	13-16	13-14	14-13	14-11	14-17	14-15	15-14	15-12	15.18
DF	0.352	0.268	0.38	0.295	0.275	0.21	0.22	0.31	0.39	0.30
FEM	–	–	–124.93	124.93	–	–	–315.61	259.94	–	–
Balance	43.98	33.48	47.47	56.25	52.44	40.04	41.95	–80.58	–101.37	–77.99
C.O	–	–	28.12	23.73	–	–	–40.29	20.97	–	–
Balance	–9.90	–7.54	–10.68	4.89	4.55	3.48	3.64	–6.50	–8.18	–6.29
C.O.	–	–	2.44	–5.34	–	–	–3.25	1.82	–	–
Balance	–0.86	–0.65	–0.93	2.53	2.36	1.80	1.89	–0.56	–0.71	–0.55
Final mmts	33.22	25.29	–58.51	206.99	59.35	45.32	–311.67	195.09	–110.26	–84.83

Shear :

V_{13-14} = 39 x 6.2/2 – (206.99 – 58.51)/6.2 = 96.95 *kN*

V_{14-13} = 39 x 6.2/2 + (206.99 – 58.51)/6.2 = 144.85 *kN*

V_{14-15} = 39 x 8.4/2 + 72 x 6.2/8.4 + (311.67 – 195.09)/8.4 = 230.82 *kN*

V_{15-14} = 39 x 8.4/2 + 72 x 2.2/8.4 – (311.67 – 195.09/8.4 = 168.78 *kN*

Maximum Span Moment :

Span 13-14 x_{max} = 96.95/39 = 2.486 *m*

M_{max} = 96.95 x 2.486/2 – 58.51 = 62.0 *kN.m* from joint 13

Span 14-15 x_{max} = 168.78/39 = 4.328 *m*

M_{max} = 168.78 x 4.328/2–195.09 = 170.15 *kN.m*

Results of Bottom Frame

Loading Case	*Joint* 13				*14*					*15*		
	13-10	13-16	13-14	Span mmt	14-13	14-11	14-17	14-15	Span mmt	15-14	15-12	15-18
I	44.32	33.74	78.06	77.8	143.92	4.42	3.37	136.13	53.81	62.51	35.33	27.18
II	1.25	0.95	2.19	14.69	135.90	87.63	66.92	290.45	177.52	202.61	114.51	88.10
III	33.22	25.29	58.29	62.0	206.99	59.35	45.32	311.67	170.15	195.09	110.26	84.83
Max.mmt kN.m	44.32	33.74	78.06	77.8	206.99	87.63	66.92	311.67	177.52	202.61	114.51	88.10

Maximum Shear :

Loading Case	Jt 4	5		6
I	110.27	131.52	89.20	55.00
II	18.03	62.57	229.40	172.20
III	96.95	144.85	230.82	168.78
Max. Shear kN	110.27	144.85	230.82	172.20
Column Load kN	110.27	375.67		172.20

8.15 RESULTS OF SUBSTITUTE FRAME-I

The results of top frame, floor frame and bottom frame are given in the following *Tables 8.15.*

Table 8.15.1 Results of Beam and Column Moments in *kN.m*

Frame Type	Col.22	Span *mmt*			Col.13	Span *mmt*			Col.3
Top storey	54.9	54.9	74.78	180.13	66.54	221.14	143.95	129.24	129.24
Middle storey	41.32	82.63	76.0	201.63	81.54	312.16	173.17	208.97	104.48
Bottom storey	44.32/33.74	78.06	77.8	206.99	59.35/66.92	311.67	177.52	201.61	114.51/88.1

Table 8.15.2 Results of Shear in *kN.*

Frame Type	Col.22	Col.13		Col.23
Top Storey	95.28	130.44	158.58	138.29
Column Load	95.28	289.02		138.29
Middle storey	111.24	143.34	230.06	172.64
Column Load	111.24	373.40		172.64
Bottom storey	110.27	144.85	230.82	172.20
Column Load	110.27	375.67		172.20

8.16 DESIGN OF BEAMS

8.16.1 Design of Middle Storey Transverse Beam 4-5-6

Design is pressented here only for floor beam in a compact tabular form.

Step	Description		Unit	Jt.4	Mid-span	5		Mid-span	6	Note
1.	Span	L	mm		6200			8400		
2.	End Condition :		mm	Fixed		Continuous			Fixed	
3.	Section :	b	mm		230			230		
	(Assumed)	D	mm		500			500		1
		d'	mm		60			60		
		d	mm		440			440		
		D_f	mm		110			110		
		b_s	mm		500			500		
4.	Load : UDL	w_u	kN/m		39			39		
	: PTL	P_u	kN		-			72		
	Dist. from left		m		-			2.2		
5.	$M_{u.max}$		kN.m	82.63	76.00	201.63	312.16	173.17	208.97	2
6.	$M_{ur.max}$		kN.m	122.9	122.9	122.9	122.9	122.9	122.9	3
	Section Type			SR	SR/(T)	DR	DR	T	DR	
		L_o	mm		4340			5880		
		b_f	mm		1613			1870		
	M_{u1} for $x_u = D_f$		kN.m		503			584		
		x_u	mm		$< D_f$			$< D_f$		
		b	mm	230	230/1613	230	230	1870	230	
	$R_u = M_u/bd^2$			1.85	1.70/(0.243)	4.53	7.00	0.478	4.7	
7.	Main steel :									
		p_t	%	0.584	0.53/(0.069)	1.52	2.32	0.136	1.58	5
		p_c	%	-	-	0.61	1.46	-	0.668	
	Required area Top			591	-	1538	2348	-	1599	
	Required area bottom			-	536/490	617	1477	1119	676	
	Top		N-ϕ	3#16	3#16	3#16+3#20	3#16+6#20	2#16	2#16+4#20	
	Bottom		N-ϕ	3#16	3#16	3#16	3#16+3#20	3#16+2#20	2#16+1#20	6
	Provided Area Top			603	603	1545	2487	402	1658	
	Provided Area Bottom			603	603	603	1545	1231	716	
8.	Shear :									
	$V_{u.max}$		kN	111.24		143.34	230.06		172.64	7
	Loading case		No.	I		III	III		II	
	N_1 - ϕ		mm	3#16		3#16+3#20	3#16+6#20		2#16+4#20	
	A_{st1}		mm^2	603		1545	2487		1658	
	p_t		%	0.59		1.527	2.457		1.641	
	V_{uc}		kN	51.49		73.19	82.8		74.56	8
	$V_{usv.min}$		kN	40.48		40.48	40.48		40.48	9
	$V_{ur.min}$		kN	91.97		113.67	123.28		115.04	
	V_{uD}		kN	89.59		121.70	208.41		150.99	10
	$V_{us} = V_{uD} - V_{uc}$		kN	-		48.51	125.61		76.43	11
	V_{us}/d			-		110	285		174	
	Des.Stir. N-ϕ		mm	-		#8@300	#8@120		#8@200	12
	L_{s1}		mm	-		760	2200		1480	13
	Dist. of P.I.	x_1	mm	-		1870	1560		1446	14
	Min. Stir. N-ϕ		mm	ϕ6@130		ϕ6@130	-		ϕ6@130	

Explanatory Notes and Calculations :

(1) For mild environment, nominal cover = 20*mm* *(see Table C-1)*
Assuming diameter of main steel of 20*mm* and diameter of stirrups 8*mm*,
Required effective cover for two rows of bar = 20 + 8 + 20 + 20/2 = 58*mm* < 60*mm* ∴o.k.

(2) $M_{ur.max}$ for rectangular section = $R_{u.max.} bd^2$.For M20-Fe415, $R_{u.max}$ = 2.76 N/mm^2 *(Table 4.1.1)*
Therfore, $M_{ur.max}$ = 2.76 bd^2 x 10^{-6} *kN.m* = 2.76 x 230 x 440^2 x 10^{-6} = 122.9 *kN.m*

(3) Since $M_{u.max}$ is far less than $M_{ur.max}$ of rectangular section in span 4-5, there is less advantage to some extent in designing the section as a flanged section. The excess is about 9%. The areas required for flanged section are given in brackets. But it should be noted that the mid-span moment (= 173.17 *kN.m*) in span 5-6 is greater than the moment of resistance of a singly reinforced section($M_{ur.max}$= 122.9 *kN.m*). ***Therefore, it must be designed as a flanged section only.***

(4) In order to determine whether the neutral axis lies in the flange M_{ul} is required to be calculated for $x_u = D_f$ using *Eq. 4.3.8.*

(5) Values of required p_t are obtained by using *Table F-4* for Singly reinforced rectangular section (*SR*) and by using *Table F-5* for Doubly reinforced rectangular (*DR*) section.
For flanged section, it is obtained using *Eq. 4.3.5.*

(6) Number Diameter (*N*-ϕ) combination is selected for required A_{st} using *Table H-2*.

(7) $V_{u.max}$ is taken directly from results of frame analysis *Table 8.15.2*

(8) τ_{uc} corresponding to available N_t-ϕ combination of tension steel at critical section is obtained from *Table.4.4.1 or Eq. 4.4.3b* , $V_{uc} = \tau_{uc} \times b \times d$

(9) Value of $V_{usv.min}$ = 0.4*bd* is obtained by using *Eq. 4.4.8.*

(10) V_{uD} is calculated at a distance (b_s/2+*d*) from the centre of support using *Eq. 4.4.1a* only when it is found that $V_{ur.min} < V_{u.max}$.
If $V_{ur.min} > V_{u.max}$ *or* $V_{ur.min} > V_{uD}$ only minimum stirrups are required *(see Table F-6)*

(11) $V_{us} = V_{uD} - V_{uc}$.

(12) Design of stirrups is done using *Table F-7.*

(13) $L_{sl} = (V_{u.max} - V_{ur.min})/w_u$ for beams carrying *UD* Loads. For span 5-6 from support 5, L_s , calculated is greater than 2.2*m* and hence limited to 2.2*m* where point load acts.
The first stirrup is provided at a distance of 50*mm* from the face of support.

(14a) *Distance of point of inflection from left support Jt - 5 (Span 5-6):*
In this case *Loading Case - III (Section 8.9)* is governing , $V_{5\text{-}6}$ = 230.06 *kN* ,
$M_{5\text{-}6}$ = 312.16 *kN.m* , w_u = 39 *kN/m and* Point load = 72 *kN* at 2.2*m* from left support.
Let *x* be the distance of point of inflection *(TPC)* from left support (*i.e.*joint 5)
$230.06x - 39x^2/2 - 312.16 = 0$ ∴ $x^2 - 11.8x + 16 = 0$ ∴ $x = 1.56m$
Actual point of cut off = 1.56+ greater of (*d or* 12ϕ *or* clear span/16) = 1.56 + 0.5 say 2.2*m*
Shear at point of inflection = V_1 = 230.06 − 39 x 1.56 = 169.2 *kN* > $V_{ur.min}$ of top bars
∴ Design stirrups will be required only upto the point of inflection.*(PI)*
Design of stirrups will have to be done afresh beyond point of inflection *(TPC)*taking bottom steel as tension steel.
At *PI*, for bottom steel, A_{st} = 3#16 + 3#20 = 1545 mm^2, p_t = 1.526%, τ_{uc} = 0.723 N/mm^2 *(Table 4.4.1)*

$$V_{uc} = 0.723 \times 230 \times 440 \times 10^{-3} = 73.2\ kN$$
$$V_{ur.min} = 73.2 + 0.4 \times 230 \times 0.44 = 113.68\ kN < 169.2\ kN$$
At *TPC*
$$V_{us} = 169.2 - 73.2 = 96\ kN$$
$$V_{us}/d = 96/0.44 = 240\ kN/m$$

Required stirrups will be #8@150*mm* < 120 *mm* (obtained for 3#16 + 6#20) *(Table F-7)*
∴ Provide spacing of #8*mm* 2-legged stirrups at 120*mm* upto the point load.

Explanatory Notes and Calculations Continued.....

V_u just to the left of point load = 230.06 −39 x 2.2 = 144.26 *kN*

V_u just to the right of point load = 144.26 − 72 = 72.26 *kN* < $V_{ur.min}$ (=113.68*kN*)

∴ Minimum stirrups are adequate beyond the point load. *(Table F-6)*

Provide φ6*mm* 2-legged stirrups at 130*mm* c/c

(14b) *Distance of point of inflection (PI) from right support (i.e. Jt. 6)*

In this case loading *Case - II* is governing (See *Sect. 8.9*)

$V_{6\text{-}5}$ = 172.64 *kN.* , $M_{6\text{-}5}$ = 208.97 *kN.m* , w_u = 39 kN/m

x_{max} = $V_{6\text{-}5}/w_u$ = 172.64/39 = 4.427*m*

Distance of *TPC* from right support,

$x_2 = 4.427 - \sqrt{4.427^2 - 2 \times 208.97/39}$ *(Eq. 2.6.4)*

= 1.446 *m.*

Actual point of curtailment = 1.446 + 0.5 = say 2*m* from joint 6

At *TPC* , V_u = 172.64 − 39 x 1.446 = 116.2 *kN*

(14c) *Span 4-5*

Distance of point of inflection (*TPC*) from joint 5 (see loading *Case - III*)

$V_{5\text{-}4}$ = 143.34 *kN* , $M_{5\text{-}4}$ = 201.63 *kN.m* , w_u = 39 *kN.m*

Let *x* be the distance of *TPC* from right support joint - 6

$143.34x - 39\,x^2/2 - 201.63 = 0$

∴ *x* = 1.87 *m*

Actual point of curtailment *(APC)* = 1.87 + 0.44 = say 2.3 *m*

Shear at *TPC* = 143.34 − 39 x 1.87 = 70.41 *kN* < 113.67 *kN* (= $V_{ur.min}$)

∴ Provide Minimum shear reinforcement of φ6 *mm* @ 130 *mm c/c*

The details of reinforcement are shown in *Fig. 8.18*

8.16.2 Design of Middle Storey Longitudinal Beams

Step No.	Details			Floor Beams Fig. 8.7.1a) External	Internal	Reference
1.	*Span*	*L*	*m*	3.05	3.05	External : B22,B23,B212,B222
2.	*End Condition No.*			3	3	Internal : B122, B132, B126,B136
3.	*Section assumed :* Width	*b*	*mm*	230	230	
	Depth	*D*	*mm*	450	300	
	Depth of slab	D_f	*mm*	110	110	
	Effective cover	*d'*	*mm*	31	31	
	Effective depth	*d*	*mm*	419	269	
4.	*Loads*	w_u	*kN/m*	30.60	23.37	*Sect. 8.8.2*
5.	$M_{u.max} = w_u L^2/12$		*kN.m*	23.72	18.12	
6.	*Main Steel :*					
	(a) At Midspan					
	Bottom Bent	*N-φ*	*mm*	1-#10	1-#10	
	Bottom Straight	*N-φ*	*mm*	2-#10	2-#10	
	Provided	A_{st}	mm^2	236	236	
	Provided	*d*	*mm*	419	269	
	Provided	M_{ur}	*kN.m*	33.82	21.07	*Table F-1*

Design of Middle Storey Longitudinal Beams Continued ...

Step No.	Details			Floor Beams External	Floor Beams Internal	Reference
	(b) At support Top	N-ϕ	*mm*	3-#10	3-#10	
	Bottom	N-ϕ	*mm*	2-#10*	2-#10*	
	Provided	A_{st}	mm^2	236	236	
	Provided	d	*mm*	419	269	
	Provided	M_{ur}	*kN.m*	33.82	21.07	*Table F-1*
7.	Shear :	$V_{u.max} = w_u\ L/2$	*kN*	46.67	35.64	
		A_{stl} N-ϕ	*mm*	3-#10	3-#10	
		V_{uc}	*kN*	34.0	26.2	*Table 4.4.1*
		$V_{usv.min} = 0.4\ bd$	*kN*	38.55	24.8	
		$V_{ur.min}$	*kN*	72.55	51.0	
	Stirrups - Minimum ϕ-s		*mm*	ϕ6-130	ϕ6-130	*Table F-6*
8.	Overlapping triangular load			$1.5(1.91 + 2.03)\frac{3.05}{2}$ = 9.02	$1.5(3.82 + 4.06)\frac{3.05}{2}$ = 18.03	*Sect. 8.8.2(1)*
	End Shear for column load excluding triangular slab load		*kN*	2(46.67 − 9.02) = 75.3	2(35.64 − 18.03) = 35.22	
9.	*Total load on column* due to end shear from two beams		*kN*	***75.30***	***35.22***	

Note : *(1)* At support, in addition to 2-#10mm, 1-#10 is obtained by single bar bend up from left span and again going to bottom in right span.*

(2) For external beam, though provided M_{ur} appears to be large compared to moment acting on it, reduction in A_{st} is not possible due to requirement of minimum steel and depth is decided from practical consideration of avoiding seperate lintel.

8.16.3 Roof Beams

The design of roof beams is left to the reader. It can be done on the same lines as floor beams. However, the end shears from longitudinal and transverse roof beams are given in *Sect. 8.8.1 (1)* and *Sect 8.8.1(2)* respectively as the same are required for design of columns.

8.17 DESIGN OF COLUMNS

Column in Frame	*Left*	*Middle*	*Right*	
Column Mark	C22	C13	C3	
Assumed section *mm* x *mm*	230 x 500	230 x 500	230 x 500	
Floor to floor height *m*	3.35	3.35	3.35	
Category of Column	*II*	*I*	*II*	*(Sect.5.4.3)*

8.17.1 Calculation of Column Loads in different storeys : Exact Method

Column between		***C22***	***C13***	***C3***	
(a) Roof and 3rd Floor (R-3) :					
Shear from Roof level longitudinal beam		27.92	5.04	27.92	*Sect. 8.8.1(1)*
Max. shear from Transverse beam	*kN*	95.28	289.02	138.29	*Table 8.15.2*
Factored Self weight = 1.5x25x0.23x0.5x3.35=14.4	*kN*	14.4	14.4	14.4	
Shear from floor Longitudinal beams	*kN*	75.30	35.22	75.3	*Sect.8.16.2(9)*
Total	*kN*	212.9	343.68	255.91	
Roof Load : P_r say	***kN***	**213**	**344**	**256**	
Ratio of *LL*/Total Load from transverse beam at roof level		0.2	0.2	0.2	*Sect 8.8.1(2)* This will be required later for column load in lower storey
Beam shear due to *LL* = above ratio x transverse beam shear = P_{rL}	***kN***	**19.00**	**58.0**	**27.6**	
Load from above	*kN*	Nil	Nil	Nil	
Total Gross load = $P_{r3.gr}$	*kN*	*213*	*344*	*256*	
Reduction in Load due to reduction in *LL* on upper floors	*kN*	Nil	Nil	Nil	
Net Design Load at III floor $=P_{r3.net}$	*kN*	213	344	256	
(b) 3rd Floor and 2nd Floor (3-2)					
Max. shear from Transverse floor beam	*kN*	111.24	373.40	172.64	*Table8.15.2*
Self weight	*kN*	14.4	14.4	14.4	
Shear from Longitudinal floor beams	*kN*	75.30	35.22	75.30	*Sect.8.16.2*
Total load at each mid-floor	*kN*	201	423	263	
Floor Load : P_f	***kN***	**201**	**423**	**263**	
Total gross load = $(P_r+P_f)=P_{32.gr}$	*kN*	414	767	519	
Ratio of *LL*/Total Load for transverse beam at each floor level		0.47	0.47	0.47	*Sect 8.8.2 (2)*
Beam shear due to LL at each floor level = above ratio x shear from transverse floor beam = P_{fL}	*kN*	52.3	175.5	81.1	
Total shear due to *LL* on upper floors $P_{r3L} = P_{rL} + 1xP_{fL}$	*kN*	71.3	233.5	108.7	
Reduction factor = r_3		0.1	0.1	0.1	
Actual load reduction=R_d= $r_3 x P_{r3L}$	*kN* say	–7	–23	–11	
Net Design Load at II floor top $= P_{32.net} = P_{32.gr} - R_d$	*kN*	*407*	*744*	*508*	

Calculation of Column Loads in Different Storey -Exact Method Continued....

		C22	C13	C3	
(c) 2nd Floor to 1st Floor (2-1)					
2nd floor load $P_2 = P_f$	kN	201	423	263	
Total gross load $P_{21.gr} = P_r + 2P_f$	kN	615	1190	782	
Total shear due to LL on upper floors $P_{r2L} = P_{rL} + 2xP_{fL}$	kN	123.6	409	189.8	
Reduction factor $= r_2$		0.2	0.2	0.2	
Actual load reduction$=R_d = r_2 x P_{r2L}$ in LL due to upper floor	kN say	–24	–81	–38	
Net Design Load at I floor top $= P_{21.net} = P_{21.gr} - R_d$	kN	***591***	***1109***	***744***	
(d) 1st Floor and Plinth (1-P_p)					
Shear from transverse beam		111.24		172.64	
Self weight		14.4		14.4	
Floor load $P = P_{fL1}$	kN	125.64	423	187.04	
Total gross load $P_{1P.gr} = P_r + 2P_{fL} + P_{fL1}$	kN	740	1613	969	
Total shear due to LL on upper floors $P_{rpL} = P_{rL} + 3P_{fL}$	kN	176.0	584.5	270.9	
Reduction factor r_3		0.3	0.3	0.3	
Actual load reduction$=R_d = r_3 x P_{rpL}$ in LL due to upper floors	kN say	–53	–175	–81	
Net Design Load at Plinth top $= P_{1PL.net} = P_{1p.gr} - R_d$	kN	***687***	***1438***	***888***	
(e) Plinth to Footing (P_p to F_t)					
Load from above		740	1613	969	
Shear from Transverse beam		110.27	375.67	172.2	*Sect. 8.14.1*
Shear from Longitudinal beam [1.5 (20 x 0.25 x (3.35 + 1.2) x 3.05]		104.1	–	104.1	
Self weight		14.4	14.4	14.4	
Floor Load P_{fl}		228.77	390.07	290.7	
Total gross Load $= P_{rL} + 2P_{fL} + P_{fL1} + P_{fl}$		969	2003	1259	
Total LL shear due to upper floor$=P_{rfL} + 4P_{fL}$		228	760	352	
Reduction factor $= r_4$		0.4	0.4	0.4	
Actual reduction $= R_d = r_4 \times P_{rfL}$		–91	–304	–141	
Net Design Load at Foundation $P_{pft.net}$		***878***	***1699***	***1118***	

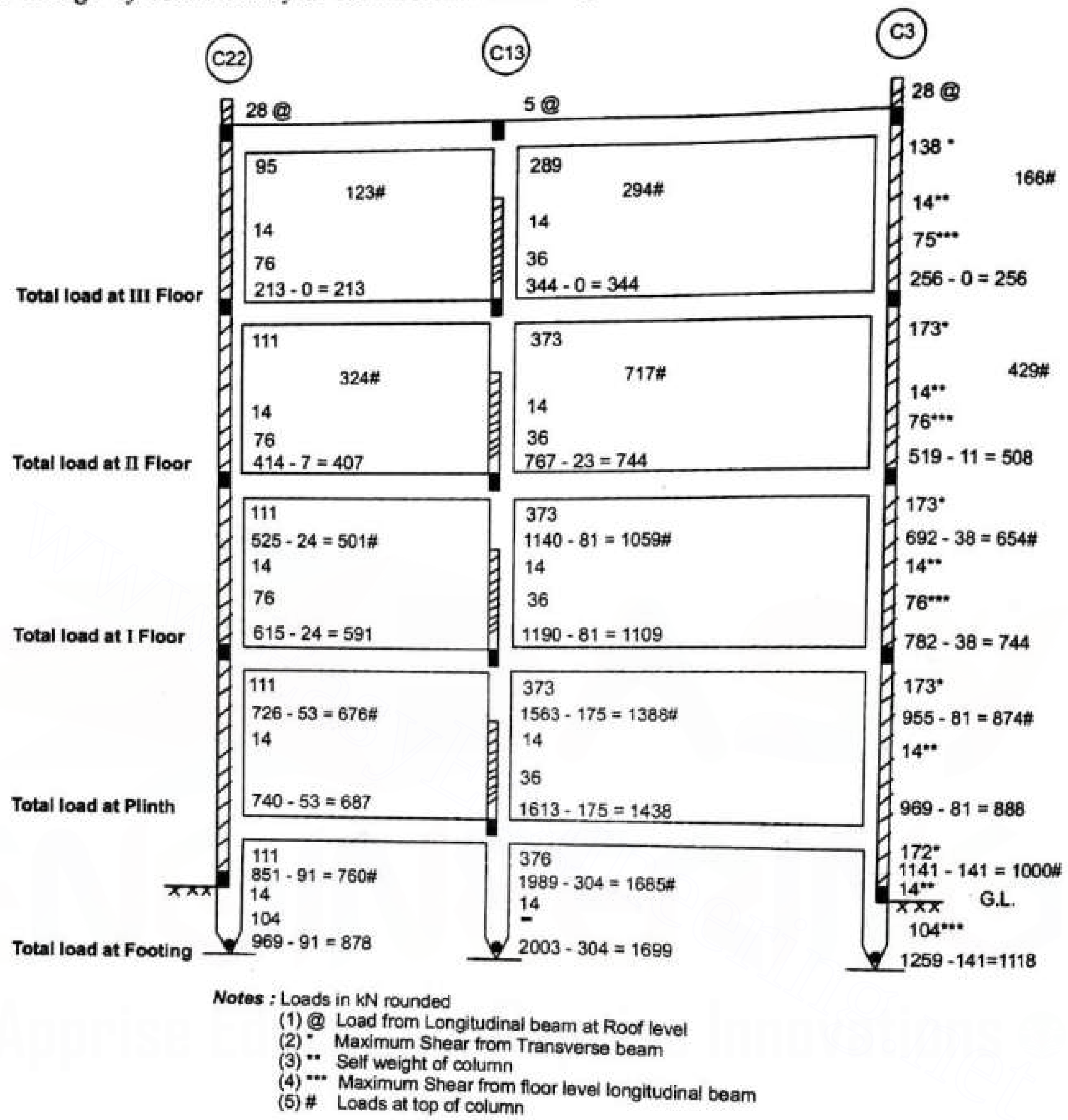


Fig. 8.17.1 Load Flow Diagram - *Exact Method*

8.17.2 Approximate Method for Calculation of Column Loads.

The detailed design calculations of column load *(Sect. 8.17.2)* at the footing level to obtain the size of footing take considerable time. But when the work is to be started urgently the details of the footing along with the size of the column are required in a short time to give the line out. In such a case approximate method of calculation of load is used *(see Sect. 5.4.2b)*

In this method the following approximations are made :

(1) If the wall is to be constructed between the floor, the self weight of the beam is not calculated seperately but the full height of the wall (*i.e* floor to floor height) is taken ignoring the difference between the unit weight of concrete of beam and unit weight of mansonry.

(2) Similarly the length of the wall is taken between centre to centre distance between the columns and the self weight of column is not calculated separetly.

(3) Since the aim of computation is simplicity and quick assessment of load, the deduction in live load due to upper floors and continuity effect are ignored.

The procedure for calculation of loads is as under :

(1) *Assessment of unit weight of slab.*

Initially determine the thickness of the slab (*D*) as per the procedure given earlier. In this case the total depth of the slab is 110*mm*

Roof : Intensity of *LL* = 1.5 kN/m^2 and *FF* = 2.5 kN/m^2

Intensity of ultimate load w_{uR} = *LF* (25*D* + *LL* + *FF*)
= 1.5 (25 x 0.11 + 1.5 + 2.5) = 10.13 kN/m^2

Floor : Intensity of *LL* = 4 kN/m^2 and *FF* = 1.0 kN/m^2

Intensity of ultimate load w_{uf} = 1.5(25 x 0.11 + 4 + 1) = 11.63 kN/m^2

Wall : Unit weight of mansonry = 20 kN/m^3 and unit weight of concrete = 25 kN/m^3

Weight of Parapet wall 250*mm* thick and1.45*m* high upto the centre of transverse beam

Ultimate load/*m* w_{up} = 1.5x 20 x 0.25 x 1.45 = 11*kN/m*

Weight of External wall 3.35*m* high and 250*mm* thick,

Ultimate load/*m* w_{ue} = 1.5(20 x 0.25 x 3.35) = 25.13 *kN/m*

Weight of Internal wall 150*mm* thick and 2.5*m* high (= 2.2 + 0.3 depth of longitudinal beam)

Ultiamte load/*m* w_{ui} = 1.5(20 x 0.15 x 2.5) = 11.3 *kN/m*

Beam : (a) Since there is no wall below the transverse beam of size 250*mm* x 500*mm*

Self weight of transverse beam w_{ut} = 1.5 (25 x 0.25 x 0.5) = 4.7 *kN/m*

Plinth : External wall 250*mm* thick and 4.55*m*/high (=3.35 + 1.2)

Ultimate load/*m* = 1.5 x 20 x 0.25 x 4.55 = 34.13 *kN/m*

(2) *Assessment of loads on Columns in Different storeys :*

The column load area (A_{col}) is shown in *Fig. 8.17.2*

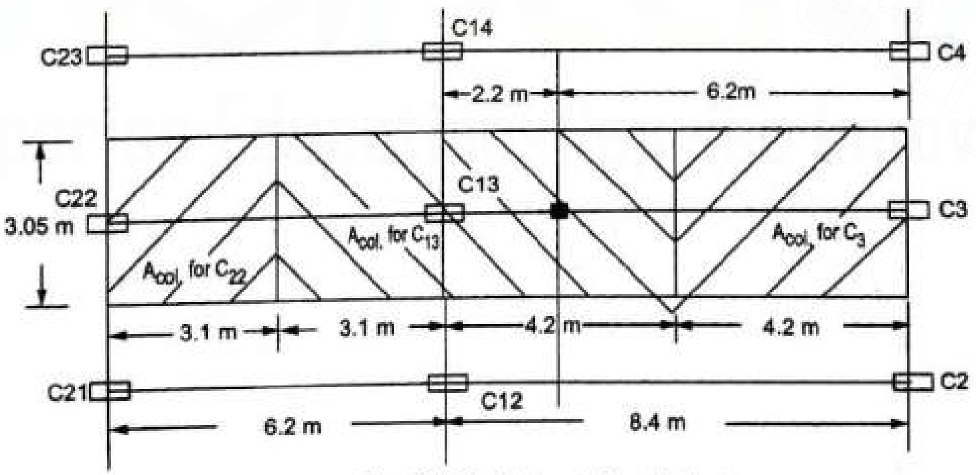


Fig. 8.17.2 Column Load Area

Column Load Area :

Floor area transferring load to column C_{22} = A_{col} for C_{22} = 3.05 x 3.1 = 9.46m^2

Floor area transferring load to column C_{13} = A_{col} for C_{13} = $A_{col.13}$ = 3.05 x (3.1+ 4.2)= 22.27m^2

Floor area transferring load to column C_3 = $A_{col.}$ for C_3 = 3.05 x 4.2 = 12.81m^2

External wall load 250*mm* thick and 3.35*m* high = 1.5 x 20 x 0.25 x 3.35 = 25.13 *kN/m*

Internal wall load 150*mm* thick and 2.2*m* high = 1.5 x 20 x 0.15 x 2.2 = 9.9 *kN/m*

Assessment of Column Loads in different storeys - Approximate Method

	Column Mark	C_{22}	C_{13}	C_3
1.	*Column load Area* m^2	9.46	22.27	12.81
	Intensity of ultimate load of roof slab w_{ur} kN/m^2	10.13	10.13	10.13
	Intensity of ultimate load of floor slab w_{uf} kN/m^2	11.63	11.63	11.63
	Wall length L_w m	3.05	3.05 + 3.05 x 6.2/8.4 =5.3	3.05 + 3.05 x 2.2/8.4 = 3.85
	Transverse beam length L_{tb} m	6.2/2 =3.1	(6.2 + 8.4)/2 =7.3	8.4/2 = 4.2
2.	*Load on Column in kN in each storey:*			
	(a) Roof and 3rd floor (R-3)			
	Parapet wall	11x 3.05 =34	-	11 x 3.05 = 34
	Roof slab = w_{uR} x A_{col} kN	10.13x9.46=96	10.13 x 22.27 =226	10.13 x 12.81 = 130
	Roof Level Transverse beam = w_{ut} x L_{tb}	4.7 x 3.1=15	4.7 x 7.3 =34	4.7 x 4.2 = 20
	Wall between 3rd floor and roof beam kN	25.13x3.05=77	11.3 x 3.05 + 11.3x3.05x6.2/8.4 =60	25.13 x 3.05 + 11.3x3.05x2.2/8.4= 86
	Floor level transverse beam	15	34	20
	Total load $P_{r3.gr}$	**237**	**354**	**290**
	(b)3rd floor and 2nd floor(3-2)			
	Floor slab = w_{uf} x A_{col}	11.63 x 9.46=110	11.63 x 22.27 =259	11.63 x 12.81 = 149
	Wall between 3rd floor and 2nd floor	25.13x3.05=77	11.3 x 3.05 +11.3x3.05x6.2/8.4=60	25.13 x 3.05 +11.3x3.05x2.2/8.4=86
	Transverse beam w_{ut} x L_{tb}	15	34	20
	Total Floor load $P_{32.gr}$	**202**	**353**	**255**
	Total design Load = $P_{r3.gr}$+$P_{32.gr}$	**439**	**707**	**545**
	(c) 2nd floor and 1st floor (2-1)			
	Total Floor load $P_{21.gr}$ = $P_{32.gr}$	202	353	255
	Total Design load = $P_{r3.gr}$ + 2 x $P_{32.gr}$	641	1060	800
	(d) 1st floor and Plinth (1-p)			
	Total Floor load $P_{1p.gr}$=$P_{32.gr}$	202	353	255
	Total Design load = $P_{r2.gr}$ + 3 x $P_{32.gr}$	**843**	**1413**	**1055**
	(e) Column between Plinth and footing			
	Floor load	202	353	255
	Extra load of external wall height of 1.2 m (bet. G.L. and Plinth) = 1.5 x 20 x 0.25 x 1.2 x 3.05	27	-	27
	Self wt of column = 1.5 x 25 x 0.23 x 0.5 x 3.3 = 14	14	14	14
	Total floor load	243	367	296
	Total load at footing	**1086**	**1780**	**1388**

Assessment of Column Loads in different storeys - Approximate Method continued.....

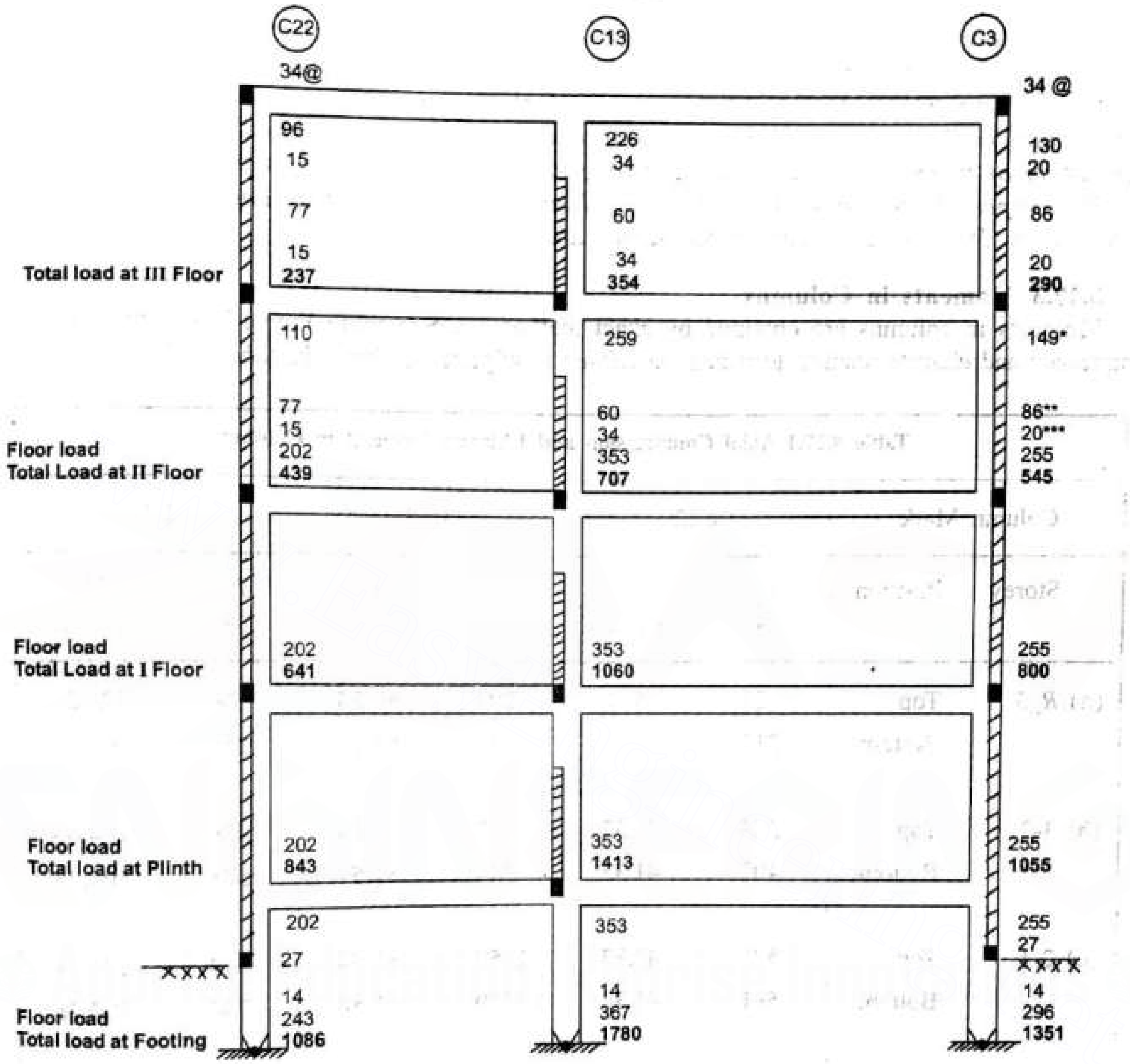


Fig. 8.17.3 Load Flow Diagram - *Approximate Method*

Comparision of Loads :

In approximate method the deduction for live load has not been made. If these loads are compared with those of exact method, without deduction of live load, the details are as under :

		C22	*C13*	*C3*
Total load at footing exact method	*kN*	969	2003	1259
Total load at footing approximate method	*kN*	1086	1780	1350
Difference	*kN*	–117	+223	–91

Load to be taken for design of footing = approximate values x 1.1 *kN* 1195 1958 1485

The values obtained by approximate method are even more than the those obtained by exact method except at interior column, therefore to err on the safe side they are increased by 10%.

Remarks :

In approximate method, for simplicity, the deduction of live load has not been made. If these values are compared with the corresponding values with exact method (as shown above) it will be observed that the values obtained for end columns are more while they are less for internal column (C13). This is obvious because the continuity effect has not been taken in approximate method to reduce the computational efforts. The other reason is there has been duplication of triangular load in interior longitudinal beam which acts as point load on span 5-6. To compensate for these differences the design loads obtained by approximate method are increased by 10%, so as to err on the safer side.

8.17.3 Moments in Columns

Moments in columns are obtained by exact method *i.e*, Substitute Frame-I. The results of axial compression and ultimate moment to which the column is subjected is shown in *Table 8.17.1* and *Fig. 8.17.4*

Table 8.17.1 Axial Compression and Ultimate Moment in Column

Column Mark		C22		C13		C3	
Storey	Position	P_u kN	M_u kN.m	P_u kN	M_u kN.m	P_u kN	M_u kN.m
(a) R_f-3	Top	123	54.90	294	66.54	166	129.24
	Bottom	213	41.32	344	81.54	256	104.48
(b) 3-2	Top	324	41.32	717	81.54	429	104.48
	Bottom	407	41.32	744	81.54	508	104.48
(c) 2-1	Top	501	41.32	1059	81.54	654	104.48
	Bottom	591	41.32	1109	81.54	744	104.48
(d) 1-P_1	Top	676	41.32	1388	81.54	874	104.48
	Bottom	687	44.32	1438	87.63	888	114.51
(e) P_1-F_t	Top	760	33.74	1685	66.92	1000	88.10
	Bottom	878	0	1699	0	1118	0

8.17.4 Determination of Effective Length and Slenderness of Column :

(A) Exact Method

I - Bending about x-axis i.e. Plane of Transverse Frame

Since all columns and beams are of the same cross section for all storeys, only the lengths being different, the Moments of Inertia of common sections and stiffnesses will be calculated first.

Determination of L_{eff} and slenderness of column - Bending about x-axis

Description	Columns	Beam 4-5	Beam 5-6
Section bxD mm	230x500	230x500 Flanged	230x500 Flanged
Moment of Inertia $I = bxD^3/12$ mm^4	$230x500^3/12$ $=2396 \times 10^6$	$2 \times 230 \times 500^3/12$ $= 4792 \times 10^6$	$2x230x500^3/12$ $= 4792 \times 10^6$
Length above 1st floor L mm	3350	6200	8400
Below Plinth mm	3300	——	3300
Sitffness above Plinth $k= I/L$ mm^3	715×10^3	773×10^3	570×10^3
Below 1st floor	726×10^3	———	———
Roof Level	715×10^3	Jt-4 : 773×10^3	
3rd and 2nd Floor.level	1430×10^3	Jt-5 : 1343×10^3	
1st Floor level Jt. Σk mm^2	1441×10^3	Jt-6 : 570×10^3	

Calculation of Rotation Release Factors β : Bending about x - Axis

Level	C22	C13	C3
Roof ΣK_c $x10^3$ mm^3	715	715	715
ΣK_b $x 10^3$ mm^3	773	1343	570
$\Sigma K_c+\Sigma K_b/2$ mm^3	715+773/2 = 1101	715+1343/2 = 1386	715+570/2 =1000
	715/1101 = **0.65**	715/1386 = **0.52**	715/1000 = **0.71**
3rd Floor ΣK_c $x10^3$ mm^3	1430	1430	1430
ΣK_b $x 10^3$ mm^3	773	1343	570
$\Sigma K_c+\Sigma K_b/2$ mm^3	1430+773/2= 1817	1430+1343/2= 2102	1430+570/2= 1715
$\beta = \Sigma K_c/(\Sigma K_c+\Sigma K_b/2)$	1430/1817 = **0.79**	1430/2102 = **0.68**	1430/1715 = **0.83**
2nd Floor Same as 3rd Floor	**0.79**	**0.68**	**0.83**
1st Floor Same as 2nd Floor	**0.79**	**0.68**	**0.83**
Plinth and Footing $\Sigma K_c x10^3$ mm^3	1441	1441	1441
ΣK_b $x 10^3$ mm^3	773	1343	570
$\Sigma K_c+\Sigma K_b/2$ mm^3	1827	2112	1726
$\beta = \Sigma K_c/(\Sigma K_c+\Sigma K_b/2)$	**0.79**	**0.68**	**0.83**
Footing Hinged End	**1.00**	**1.00**	**1.00**

Calculation of Effective Length and Slenderness : Bending about x-Axis

Storey	End	Level		C22	C13	C3
R_f - 3	Top	Roof	β_1	0.65	0.52	0.71
	Bottom	3rd Floor	β_2	0.79	0.68	0.83
			$\alpha = L_{effx}/L$	0.81	0.74	0.84
			L_{cc} mm	3350	3350	3350
			D mm	500	500	500
			$L = L_{cc}-D$ mm	2850	2850	2850
			$L_{eff} = \alpha \times L$ mm	2308	2109	2394
			L_{eff}/D	4.62	4.22	4.79
			Column Type	Short	Short	Short

Note : For obtaining the value of $\alpha = L_{effx}/L$ corresponding to β_1 and β_2 see *Fig. 4.8.1*

Calculation of Effective Length and Slenderness : Bending about x-Axis continued ....

Storey	End	Level		C22	C13	C3
3-2	Top	3rd Floor 2nd Floor	β1 β2 α = L_{effx} / L L_{cc} mm D mm L = L_{cc} −D mm L_{effx} = α x L L_{effx} / D Column Type	0.79 0.79 0.84 3350 500 2850 2394 4.79 Short	0.68 0.68 0.77 3350 500 2850 2194 4.39 Short	0.83 0.83 0.87 3350 500 2850 2480 4.96 Short
2-1	Top Bottom	2nd Floor 1st Floor	L_{eff} /D Column Type	4.79 Short	4.39 Short	4.96 Short
P -P_L	Top Bottom	1st Floor Plinth	L_{eff} /D Column Type	4.79 Short	4.39 Short	4.96 Short
P_L-F_t	Top Bottom	Plinth Footing	β1 β2(Hinged) $L_{eff.x}$/L L_{cc} D mm L = L_{cc}-D mm $L_{eff.x}$ mm $L_{eff.x}$/D Column Type	0.79 1 0.91 3300 500 2850 2593 5.2 Short	0.68 1 0.87 3300 500 2850 2479 4.9 Short	0.83 1 0.93 3300 500 2850 2650 5.3 Short

II - Bending about y-axis : Plane of Longitudinal Beam

Calculation of Stiffnesses :

Level	Member		Column Upper	Column Lower	Beam External	Beam Internal
Roof	Section bxD I = $bD^3/12(x10^6)mm^4$ Length = L_{cc} $K=I/L_{cc}(x10^3)$ ΣK ($x10^3$)	mm mm mm^3 mm^3	--- --- --- --- ---	500x230 500x230³/12 =507 3350 151.3 151.3	230x300 230x300³/12 =517.5 3050 169.7 339.4	230x300 230x300³/12 =517.5 3050 169.7 339.4
3rd,2nd 1st and P_L Floors	Section bxD $I=bD^3/12(x10^6)$ Length = L_{cc} $K=I/L_{cc}$ ($x10^3$) ΣK ($x10^3$)	mm mm^4 mm mm^3 mm^3	500x230 507 3350 151.3 151.3	500x230 507 3350 151.3 302.6	230x450 1746.6 3050 572.6 1145.2	230x300 517.5 3050 169.7 339.4
Plinth	Section bxD $I=bD^3/12(x10^6)$ Length = L_{cc} $K=I/L_{cc}$ ($x10^3$) ΣK ($x10^3$)	mm mm^4 mm mm^3 mm^3	500x230 507 3300 153.6 307.2	500x230 507 3300 153.6 307.2	230x450 1746.5 3050 572.6 1145.2	230x300 517.5 3050 169.7 339.4

Calculation of Rotation Release factors β : Bending about y-Axis :

(All values of K are actual $K \times 10^{-3}$ mm^3)

Level	External C22 and C3 ΣK_c	ΣK_b	$(\Sigma K_c+\Sigma K_b/2)$	β	Internal C13 ΣK_c	ΣK_b	$(\Sigma K_c+\Sigma K_B/2)$	β
Roof	151.3	339.4	321.0	0.47	151.3	339.4	321.0	0.47
3rd Fl	302.6	1145.2	875.2	0.34	302.6	339.4	472.3	0.64
2nd Fl	302.6	1145.2	875.2	0.34	302.6	339.4	472.3	0.64
1st	302.6	1145.2	875.2	0.34	302.6	339.4	472.3	0.64
Plinth	307.2	1145.2	879.8	0.35	307.2	339.4	476.9	0.64

Calculation of Effective Length and Slenderness : Bending about y-Axis :

Width of all columns for full height = b = 230mm

Storey	End	Level		C22 and C3	C13
Rf-3	Top	Roof	β_1	0.47	0.47
	Bottom	3rd Floor	β_2	0.34	0.64
			$L_{eff.y}/L$	0.64	0.72
			L_{cc} mm	3350	3350
			D of beam mm	300	300
			$L = L_{cc} - D$ mm	3050	3050
			$L_{eff.y}$ mm	1952	2196
			$L_{eff.y}/b$	8.5< 12	9.5< 12
			Column Type	Short	Short
3-2	Top	3nd Floor	β_1	0.34	0.64
	Bottom	2nd Floor	β_2	0.34	0.64
			$L_{eff.y}/L$	0.61	0.76
			L_{cc} mm	3350	3350
			D of beam mm	450	300
			$L = L_{cc} - D$ mm	2900	3050
			L_{effy} mm	1769	2318
			$L_{eff.y}/b$	7.7< 12	10.1< 12
			Column Type	Short	Short
2-1		2nd Floor	L_{effy}	1769	2318
		1st Floor	L_{effy}/b	7.7 < 12	10.1 <12
			Column type	short	short
1-P_L		1st Floor	L_{effy}/b	7.7<12	10.1<12
		Plinth	Column type	short	short
P_L-F_t	Top	Plinth	β_1	0.35	0.64
		Bottom	β_2	1	1
			L_{effy}/b	0.77	0.86
			L_{cc}	2100	3300
			D of beam	450	300
			$L = L_{cc} - D$	1650	3000
			L_{effy}	1271	2580
			L_{effy}/b	5.5<12	11.2<12
			Column type	Short	Short

Determination of Effective Length and Slenderness of Column

(B) Approximate Method

The computation of effective length of column is too much involved it will become much more difficult when the column section changes. Since in the framed structure full rotational fixity at the joint cannot occur, for practical design the partial fixity at top and bottom for the braced column may be assumed. In such a case the effective length factor will be between 0.8 to 1.0. Assuming effective length factor equal to 0.85 for the condition of partial fixity at top and bottom,

∴ Approximate effective length $= 0.85\,L$

$D = 500\ mm$

Bending about x- axis :

Maximum unsupported length $= 3350 - 500 = 2850\ mm$

∴ Effective length $= L_{effx} = 0.85 \times 2850 = 2423\ mm$

$\therefore L_{effx}/D = 2423/500 = 4.8 < 12$

∴ Column is short

Bending about y - axis

(a) At any intermediate storey

Maximum unsupported length $= 3350 - 300 = 3050\ mm$

Effective length $= L_{eff} = 0.85 \times 3050 = 2592\ mm$

$\therefore L_{effy}/b = 2592/230 = 11.2 < 12$

∴ Column is short

(b) Column between Plinth and Footing

Unsupported length $= L = 3300 - 300 = 3000\ mm$

Since the bottom end is hinged the effective length factor is assumed equal to 0.9

∴ Effective length $= 0.9 \times 3000 = 2700\ mm$

$\therefore L_{effy}/b = 2700/230 = 11.7 < 12$

∴ Column is short

8.17.5 Grouping of Columns

Since loads and moments in the three columns are totally different, each of the column is required to be designed seperately. However, when entire building is to be designed, there will be a number of other columns which can be group together.

8.17.6 Design of Column Section

Since exact values of P_u and M_u are known for all storeys for all columns, the column sections will be designed, using charts, software, and tables.

Various combinations of axial load and bending resistances of column sections 230 *mm* and 200 *mm* wide along with various depths, for areas of bars ranging from minimum to maximum are given in Table G-2.. The maximum reinforcement is limited to 3% because with increase in percentage of steel the cost goes on increasing. The values of combination of P_u-M_u from tables may be referred for the given depth of section starting from the rightmost column (where P_u is much larger than M_u)on the first line corresponding to minimum steel and moving leftwards and downwards till you arrive at the values of P_u and M_u greater than design P_u and M_u. The tables avoid all calculations required in use of charts.

Charts are in non-dimensional form and can be used for any width and depth of the column. The values of p/f_{ck} in between the curves are to be judged visually. Further, they have been given for specific ratios of d_c/D (= 0.05, 0.10, 0.15, 0.20, 0.25) and required to be interpolated / extrapolated for values less than *or* greater than the above ratios.

The software prepared by the Author[8.9] is very simple in operation and gives accurate results very fast. It enables one to try different combinations of main steel for column, to achieve economy.

All the columns are subjected to axial loads and uniaxial bending. They will be designed to resist P_u and M_u for bending about x - axis *or* y - axis corresponding to minimum eccentricity whichever governs. The minimum eccentricity is calculated using the *Eq.*4.8.5 given by $e_{min} = L/500 + h/30 < 20\ mm$.

The axial compression and ultimate moments at the top and bottom of columns are given in *Table 8.17.1* and shown in *Fig. 8.17.4* for ready reference. For design, the load at the base of the column is considered in combination with maximum of the two moments at top and bottom of column.

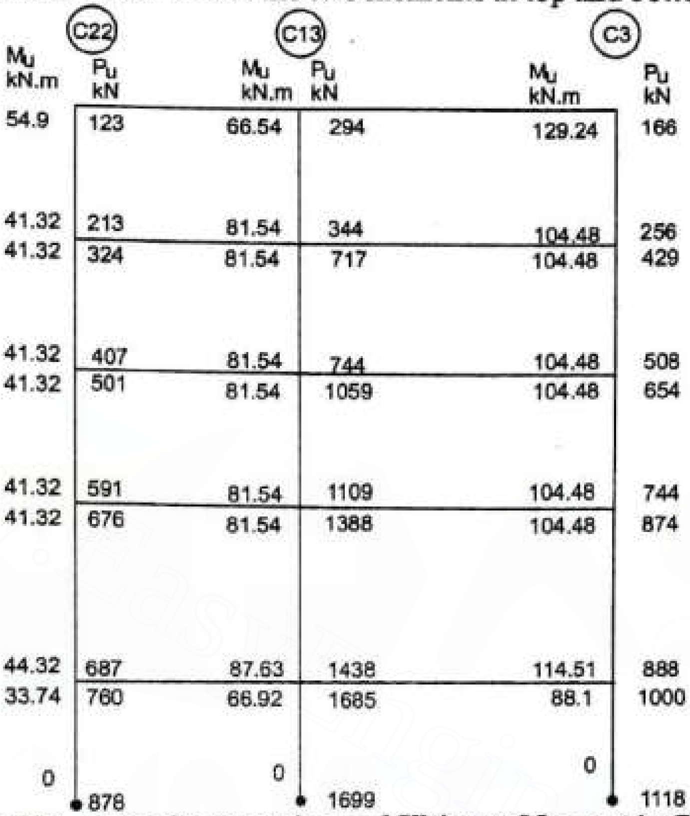


Fig. 8.17.4 Axial Compression and Ultimate Moment in Columns

Column C13 – *Design of Column using Charts (see Appendix G-3)*

Storey	L mm	P_u kN	M_{ux} kN.m	$M_{uy.min}$ kN.m	M_{uD} kN.m	d_c/h	$\frac{P_u}{f_{ck}bD}$	$\frac{M_u}{f_{ck}bD^2}$	$\frac{p}{f_{ck}}$	No - Dia. mm mm mm	Links ϕ - s mm mm	p_c%
R_f-3	2890	344	81.54	6.88	81.54	0.11	0.15	0.071	0.046	4#16+2#12	ϕ6 - 190	0.9
3-2	2850	744	81.54	14.88	81.54	0.11	0.32	0.071	0.0284	4#16+2#16	ϕ6 - 230	1.05
2-1	2850	1109	81.54	22.8	81.54	0.11	0.48	0.071	0.0635	6#6+4#16	ϕ6 - 230	1.28
1-P_L	2850	1438	87.63	28.76	87.63	0.12	0.625	0.076	0.109	4#25+4#16	#8 - 230	2.18
P_L - F_t	2500	1699	66.92	33.98	*33.98	0.26	0.73	0.064	3248	6#25+4#16	#8 - 230	3.3

Column C13 – *Design of Column using Software*[8.9]

Storey	L mm	P_u kN	M_{ux} kN.m	$M_{uy.min}$ kN.m	M_{uD} kN.m	$(P_u)_{prov}$ kN	$(M_u)_{prov}$	k_u	No - Dia. mm mm	R_o	ϕ - s mm -mm	p_c%
R_f-3	2890	344	81.54	6.88	81.54	345	113.3	0.441	4#16+2#12	3	ϕ6-190	0.9
3 - 2	2850	744	81.54	14.88	81.54	744.5	100.95	0.715	4#16+2#16	3	ϕ6-230	1.05
2 - 1	2850	1109	81.54	22.18	81.54	1109	98.15	0.86	6#16+4#16	4	ϕ6-230	1.28
1 - P_L	2850	1438	87.63	28.76	87.63	1439	92.79	0.973	4-#25+4#16	4	#8-230	2.4
P_L - F_t	2500	1699	66.92	33.98	*33.98	1701	36.03	0.91	6-#25+4#16	4	#8-230	3.3

Remark : *Even though* $M_{ux} > M_{uy.min}$, * *the minimum moment about y - axis governs.*

Column C3 - *Design of Column using Software*[8.9]

Storey	L	P_u	M_{ux}	$M_{uy.min}$	M_{uD}	$(P_u)_{prov}$	$(M_u)_{prov}$	k_u	N#1	RO	ϕ-s	p_c
R_f-3	2.85	256	129.24	5.12	129.24	256	136.7	0.38	6#16+2#12	3	ϕ6-190	1.24
3-2	2.85	508	104.48	10.16	104.48	508	141.3	0.58	6#16+2#12	3	ϕ6-190	1.24
2-1	2.85	744	104.48	14.88	104.88	745	121.2	0.72	6#16+2#12	3	ϕ6-190	1.24
1-P_L	2.85	888	114.51	19.28	114.51	889	123.2	0.73	6#16+4#16	4	ϕ6-230	1.75
P_L-F_L	2.50	1118	88.1	23.9	88.1	1119	96.9	0.87	6#16+4#16	4	ϕ6-230	1.75

Column C22 - *Design of Column using Software*[8.9]

Storey	L	P_u	M_{ux}	$M_{uy.min}$	M_{uD}	$(P_u)_{prov}$	$(M_u)_{prov}$	k_u	N#1	RO	ϕ - s	p_c%
R_f-3	2.85	213	54.9	4.26	54.9	302	112.5	0.43	4#16+2#12	3	ϕ6-190	0.9
3-2	2.85	407	41.32	8.14	41.32	408	115.2	0.50	4#16+2#12	3	ϕ6-190	0.9
2-1	2.85	591	41.32	11.82	41.32	592	110.9	0.64	4#16+2#12	3	ϕ6-190	0.9
1-P_L	2.85	687	44.32	15.26	44.32	688	102.8	0.7	4-#16+2#12	3	#6-190	0.9
P_L-F_L	2.50	878	33.74	19.08	44.32	878	33.9	0.82	4-#16+2#12	3	#8-190	0.9

8.18 DESIGN OF FOOTING

The design of footing has beesn made using the software developed by the Author.[8.10]

Step	Design Calculations		C22	C13	C3
I	***Data :***				
	Maximum column load P_u		878	1699	1118
	Design working load = $P = P_u / 1.5$		585.3	1133	745.3
	Column Section	*mm x mm*	230 x 500	230 x 500	230 x 500
	Bearing capacity of soil	*kN/m²*	200	200	200
	Material used		M20, Fe415	M20, Fe415	M20,Fe415
	Offset at top of footing	*mm*	75	75	75
II	***Proportioning of Base size***				
	Area of footing required	*m²*	3.22	6.23	4.10
	Area of footing provided	*m²*	3.23	6.26	4.12
	Length of footing provided	*mm²*	1940	2440	2170
	Breadth of footing provided	*mm²*	1670	2370	1900
	Projection from column face	*mm²*	720	1070	835
	$w_u = P$ x 1.5/Area of footing	*kN/m²*	270.9	271.62	271.1
III	***Depth of footing required from B.M. Considerations :***				
	M_{ux}	*kN.m*	117.3	368.51	179.6
	M_{uy}	*kN.m*	136.2	410.49	205.1
	Depth for B.M	*mm*	390	650	470

Step	Design Calculations	C22	C13	C3
IV	***Depth of footing required from Two - way shear consideration :***			
	Perimeter at critical Sect B_2 mm	2772	3860	3080
	Depth at peripheral Sect D_2 mm	294.9	486.9	351.3
	Area resisting shear A_2 mm²	817418	1879568	1082053
	Shear resisted by concrete V_{uc2} kN	877.3	2017	1161.4
	Design shear $V_u D_2$ kN	752.7	1451	962.1
	Depth for 2-way shear $(D_f)_{req}$ mm	390	650	470
V	***Depth increased to satisfy One-way shear requirements:*** $(D_f)_{prov.}$	500	700	550
	Revised calculations for two -way shear			
	B_2 mm	3212	4060	3400
	D_2 mm	359.9	524.4	407.8
	A_2 mm²	1155872	2128950	1386730
	V_{uc2} kN	1241	2285	1488.4
	V_{uD2} kN	708	1424	927
	Area of steel			
	Area of steel provided about y-axis mm²	955	2199	1178
	Area of steel provided about x-axis mm²	809	1885	1413
VI	***Check for One-way Shear for Bending about y-axis***			
	A_y mm²	424433	847138	586305
	p_{ty}	0.224	0.26	0.24
	τ_{ucy} N/mm²	0.35	0.365	0.35
	V_{ucy} kN	148.9	309.2	207.4
	V_{uDy} kN	148.2	308.3	205.9
VII	***Check for One-way shear for Bending about x-axis***			
	A_x mm²	367880	767058	519763
	p_{tx}	0.23	0.246	0.23
	τ_{ucx} N/mm²	0.348	0.356	0.34
	V_{ucx} kN	128.0	273.5	179.0
	V_{uDx} kN	124.0	270.37	175.1
(VIII)	***Results***			
	Length of Footing L_f mm	1940	2640	1900
	Breadth of Footing B_f mm	1670	2370	2170
	Total depth of footing D_f mm	500	700	500
	Minimum Depth of footing $D_{f.min}$ mm	150	200	200
	Offset at top of footing mm	75	75	75
	No.Dia of bars along long direction N_x – #	17 – #8	24 – #10	15 – #10
	No. Dia. of bars along short direction N_y – #	20 – #8	28 – #10	18 – #10
	Clear dist. of bars along long direction C_{Lx} mm	88	86	115
	Clear dist. of bars along Short direction C_{Ly} mm	88	83	111

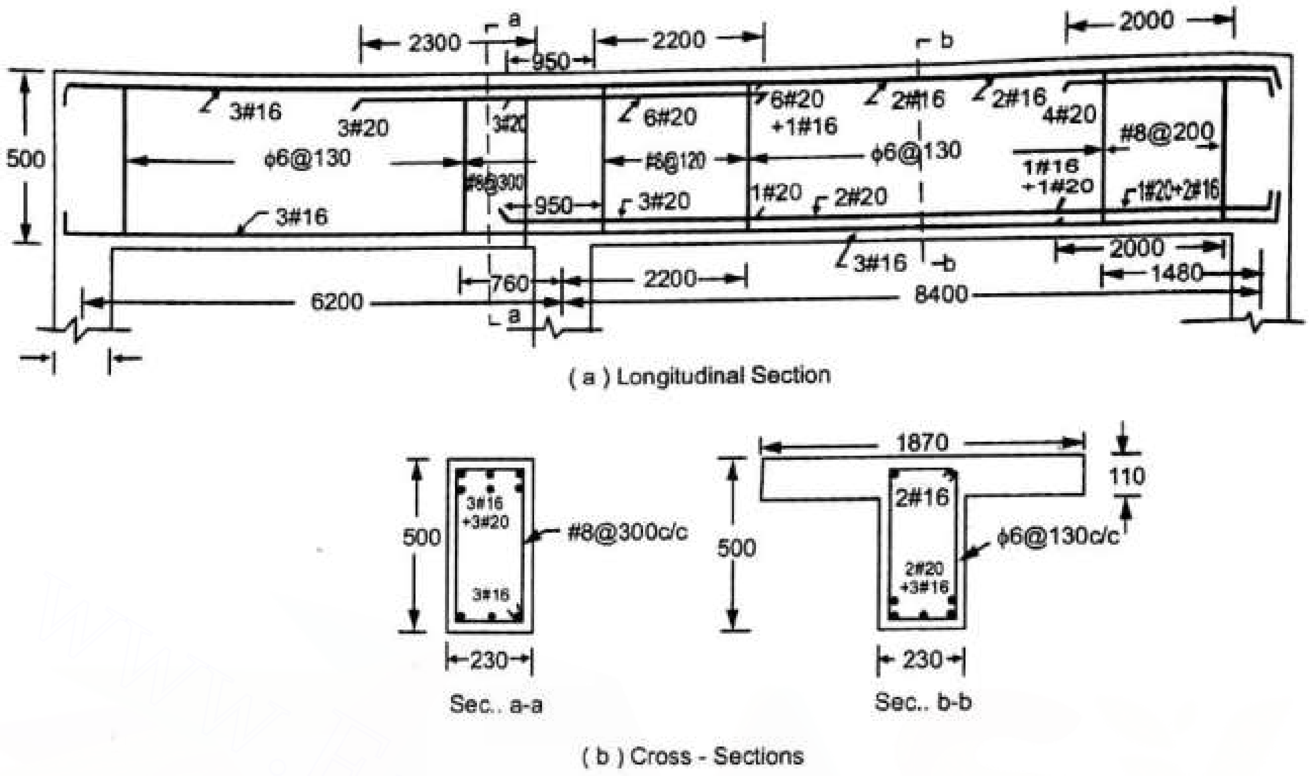


Fig. 8.18 Details of Reinforcement of Transverse Beam (Sect. 8.16.1)

8.18.1 References :

8.1 IS:456-2000, " Indian Standard Plane and Reinforced Concrete - Code of Practice", BIS, New Delhi, 2000, *pp*207

8.2 IS:875-1987(pt.2), "Code of Practice for design of loads for buildings and structures - Part 2 Imposed loads", BIS, New Delhi, 1987, *pp*208

8.3 Shah V.L. "Select observations on IS:456-2000", National workshop on IS:456-2000, Maharashtra India Chapter of AC1, Mumbai, Oct, 2000, *pp* 136-138

8.4 Basu, P.C., "Observations on design provisions", The Indian Concrete Journal, Vol. 75, No.2, 2001, *pp* 137-144

8.5 SP24:1983, "Explanatory hand book on IS code of practice for plain and reinforced concrete", BIS, New Delhi, Fig. E-6, *pp*50

8.6 Mosley, W.H., Bungay, J.H., Hulse, R., "Reinforced concrete design", Macmillan Press, 1999, *Sect. 3.4.1*

8.7 Rao, K.L., "Reinforced concrete", Charotar Publications, Anand, 1983, *Chap - 16*

8.8 SP24: 1983, "Explanatory hand book on Indian Standard Code of Practice for Plain and Reinforced Concrete IS:456-1978" BIS, New Delhi, 1983, Sect. 21.4.2(b), *pp*50

8.9 Shah, V.L., "Software package-3 for Design of axially loaded column, Column subjected to Uniaxial bending and Bi-axial bending including Slender column as per IS-456-2000", Structures Publications, Pune 411009

8.10 Shah, V.L., "Software package-3 for design of pad *or* sloped footing for axially loaded column as per IS:456-2000", Structures Publications, Pune 411009

CHAPTER - 9

PROJECT - III : DESIGN OF MULTI-STOREYED RESIDENTIAL BUILDING

9.1 INTRODUCTION

Design of a single storeyed structure, with simple connections, has been presented, using first principles, in *Project - I* for a public building. In *Project - II*, a commercial building having regular layout of columns and which can be divided into number of plane frames has been considered and one of the typical frame is analysed using substitute frame method. All members of one such frame have also been designed. In this *Project - III*, a typical multi-storeyed residential building has been taken for design by use of *Tables* and *Charts*. In a residential building many times it is not possible to divide the building into number of plane frames due to its irregular layout. In such cases, the only alternative left is to analyse the structure by approximate method, considering free bodies of the structural components. One such typical building, shown in *Fig. 9.1.1,* is taken to illustrate the design. As reader is now conversant with design of members from first principles, the design details have been worked out using design tables, charts, etc., which considerably help in saving time and labour. The design aids used in this project have been given in Appendices or more in tables given in author's[9.4] handbook for the benefit of users.

9.2 DATA

Type	Multi-storeyed Residential Building (G+3)
Building Plan	As shown in *Fig. 9.1.1*
Floor to floor height	= 3000 *mm*
Height of Plinth	= 450 *mm* above G.L.
Depth of Foundation	= 1000 *mm* below G.L..
Bearing Capacity of Soil	= 200 kN/m^2
External Walls	= 200 *mm* thick
Internal Walls	= 100 *mm* thick
Assumed Imposed Loads :	
Roof : Roof Finish	= 1.5 kN/m^2
Live load	= 1.5 kN/m^2
Total Load	= 3.0 kN/m^2
Floor : Floor Finish	= 1.0 kN/m^2
Live Load	= 2.0 kN/m^2
Total Load	= 3.0 kN/m^2

Assumed Materials :
Concrete M20,
Steel : Main - Fe415, Secondary - Fe250
Unit weight of concrete = 25 kN/m^2
Unit weight of bricks masonry = 20 kN/m^2
Design Basis : Limit State Method based on IS:456-2000.

The axonometric view of the building is shown on the cover page.

9.2.1 Structural Planning

The work of the designer starts with planning of structural members from the plan given by an architect or an engineer. It commences with deciding positions of columns, followed by positioning of beams and spanning of slabs. This can be done using guiding principles explained in *Sect. 1.3.* Even using these principles, there can be a number of possible solutions for layout of columns. In such cases it is only to be seen that it is neither technically incorrect nor uneconomical. The one adopted by authors is shown in *Fig. 9.2.1.* It is likely that one may suggest to take columns *C23* and *C24* at the outer end of the stair. But in that case, the span of the stair slab and floor beams *B15* increases leading to heavy sections resulting in uneconomical design. The suggested positions of these columns make the mid-landing slab overhanging beyond beam *B19* and part of the floor beam cantilever. This reduces the mid-span moments in stair slab as well as in beam *B15* making the design economical.

Once the positions of columns are decided, most of the locations of beams get automatically fixed from the positions of columns and walls. Again spanning of slabs can be done in different ways. The solution adopted is discussed in detail in *Sect. 9.4*.

9.2.2 Numbering and Nomenclature for members

The building has symmetry in both the directions as far as the layout of rooms is concerned. Therefore, the numbering of slabs and beams is done for one quadrant and central stair corridor only. Columns are, however, numbered serially starting from left corner and proceeding rightward and then downwards to facilitate setting out of the building. Due to symmetry, the design of members is required to be done for one flat and stair portion only. The details of marking for slabs, beams, columns and centre to centre dimensions are shown in *Fig. 9.2.1*.

9.2.3 Sizing of Beams and Columns

As mentioned earlier, the width of the beam will be kept as 200 *mm* to meet fire resistance requirements. (see *Sect. 6.3.3*).

However in practice, width of beam equal to 150*mm* is also provided to avoid offset from internal walls.

For residential buildings there are two practices in selecting the depth of the beam. In one case, the depth of the beam is kept equal to the difference between the top of floor and top of door frame. The advantages are that provision of lintel is not required and as the bottom form work is in level, the labour and supervision required is less. The quantity of steel is reduced but the volume of concrete is more. This is again dependent on the floor to floor height. *For example :* in the present case, for the floor to floor height of 3*m* and height of top door frame 2.1 *m* the depth of the beam works out to 900 *mm* (=3000−2100). As the depth of the beam exceeds 750 *mm* additional steel equal to 1% of web area is required to be provided along the depth. When the floor to floor height is 9 feet (*i.e.* 2.74*m*) depth of the beam required is only 640*mm*.

The other practice is to provide depth required to resist the load to which it is subjected and cast-in-situ *or* precast lintel is provided so as not to hamper the progress of the work. In the present case having floor to floor height of 3*m*, separate lintel and beam are provided except for the entrance door with ventilator requiring depth of beam of 600 *mm* only.

The width of the column is kept as 200 *mm* and steel not exceeding 3% preferably. The ratio of depth of columns to width of column will be limited to 3.As already mentioned,the number of types of beams and types of columns have been kept minimum to enable one to reuse the form work for economy.

9.3 ULTIMATE LOADS

As per *Table A-3* of Appendix.

(1) Roof : Assumed D = 120 *mm*, $w_u = 9\ kN/m^2$ (=25 x 0.12 + 1.5 +1.5) x 1.5

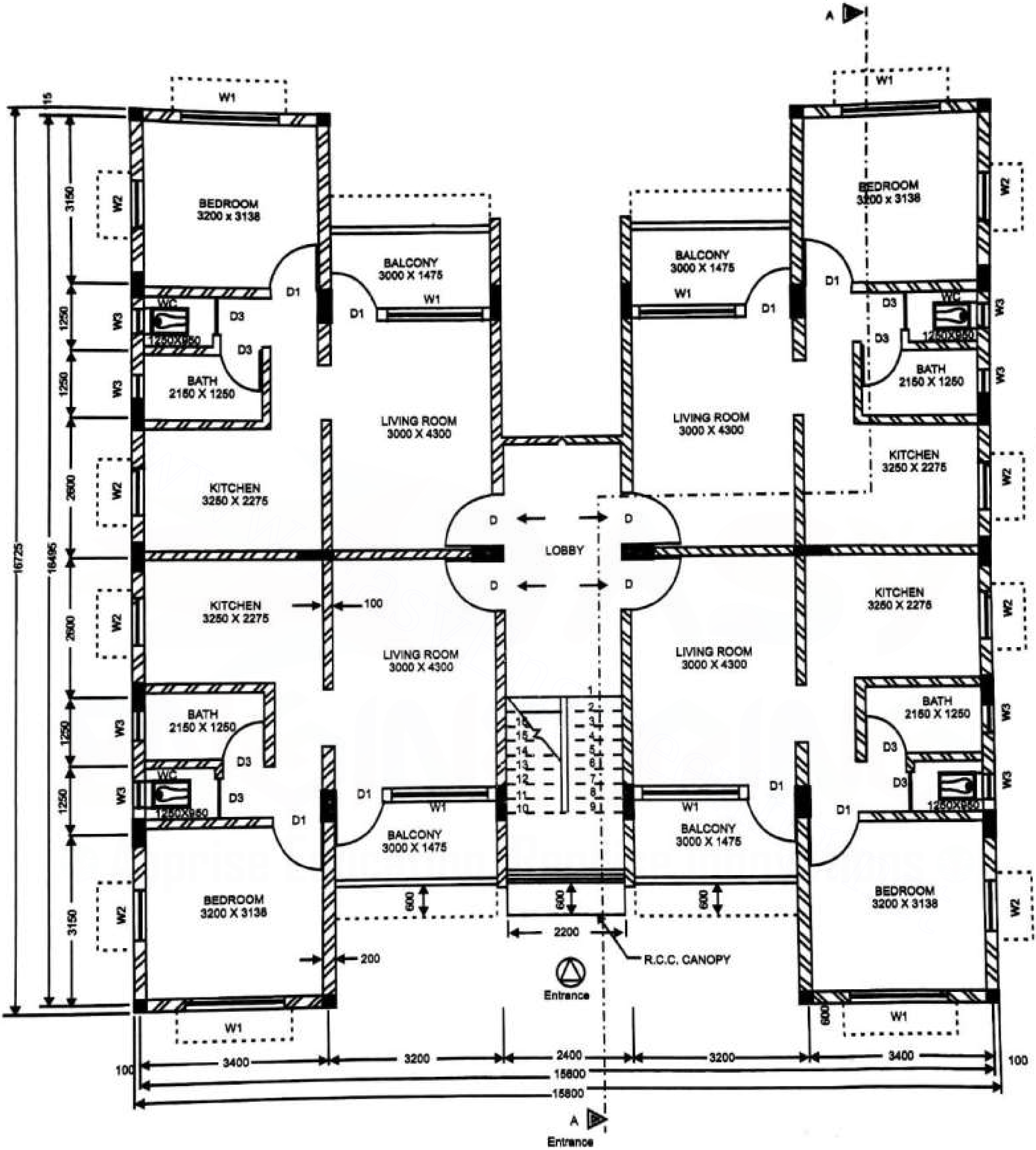
(2) Floor : Assumed D = 120 *mm*, $w_u = 9\ kN/m^2$ (=25 x 0.12 + 1+2) x 1.5

(3) Bath - W.C. : Assumed D = 100 *mm*, $w_u = 10.5\ kN/m^2$ (=25 x 0.1 + 2.5 +2) x1.5

(4) Loft : Assumed D = 100 *mm*, $w_u = 8\ kN/m^2$ (=25 x 0.1 + 0.75 +2)x1.5

(5) Balconies : Cantilever: Assumed D = 150*mm*, $w_u = 12\ kN/m^2$ (=25 x 0.15 + 1.0 + 3.0) x 1.5

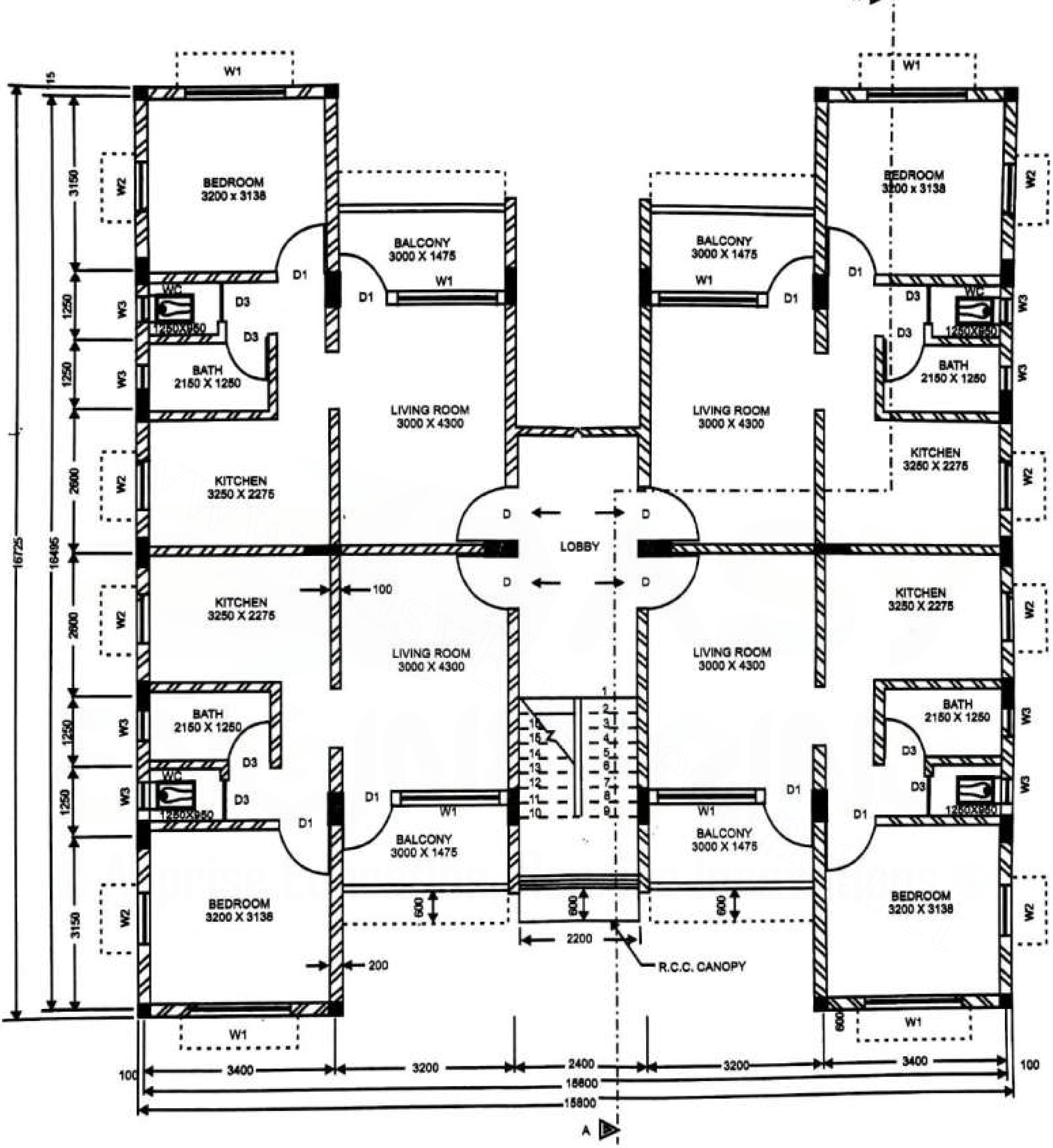
Simply supported : Assumed D = 100*mm*, $w_u = 10\ kN/m^2$



GROUND FLOOR PLAN

SCHEDULE OF DOORS :

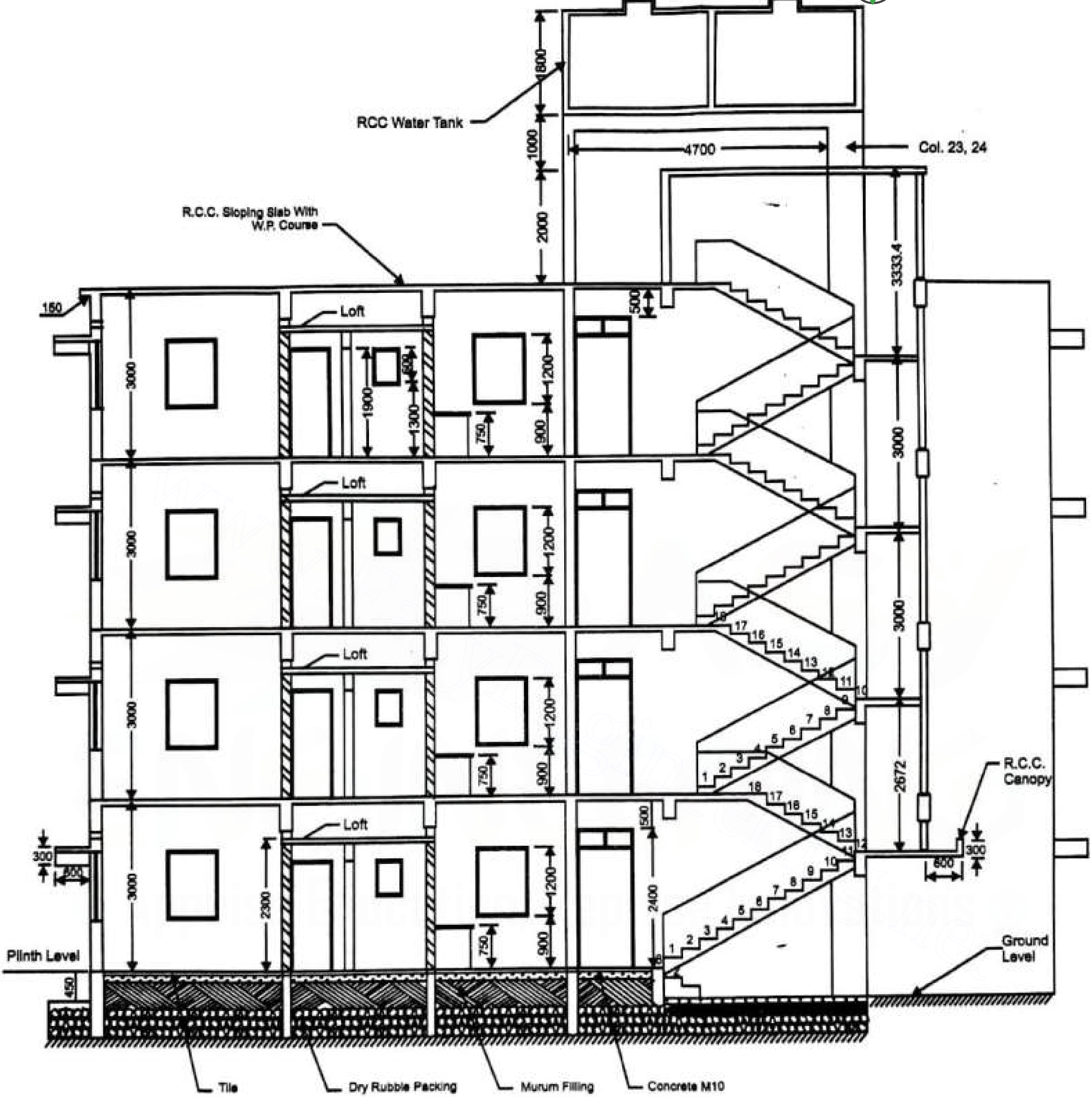
TYPE	SIZE	DESCRIPTION
D	1000 X 2400	T. W. FRAME/COMMERCIAL BLOCK BOARD WITH GLAZED FAN LIGHT
D1	900 X 2100	T. W. FRAME/COMMERCIAL BLOCK BOARD SHUTTER
D2	900 X 2000	T. W. FRAME/COMMERCIAL BLOCK BOARD SHUTTER
D3	800 X 1900	T. W. PANELLED SHUTTER



TYPICAL FLOOR PLAN

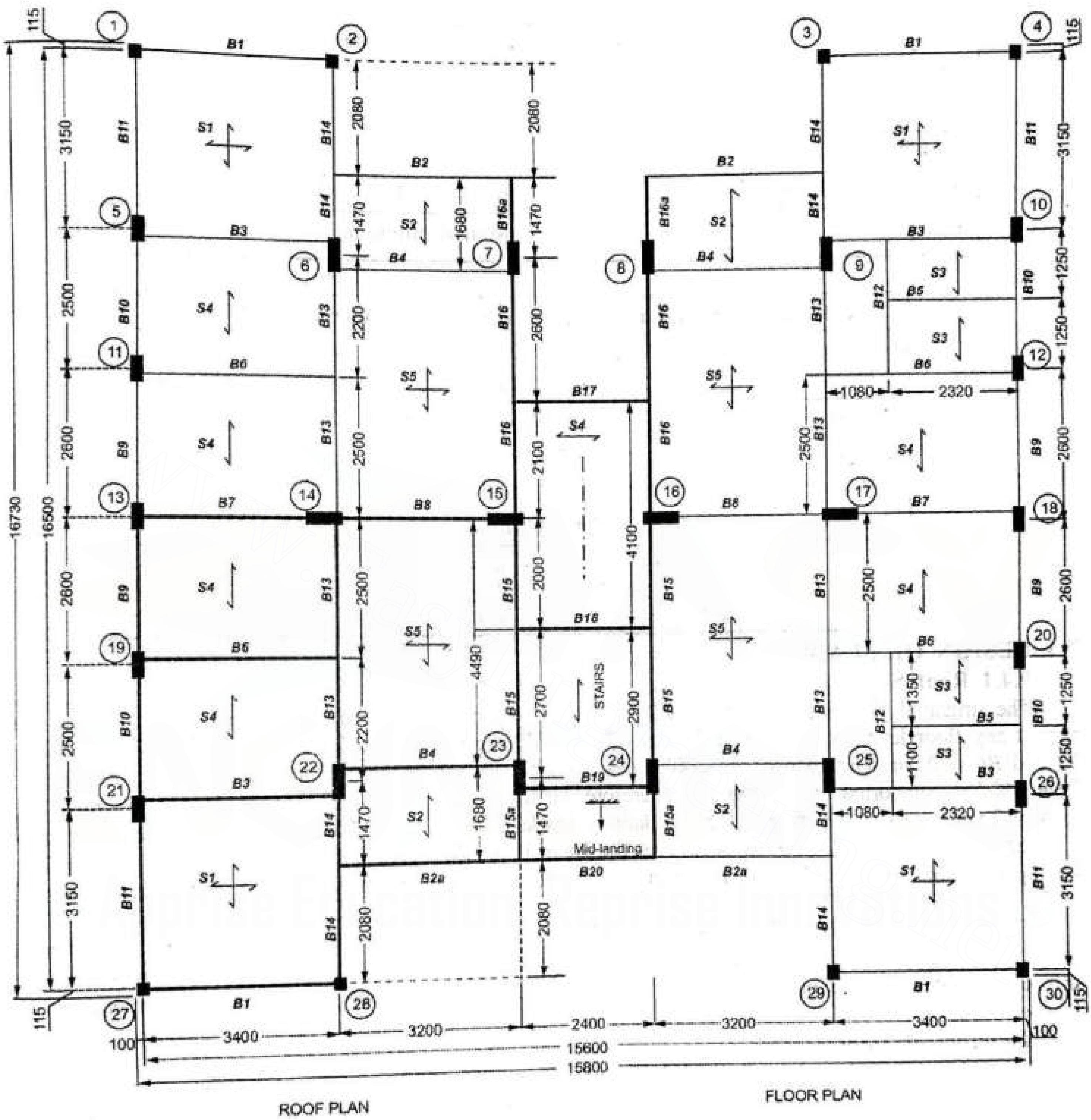
SCHEDULE OF WINDOW :

TYPE	SIZE	DESCRIPTION
W1	1800 X 1200	ALUMINIUM SLIDING GLAZED WINDOW (Three Shutters)
W2	900 X 1200	ALUMINIUM SLIDING GLAZED WINDOW (Two Shutters)
W3	450 X 650	ALUMINIUM LOUVERED WINDOW



Section A-A

	STRUCTURES PUBLICATIONS		
	Four Storeyed Residential Building		
PROJECT NO.	3	STRUCTURAL ENGR.	
PLOT NO.		DRG. NO.	
S. NO.		DRG. BY	
DATE		SIGN	



Notes :

1. Width of column = 200 *mm*
2. Width of internal walls = 100 *mm*
3. Width of external walls = 200 *mm*
4. Beam *B19* is at mid-landing level.
5. Mid-landing slab supported by mid-landing beam cantilevering from beams *B19*
6. Beam *B20* is at floor level supporting the staircase and grill

Fig. 9.2.1 Structural Plan

(6)

Table 9.3.1 Factored self weight of Beams w_{us}

Depth of Beams D in mm	300	380	450	600
	w_{us} in kN/m			
* Flanged Beam b_w = 200 mm	1.5	2.1	2.6	3.8
Rectangular Beam b = 230 mm	2.6	3.3	3.9	5.2
* Depth of rib = $(D-D_f)$ = $(D-100)$, for assumed minimum slab depth of 100 mm				

(7) Walls

Table 9.3.2 Ultimate Loads of Brick masonry for Different Heights

Height in m	*Solid Brick 200 mm* (225 mm with plaster)	*Solid Brick 100 mm* (125 mm with plaster)
1.0 m	6.8 kN/m	3.8 kN/m
2.0 m	13.5 kN/m	7.5 kN/m
2.7 m	18.2 kN/m	10 kN/m
3.0 m	20.2 kN/m	11.3 kN/m

Note : 1) Unit weight of brick masonry with plaster assumed = 20 kN/m^3
2) Floor to floor height of 3m and assuming minimum depth of beam=0.3m net height of brick wall = 2.7 m (=3 − 0.3)

9.4 DESIGN OF SLABS

9.4.1 Roof Slab

The structural plan is shown in *Fig.9.2.1*. The left half is a plan at roof level and right half shows the plan at any floor level. At roof level, beams *B5* and *B12* are not provided. The span of slab supported on *B3* and *B6* is 2.5*m*. The span of this slab does not differ by more than 15% of the longest slab span of 2.6*m* which is supported by *B6* and *B7*. Therefore, all these slabs are categorised as *S4* and designed for a span of 2.6*m*. The slab *S3* is provided for sanitary blocks at floor level. Therefore, the category of slabs *S3* is absent at the roof level. The cap slab over the stairs is provided above the door level (*i.e.* at 2*m* height). The span of this slab will be 2.4 *m* and it will be supported by beams *B15a* and *B15* over a length of 4.17*m*.

The water tank of size 4.17 x 2.4 *m* x 2*m* having capacity of 17,000 liters will be provided over the columns *C15,C16,C23,C24*. The bottom of the tank shall be 1*m* above the staircase cap slab so that any unforeseen water leakage problem can be attended independently without causing inconvenience to the occupants.

Spanning of Slabs

It has been mentioned in *Sect. 1.3.3*, that in case of residential buildings when short-span is less than 3*m*, there is no special advantage in designing a slab as two-way even though the slab may be supported on all sides and the aspect ratio L_y / L_x is less than 2. This is because minimum requirement of main steel (viz.#8 *mm* bars at maximum spacing of 3*d*) governs.

For example,

(a) When designed as two-way slab.

For slab *S4*, $w_u = 9kN/m^2$, $L_y = 3.4\ m$, $L_x = 2.6m$, $L_y/L_x = 1.3$, $\alpha'_x = 0.051$, *(Table D-7, Case2)*

$M'_{ux} = 0.051 \times 9 \times 2.6^2 = 3.1\ kN.m$

For minimum total depth of 100 mm, $d = 100 - 20 - 8/2 = 76\ mm$,

$A_{st}' = 117\ mm^2$, requires #8 mm at #420mm > 225 mm (=3d)

∴ #8mm at 225mm c/c will be required to be provided for +ve and −ve steel along short span and along long span.

(b) When Designed as one-way slab

$M_u = w_u L^2/12 = 9 \times 2.6^2/12 = 5.1\ kN.m$ for minimum depth of 100 mm, $d = 76\ mm$

A_{st} = 197 mm^2 requires #8*mm*@225 *mm (Table E-2b)*

Provide #8*mm* at 225*mm* c/c for +ve and -ve steel along short span and distribution steel of φ6*mm* at 180 *mm* c/c *or* #8*mm* at 400 *mm* c/c.

However, as mentioned earlier minimum steel (0.12% of *bD* for *HYSD or* 0.15% of *bD* for mild steel) should be provided for a distance of 0.3*L* across the short edge support to avoid cracking. Therefore, slabs *S2* and *S4* having spans less than 3*m* have been designed as one-way spanning across short spans.

Slabs *S1* and *S5* which are supported along all edges, have short span greater than 3*m* and $L_y/L_x < 2$, prove to be uneconomical if designed as one-way.

If slab *S1* is designed as a one-way continuous slab spanning across beam *B1* and *B3* requires *D* = 110 *mm* for span of 3.15 *m* and for M_u = 8.93 *kN.m*(=9 x 3.15²/10) requires #8*mm* at 160 *mm* c/c *(see Table E-2B)*. If the same slab is made to span across *B11* and *B14*, the slab has a span of 3.4 *m* and is discontinuous at both ends. It requires *D* = 140 *mm* with #8 at 140 *mm c/c*. On the contrary, if it is designed as two-way, it requires *D* = 100 *mm* with #8*mm* at 225 *mm* c/c along short span and #8*mm* at 200 *mm* c/c along long span, thus proving economical.

Slab *S5*, if designed as one-way across beam *B13* and *B15*, does not get continuity over *B13* because slabs *S4* beyond *B13* is a one-way slab spanning in the direction at right angles to it. It also does not get continuity over the full length of beam *B15* because slab *S4* beyond it even though spans in the same direction as *S5* under discussion, exists only over smaller part of its length. Thus, it would be discontinuous at both opposite edges and would require *D* = 130 *mm* with #8*mm* at 160 *mm* for *L* = 3.2 *m*. On the contrary, if *S5* is designed as two-way, it gets continuity at both supports *B4* and *B8* in short direction, requiring *D* = 100 *mm* with #8*mm* at 220*mm* c/c, thus proves to be economical. This is essentially because of stringent requirements of serviceability for one-way slabs *(Cl.22.2 of code)* as compared to those of two-way slab *(Cl.23.1 Note 2 of the code)*.

In two-way slab category, since *S1* and *S5* have different aspect ratios (L_y/L_x) and boundary conditions, therefore they are not grouped together but designated and designed separately.

As per *Note -1* of *Table 16* of IS:456-2000, for main steel up to 12*mm* diameter bar for mild exposure the nominal cover will be 15*mm*. But nominal cover of 20*mm* for slab has been provided to meet the fire resistance of 0.5 hour *(see IS:456-2000, Table 16A)*

Category - 1 : One - Way Slabs

Step No.	Slab Mark		S4	S2	Reference	Note No.
1.	*Span*	*L m.*	2.6	1.68		
2.	*End Condition (EC) Number*		3/(2)*	2		1
3.	*Ultimate Load*	w_u *kN/m²*	9	10	*Table A-3*	
4.	*Ultimate moment* M_u = 9 x 2.6²/12	*kN.m*	5.07	2.82		
5.	*Required Depth*	*D mm*	100	100	*Table E-1*	
6.	*Required Steel*					
	(a) Short span Steel : Dia. (*mm*) - Spacing (*mm*)		#8-225	#8-225	*Table E-2B*	2
	(b) Long Span Steel : Dia. (*mm*) - Spacing (*mm*)		φ6-180	φ6-180		
7.	(a) End Shear in *kN* :					
	Long edge -					
	Penultimate support,	$SF = 0.6w_uL$ for EC = 2	*14.0	10.1		
	Continuous support ,	$SF = 0.5\ w_uL$ for EC = 3	11.7	-		
	Simple support ,	$SF = 0.45\ w_uL$ for EC =2*	10.5	7.6	*Sect. 5.3.1*	
	(b) Short edge - Discontinuous end	$SF = w_uL/6$	3.9	2.80		3

Category - 1 : One - Way Slabs continued.....

Step No.	Slab Mark		S4	S2	Reference	Note No.
8..	*Check for Development length*					
	(a) At Continuous End :					
	Required $L_d = 47\phi = 47 \times 8$	*mm*	376	-	*Table 4.6.2*	
	Available $L_d = L/4 = 2600/4$	*mm*	650	-		
	(b) At Simply supported End : $V_{uD} = V$	*kN*	-	7.6		
	At mid-span, M_{ur} for #8*mm*@225*mm* c/c	*kN.m*	-	5.75	*Table E-2B*	
	Assuming alternate bars bent up at support,					
	*M*l available at bottom = 5.75/2	*kN.m.*	-	2.9		
	Required $L_o = L_d - 1.3M_1/V$					
	Assuming 90^0 bend available					
	$L_o = b_s/2 - x_1 + 3\phi = 200/2 - 20 + 3 \times 8$	*mm*	-	104	*Eq. 4.6.4a*	
	However, provide minimum end anchorage					
	$= L_d/3 = 376/3$	*mm*	-	125		
	Anchorage available from inner face					
	$= L_o + b/2 = 104 + 200/2 = 204$	*mm*	-	204	∴ o.k.	
9.	*Check for Shear*					
	(a) At Continuous End :					
	Maximum design shear = V_{uD}	*kN*	11.7	10.1		
	A_{st1} for #8*mm* at 225*mm* c/c	mm^2	223	223	*Table H-3*	
	Effective depth = 100 – (20 + 8/2)	*mm*	76	76		
	p_t = 223 x 100/(1000x76)	%	0.29	0.29		
	τ_{uc}	N/mm^2	0.38	0.38	*Table 4.4.1*	
	Multiplying factor for slab Depth = 100	*mm*	1.3	1.3	*Sect. 4.4.2*	4
	$V_{uc} = 1.3 \times \tau_{uc} \times b \times d/1000 = 1.3 \text{x} 0.38 \text{x} 76 > V_{uD}$	*kN*	37.5	37.5		
	(b) At Simply Supported End :					
	Design shear V_{uD}	*kN*	-	7.6		
	A_{st1} = 50% of $A_{st.max}$ = 0.5 x 223	mm^2	-	104		
	p_t = 111 x 100/(1000 x 76)	%	-	0.14		
	τ_{uc} =	N/mm^2	-	0.28	*Table 4.4.1*	
	$V_{uc} = 1.3 \times 0.28 \times 76 > V_{uD}$	*kN*	-	27.6		4

Step No.

1. End conditions : EC = 1 for both ends simply supported. EC = 2 for one end simply supported and the other continuous. EC = 3 for both ends continuous.
   For EC = 2 , B.M. = $w_u L^2/10$
   Now, for two-span continuous slab, using elastic analysis for ultimate load.
   Support moment = $w_u L^2/8$ and Mid-span moment = $w_u L^2/16$.
   With 20% redistribution of moment at support.
   Support moment = $w_u L^2/10$ and mid-span moment = $w_u L^2/13$
   ∴ Ultimate moment of ± $w_u L^2/10$ at support and at mid-span has been used.
   * For *S4*, EC = 3 for roof slab, EC = 2 for floor slab.
2. Spacing of 225 *mm* is governed by maximum spacing of 3*d* (= 3 x 76 = 225*mm*).
3. This load shall be taken for design of short beam only and not on column supporting the beam since load over full area of slab has been taken on long beam.
4. It is observed that slabs carrying UDL are usually safe in shear and hence calculations for shear check can be omitted safely.

Category -2 : Two - way Slabs

Step No.	Slab Mark	S1	S5	Reference	Note No.
1	*Span*				
	(a) Short Span L_x *m*	3.15	3.20		
	(b) Long Span L_y *m*	3.40	4.49		
	(c) Aspect ratio $L_y/L_x = \beta$	1.08	1.4		
2.	*Boundary Case No.*	7	6	*Table D-7*	
3.	*Required Depth*				
	(a) Allowable *L/D* ratio	32	32	*Sect. 4.7.1b*	
	(b) Depth *D* for deflection *mm*	100	100		
4.	*Ultimate load* w_u kN/m^2	9	9		
	$w_u L_x^2$ kN.m/m	89.3	92.16		
5.	*Ultimate Moment*				
	B.M. coefficients α_x (maximum)	0.063	0.063	*Table D-7*	1
	α_y (maximum)	0.043	0.045		
	$M_{ux} = \alpha_x \times w_u L_x^2$ kN.m/m	5.63	5.81		
	$M_{uy} = \alpha_y \times w_u L_x^2$ kN.m/m	3.84	4.15		
6.	*Main Steel* :Short Span *d* =76,Dia (*mm*) s - (*mm*)	#8-225	#8-220	*Table E-2B*	2
	Long Span :*d* =68,Dia (*mm*) s - (*mm*)	#8-200	#8-200		3
7	*Shear*				
	(a) Equivalent ultimate UDL to be transferred from slab to beam for ***Bending Moment***				
	Long Edge : Loads in *kN*.				
	End support (EC=2) $w_{ueqb} = 0.45 w_u L_x [1-1/(3\beta^2)]$	9.11	-	*Eq. 5.3.5*	4
	Penultimate support, (EC=2) $w_{ueqb} = 0.60\ w_u L_x [1-1/(3\beta^2)]$	12.15	-		
	Continuous / Discontinuous support (EC =1 *or* 3), $w_{ueqb} = 0.50 w_u L_x [1-1/(3\beta^2)]$	-	11.95		
	Short Edge : Continuous / Discontinuous support (EC = 1 *or* 3), $w_{ueqb} = w_u L_x /3$	9.45	9.60	*Eq. 5.3.3*	
	(b) Equivalent ultimate UDL to be transferred from slab to beam for ***Shear Force :***				
	Long Edge :				
	End support (EC=2), $w_{ueqs} = 0.45 w_u L_x [1-1/(2\beta)]$	6.85	-	*Eq. 5.3.6*	
	Penultimate support, (EC =2) $w_{ueqs} = 0.60 w_u L_x [1-1/(2\beta)]$	9.14	-		
	Continuous / Discontinuous support (EC =1 or 3), $w_{ueqs} = 0.50 w_u L_x [1-1/(2\beta)]$	-	9.26		
	Short Edge : *Continuous / Discontinuous support,* (EC=1 or 3), $w_{ueqs} = w_u L_x /4$ *See Note No. 5*	7.10	7.20	*Eq. 5.3.4*	
8.	*Torsion steel* :				

Step No. :

(1) Out of the two coefficients, one at mid-span and the other at support, the greater one is considered, and the same steel is provided at both locations for convenience of bending of bars and detailing.

(2) For short span,
Spacing is governed by maximum spacing of 3*d*.
No-dia. spacing is obtained from *Table E-2B*

(3) For long span, d_i = 100–20–8–8/2 = 68*mm* but the 3*d* = 3 x 68 = 204 *mm* say 200 *mm*
In practice, maximum spacing of main bars is restricted to 200 *mm*, in which case both long span and short span steels for all slabs having Fe415 will be #8*mm* at 200 *mm*, c/c.

(4) Continuity factors of 0.45 at simply supported end and 0.6 at penultimate support, and 0.5 at continuous support have been applied.

(5) Since the span of the slabs are not large, the division of slab into edge strip and middle strip has not been made. The requirement of torsion steel will be met with by bending all bars at discontinuous support through 180^0 and taking them at top for distance of $L_x/5$.
The details have been shown in the drawings.

Schedule of Roof Slabs :

Slab No.	*Depth mm*	*Short Span steel Dia.(mm) - spacing (mm)*	*Long Span steel Dia.(mm) - spacing (mm)*	*Remarks*
S1	100	#8-225	#8 - 200	Two -way
S2	100	#8-225	ϕ6 - 180	One -way
S4	100	#8-225	ϕ6 - 180	One -way
S5	100	#8-220	#8 - 200	Two -way

End shear from Roof slab have been shown in *Fig. 9.4.1*

9.4.2 Floor Slabs

Since assumed total loads on roof and floor slabs are the same, the floor slabs will have same design details as those of roof slab. No separate design of floor slabs is, therefore, necessary except in the case of slab across beams *B3, B5, B6, B12* and *B10* since additional beams *B5* and *B12* are provided at floor level.

If the total roof load is very less compared to floor load, separate design may be done for roof slabs. However, when the difference of loads is small the roof slabs are designed for maximum load to save computational efforts . It helps to allow the use of roof slab as floor in case of extension in future. In the floor plan under consideration, slab *S3* has maximum span of 1.35*m* and is designed as one way slab, simply supported at both ends, because slabs between *B3*, *B5* and *B6* are for Indian type W.C. and bath and they are, therefore, sunk at different levels. As a result, there is no structural continuity between slabs *S1, S3* over beams *B3* and between slabs *S4, S3* over beam *B6*. Passage slab between *B12* and *B13* will not be sunk but it will be at the floor level. This situation makes the slab *S1* discontinuous on all four edges. There is, therefore, a change in design of floor slab *S1*. This is given below separately in brief. Similarly, slab *S4* between *B6* and *B7* becomes discontinuous at edge *B6* and continuous over *B7*. However, this does not materially affect the design because the span is very small and required depth of slab and steel remains the same as *S4* of roof slab.

Slab *S3* will be just minimum 100 *mm* thick with minimum steel #8*mm* at 225*mm* for short span and ϕ6*mm* at 180 *mm* as distribution steel as obtained for slab *S2*.

End shear for Bathroom = 10.5 x1.35/2 = 7.1 *kN/m*. *see Appendix Table A-3*
End shear for loft = 8 x 1.35/2 = 5.4 *kN/m*.
Total shear = 7.1 + 5.4 = 12.5 *kN/m* for bath room

Comments : *The other practice is to sink whole bath and W.C. slab at the same level and supporting floor beams are taken up to the bottom of slab. However, one may adopt any one method depending on site conditions.*

Total end shear for W.C.= 10.5 x 1.1/2 + 8 x 1.1/2 = 10.2 *kN/m.*
Total end shear for passage slab between *B12* and *B13* = 9 x 1.08/2 = 4.9 *kN/m*
Brief design calculations for slab *S1* discontinuous on all four edges are given in tabular from below :

Slab	L_x m	L_y m	$\beta = L_y/L_x$	Boun. cond.No.	Allow L_x/D	Req. D mm	Assumed w_u kN/m²	$w_u L_x^2$ kN.m/m	α_x	α_y	M_x kN.m/m	M_y kN.m/m
S1	3.15	3.40	1.08	9	28	120	9	89.3	0.0624	.056	5.58	5.00

Short span steel : #8 at 280*mm* (*d* = 120 − 20 − 8/2 =96 *mm*) *(Table E-2a)*
Long span steel : #8 at 260*mm* (*d* = 88 *mm*)
In practice, reinforcement in both direction is provided at spacing of 200 *mm.*

Equivalent UDL for B.M- Long Edge : $w_{eqb} = 0.5 \times 9 \times 3.15\left(1 - \frac{1}{3 \times 1.08^2}\right) = 10.1$ *kN/m*

- Short edge : $w_{eqb} = 9 \times 3.15/3 = 9.45$ *kN/m*

Equivalent UDL for Shear - Long edge - $w_{eqb} = 0.5 \times 9 \times 3.15\left(1 - \frac{1}{2 \times 1.08}\right) = 7.6$ *kN/m*

- Short edge : $w_{eqs} = 9 \times 3.15/4 = 7.1$ *kN/m*

Schedule Floor Slabs,

Slab	Depth *mm*	Short span steel Dia. (*mm*) - Spacing (*mm*)	Long Span steel Dia. (*mm*) - Spacing (*mm*)	Remarks
S1	120	#8-280	#8 - 260	Two-way
S2	100	#8-225	ϕ6 - 180	One-way
S3	100	#8-225	ϕ6 - 180	One-way
S4	100	#8-225	ϕ6 - 180	One-way
S5	100	#8-220	#8 - 200	Two-way

The details of reinforcement have being shown on Drawing sheet.

9.4.3 Design of Stairs

The guide lines for fixing different parts of the stairs are given in *Sect. 1.3.4* and accordingly the tread, rise have been fixed.

Type : The waist slab of stairs spans longitudinally from *B18* with mid-landing slab overhanging over *B19.* The stair slab is simply supported on *B18* because slab *S4* beyond *B18* spans at right angles.

Span : Simply supported span of 2.9 *m* and overhang of slab = 1.26*m*

Planning : Assuming Tread T= 250 *mm* and 9 risers in each of two flights,
Rise R = 3000/18 = 167 *mm*

Hence, $\sec\theta = \sqrt{R^2 + T^2}/T = 1.20$

Total load : w_l (weight of steps + FF + LL) = 25 x 0.167/2+1+3 = 6.09 *kN/m²*
Assumed D = 140 *mm* d= 140 − 20 − 8/2 = 116 *mm*

Total ultimate load : w_u = 1.5(25 x 0.14 x 1.20 + 6.09) = 15.4 *kN/m*

Maximum support moment $M_{u,max}$ = 15.4 x 1.26²/2= 12.2 *kN.m/m*

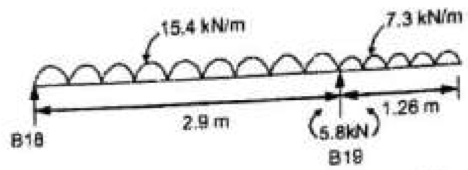


Fig. 9.4.1 Loading on Stairs for Maximum Span Moment

For maximum span moment, there shall be only *DL* on overhang, and 1.5(*DL*+*LL*)on span portion as shown in the *Fig.9.4.1* $DL = (25 \times 0.14 \times 1.2 + 6.09 - 3.0) = 7.3\ kN/m^2$

Support moment due to this $= 7.3 \times 1.26^2/2 = 5.8\ kNm/m$

Maximum span moment * $= 15.4 \times 2.9^2/8 - 5.8/2 = 13.3\ kN.m/m$

Required A_{st} at mid-span
$$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 13.3 \times 10^6}{20 \times 1000 \times 116^2}}\right] \times 1000 \times 116$$
$$= 338\ mm^2$$

Provide # 8 *mm* at 140 *mm* c/c, Area provided = 359 mm^2 (*see Table E-2B*)

Required A_{st} at support
$$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 12.2 \times 10^6}{20 \times 1000 \times 116^2}}\right] \times 1000 \times 116$$
$$= 308\ mm^2$$

Provide #8mm at 140 *mm* c/c at top of support in cantilever

Distribution steel = 0.12 x 1000 x 150/100 = 180 mm^2

Provide #8*mm* at 270 *mm* c/c (*Table H-3*)

Check for Deflection :

$f_s = 0.58 \times 415 \times 338/359 = 226\ N/mm^2$

$$(p_t)_{prov.} = \frac{100 \times 359}{1000 \times 116} = 0.31\ \%$$

for $p_t = 0.31\%$ and $f_s = 226\ N/mm^2$, Modification factor = 1.67 (*Fig. 4.4.1*)

$$\therefore \text{Required } d = \frac{1.26 \times 1000}{7 \times 1.67} = 108\ mm < 116\ mm \quad \therefore \text{ safe}$$

End Reactions : At discontinuous edge(*B18*) : $R_{B18} = 15.4 \times 2.9/2 - 5.8/2.9 = 20.3\ kN/m$

At overhanging edge (*B19*) :

Taking moments about *B18* with all spans fully loaded,

$$R_{B19} = \frac{15.4 \times (2.9 + 1.26)^2}{(2 \times 2.9)} = 46\ kN/m$$

Details of staircase Slab : Thickness = 140 *mm*

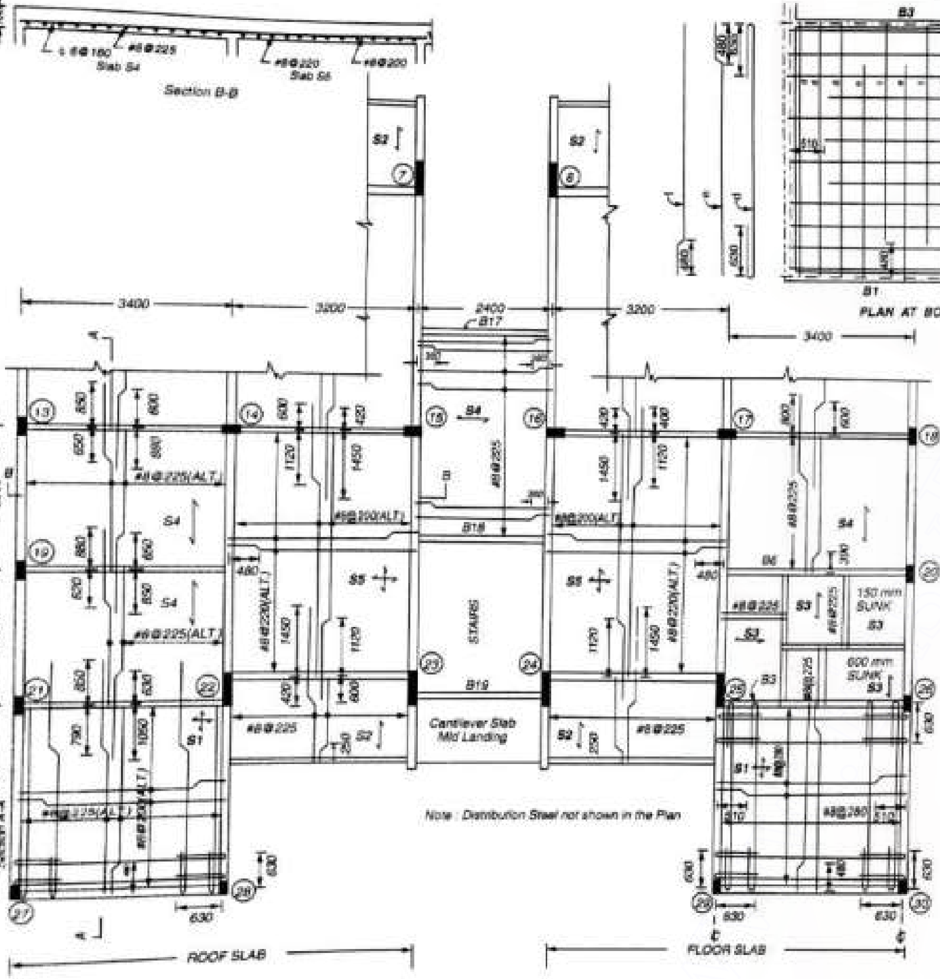
Reinforcement at mid-span and at top of cantilever = #8*mm* at 140*mm* c/c

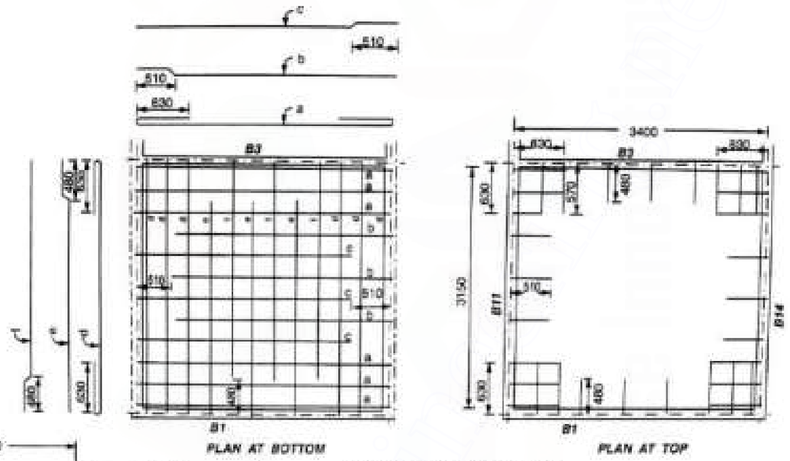
Distribution steel = #8*mm* at 270*mm* c/c ,

Reaction on *B18* = 20.3 *kN* , Reaction on *B19* = 46 *kN*

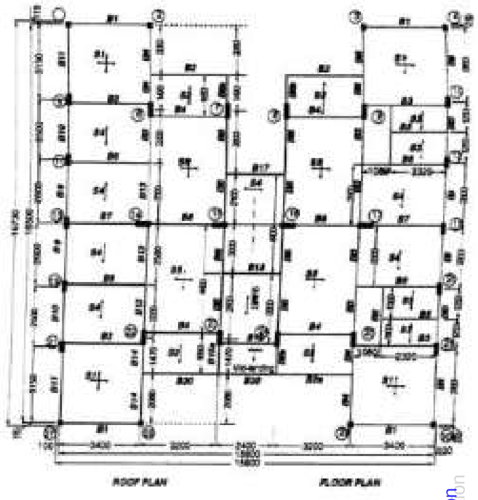
The details of End Shears from Floor slabs have been shown in *Fig. 9.4.2*

Note : *At ground floor the first flight of stairs has 11 risers while the second flight has only 7 risers and subsequently each flight has 9 risers. Thus, the length of landing slab will be different at different levels. All these cases are combined together by taking inclined slab and weight of steps over the span of 2.9m. In fact, the load on landing slab will be less than the load on the waist slab because of absence of steps and weight of horizontal slab instead of inclined slab. However, load on landing slab is considered to be the same as that of sloping portion for simplicity and to err on the safer side. Also the load transferred to the roof beam B18 will be from flight and the beam will get partially loaded. However the beam is designed to carry full load for computational simplicity.*





DETAILS OF FLOOR SLAB S1



Schedule of R.C.C. Slab

a) Roof Slabs

Slab No.	Depth	Short span steel	Long span steel	Remarks
S1	100	#8 @225	#8@200	Two way
S2	100	#8 @225	ø6@180	One way
S4	100	#8 @225	ø6@180	One way
S5	100	#8 @220	#8@200	Two way

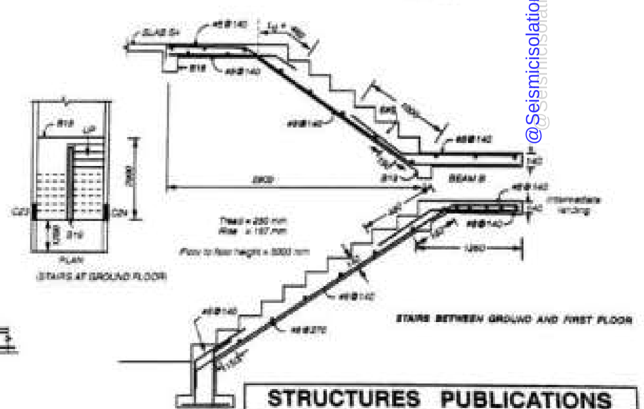
b) Floor Slabs :

Slab No.	Depth	Short span steel	Long span steel	Remarks
S1	120	#8 @280	#8@280	Two way
S2	100	#8 @225	ø6@180	One way
S3	100	#8 @225	ø6@180	One way
S4	100	#8 @225	ø6@180	One way
S5	100	#8 @220	#8@200	Two way

Note : Detailing applicable for continuous slab of approximately equal spans not differing more than 15% of the larger

Typical Details of Bar bending for Slabs

Notes :
1. Grade of Concrete M20
2. Grade of Main steel Fe415 denoted by # and Distribution steel Fe250 Mild denoted by ø
3. Environmental Exposure Condition Mild Nominal cover for slab 20 mm
4. Fire rating considered is for 1 hour maximum.



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S. NO.		DRG. BY	
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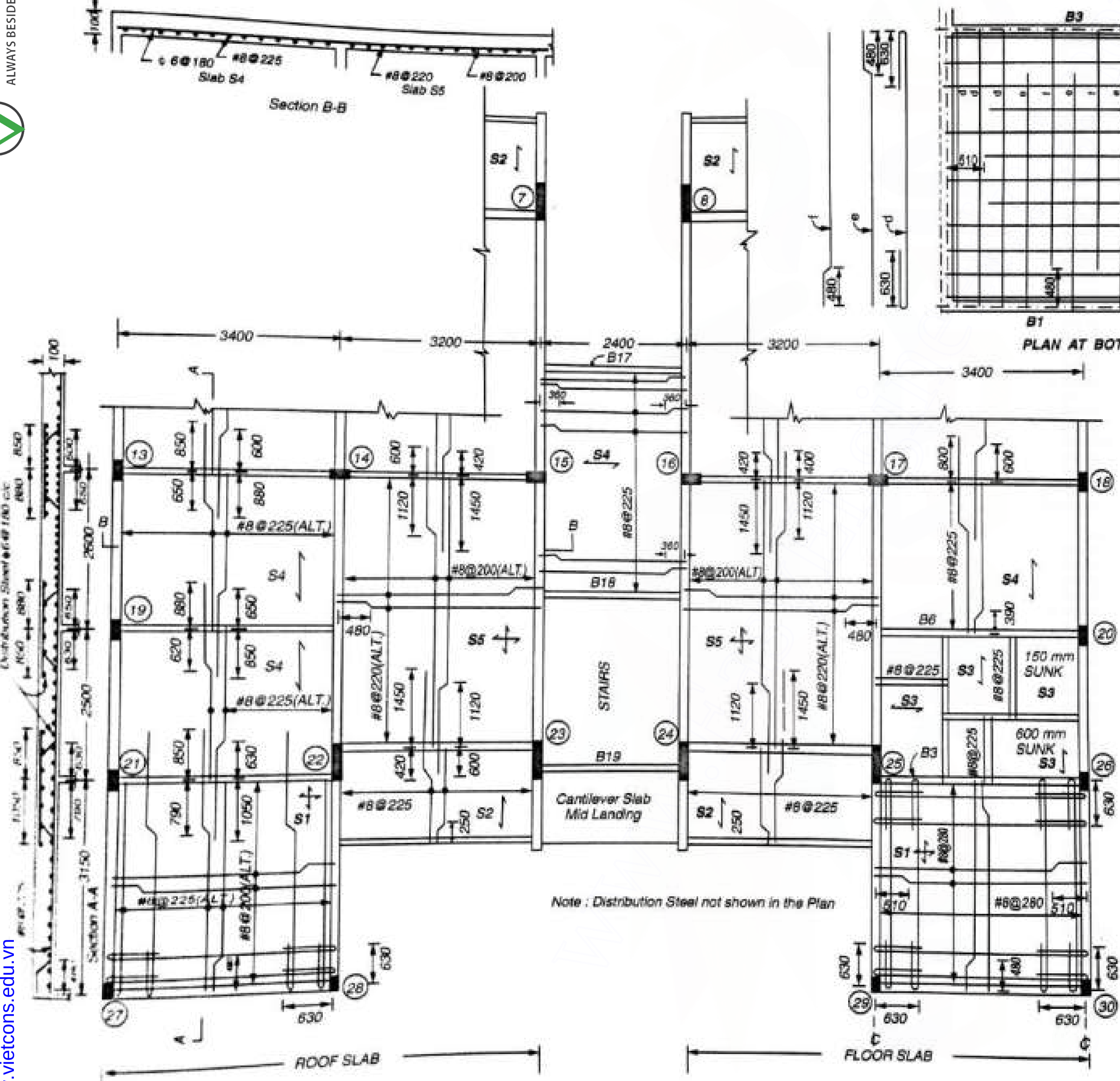
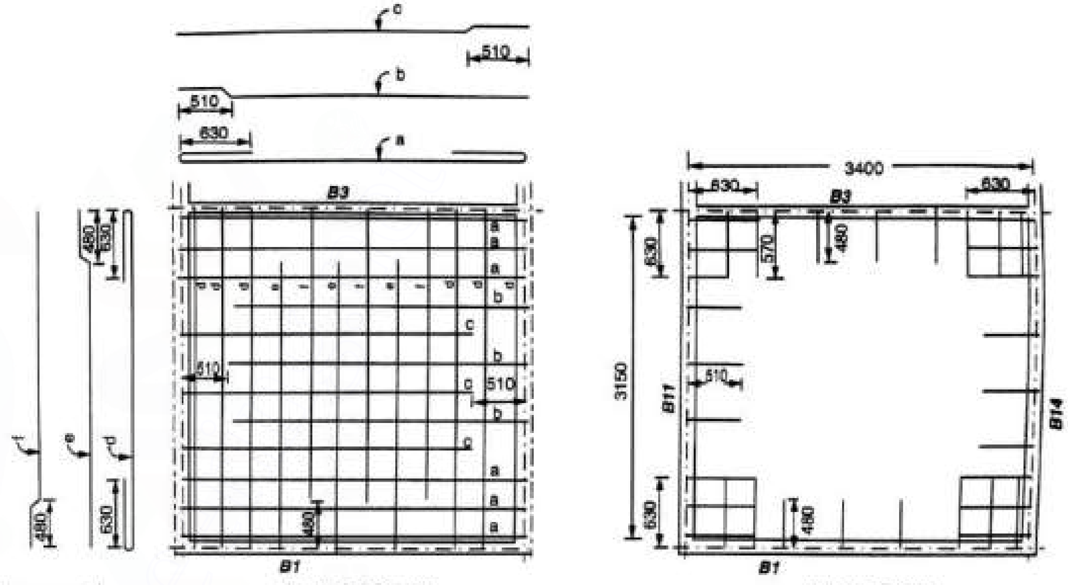




DETAILS OF FLOOR SLAB S1

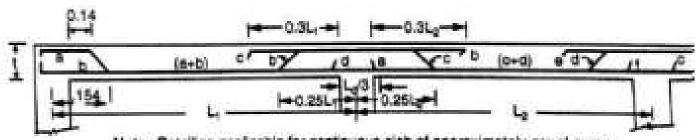
Schedule of R.C.C. Slab

a) Roof Slabs

Slab No.	Depth	Short span steel	Long span steel	Remarks
S1	100	#8 @225	#8@200	Two way
S2	100	#8 @225	φ6@180	One way
S4	100	#8 @225	φ6@180	One way
S5	100	#8 @220	#8@200	Two way

b) Floor Slabs :

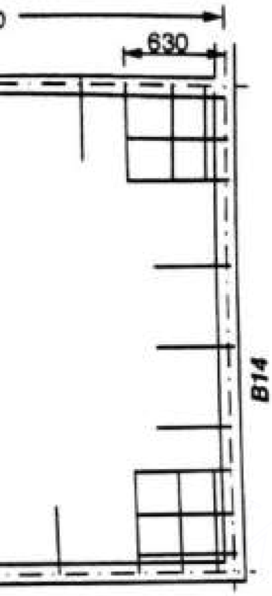
S1	120	#8 @280	#8@280	Two way
S2	100	#8 @225	φ6@180	One way
S3	100	#8 @225	φ6@180	One way
S4	100	#8 @225	φ6@180	One way
S5	100	#8 @220	#8@200	Two way



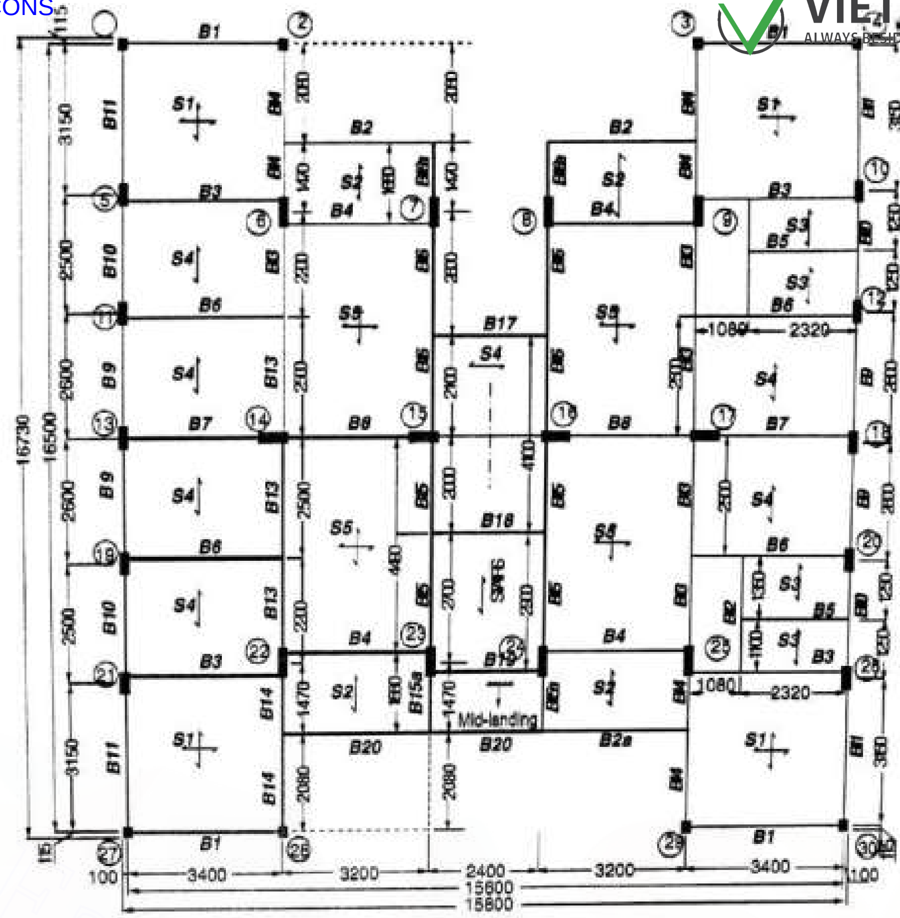
Note : Detailing applicable for continuous slab of approximately equal spans not differing more than 15% of the larger

Typical Details of Bar bending for Slabs

Notes :
1. Grade of Concrete M20
2. Grade of Main steel Fe415 denoted by # and Distribution steel Fe250 Mild denoted by φ
3. Environmental Exposure Condition Mild Nominal cover for slab 20 mm
4. Fire rating considered is for 1 hour maximum.



AT TOP

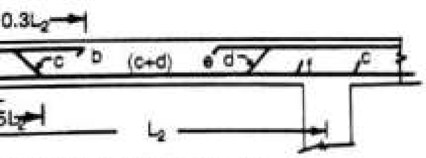


ROOF PLAN

ab

an steel	Remarks
200	Two way
180	One way
180	One way
200	Two way

260	Two way
180	One way
180	One way
180	One way
200	Two way



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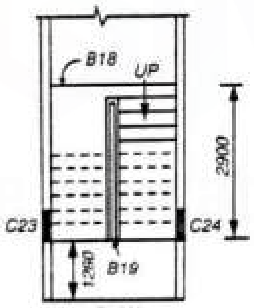
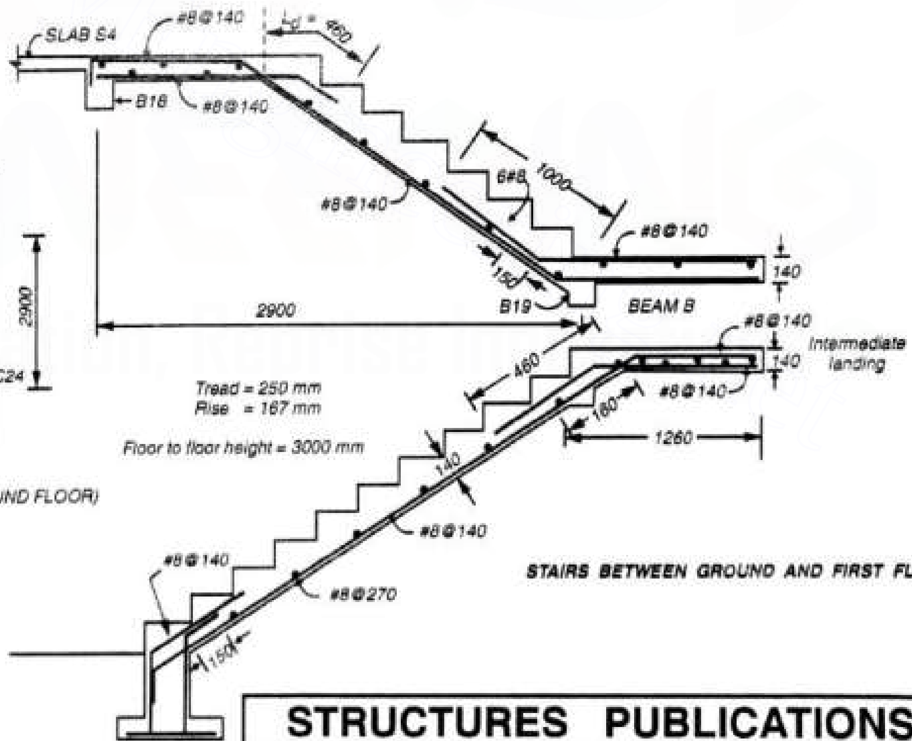


PLAN
(STAIRS AT GROUND FLOOR)



STAIRS BETWEEN GROUND AND FIRST FLOOR

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NOTES :

1. For One - Way Slab :
   (a) Values without parenthesis represent equivalent UDL in kN/m transferred to beams.
   (b) Values in rectangular parenthesis represent part of the UDL in kN/m transferred to beam.
2. For Two-Way Slab :
   (a) Values without parenthesis represent equivalent UDL in kN/m for shear transferred to beam.
   (b) Values in circular parenthesis represent equivalent UDL in kN/m for bending moment transferred to beams.

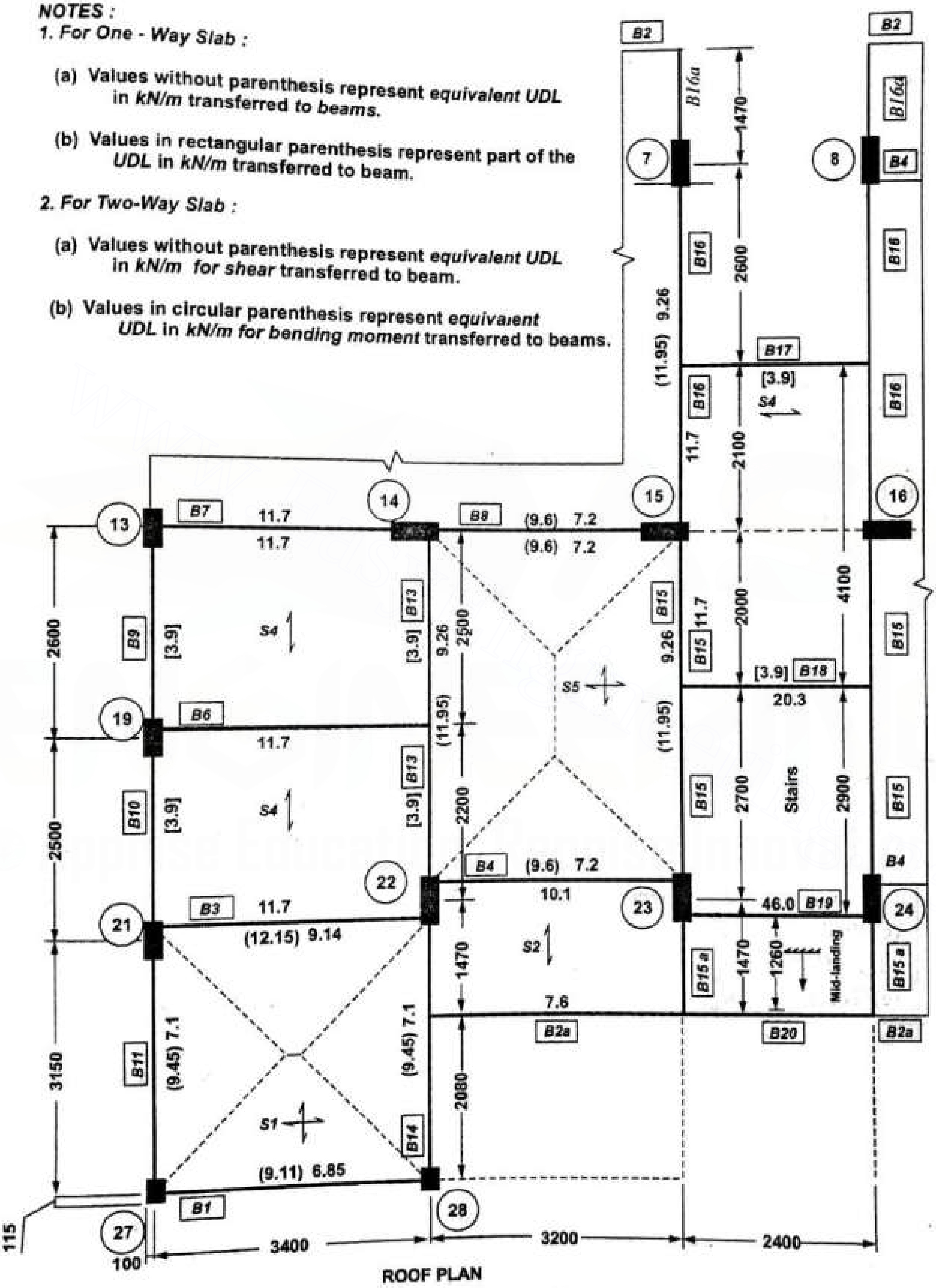


Fig. 9.4.2 Plan Showing Roof Slab End Shear Transferred to Beams

9.5 DESIGN OF BEAMS

9.5.1. Roof Beams

Categorisation of Beams :

Category : I - Simply supported - UDL		*II - One ends S.S.and other continuous* UDL		*III - Both : end continuous* UDL		*IV Miscellaneous*		
Beam	*Span*	*Beam*	*Span*	*Beam*	*Span*	*Beam*	*Span*	*Cross beam*
B1	3.4	B6	3.4	B7	3.4	B9	2.6	B13	4.7	(B6)
B2	3.2	B17	2.4	B8	3.2	B10	2.5	B14	3.55	(B2)
B3	3.4	B18	2.4	B11	3.15	B20	2.4	B15a-15	1.47/4.7	(B2a,B20,B18)
B4	3.2	B19	2.4	B2a	3.2			B16-16a	4.7/1.47	(B17,B2)
Beams *B5* and *B12* are not provided at roof level.										

(a) *Design of Beams of Category I : Simply Supported at both ends - UD Load.*

Beams have not been taken serially but in the descending order of span and loading so that heavier beams are designed first.

The left quarter of the symmetrical structure, shown by dark lines in *Fig. 9.4.1* is designed.

Reference to slab (*viz.* left *or* right) is given looking the plan along the decreasing direction of columns (*i.e.* from below and from right hand side of plan.)

1.	*Beam Mark*			*B3*	*B6*	*B1*	*B4*	*B2*	*B19**	*B18*	*B17*
2.	*Span*	*L*	*m.*	3.4	3.4	3.4	3.2	3.2	2.4	2.4	2.4
3.	*Section*										
	Width	b_w	*mm*	200	200	200	200	200	200	200	200
	Depth	D	*mm*	300	300	300	300	300	300	300	300
	Flange thickness	D_f	*mm*	100	100	100	100	100	140	100	100
4.	*Equivalent UDL for B.M*										
	(a) Slab Right			*S4*	*S4*	*S1*	*S5*	*S2*	*Stair*	*S4*	-
	Load	w_{u1}	*kN/m*	11.70	11.70	9.11	9.60	7.6	46.0	3.90	-
	(b) Slab Left			*S1*	*S4*	-	*S2*	-	-	*Stair*	*S4*
	Load	w_{u2}	*kN/m*	12.15	11.7	-	10.10	-	-	20.3	3.90
	(c) Wall	w_{uw}	*kN/m*	-	-	3.00	-	5.10	-	13.50	3.00
	(d) Self	w_{us}	*kN/m*	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	(e) **Total**	w_u	***kN/m***	**25.35**	**24.9**	**13.61**	**21.20**	**14.20**	**47.5**	**39.20**	**8.4**
5.	*Maximum Moment*										
	$M_{u.max} = w_u L^2/8$		*kN.m*	36.6	36.00	19.7	27.1	18.2	34.20	28.2	6.0
6.	*Section at Mid-span*			*T*	*T*	*L*	*T*	*L*	*L*	*R*	*R*
	$b_f = L_o/6 + 6\,D_f + b_w$			1367	1367	-	1333	-	-	-	-
	$b_f = L_o/12 + 3\,D_f + b_w$			-	-	783	-	767	820	200	200
	b_f / b_w			6.8	6.8	3.9	6.7	3.8	4.1	1.0	1.00
7.	*Main Steel :*										
	Top		*N - # mm*	2-10	2-10	2-10	2-10	2-10	2-10	2-10	2-10
	Bottom bent		*N - # mm*	-	-		1-10		-	-	-
	Bottom straight		*N - # mm*	2-16	2-16	2-12	2-12	2-12	2-16	3-12	2-10

Design of Roof Beams : Category - 1 Continued ...

	Beam Mark	B3	B6	B1	B4	B2	B19*	B18	B17
	A_{st} provided mm^2 Effective Depth d mm M_{ur} provided $kN.m$	402 266 37.70	402 266 37.70	226 268 21.38	305 268 28.94	226 268 21.37	402 266 37.1	339 268 28.5	157 269 14.32
8.	*Equivalent UDL for Shear :* (a) Slab Right Load w_{u1} kN/m (b) Slab Left Load w_{u2} kN/m (c) Wall w_{uw} kN/m (d) Self w_{us} kN/m (e) **Total** w_{ue} kN/m	 S4 11.70 S1 9.14 - 1.5 **22.34**	 S4 11.70 S4 11.7 - 1.5 **24.9**	 S1 6.85 - - 3.00 1.5 **11.35**	 S5 7.20 S2 10.10 - 1.5 **18.80**	 S2 7.6 - - 5.10 1.5 **14.20**	 Stair 46.0 - - - 1.5 **47.5**	 S4 3.9 Stair 20.3 13.50 1.5 **39.20**	 - - S4 3.9@ 3.00 1.5 **8.40**
9.	*Design for Shear* (a) $V_{u.max} = 1/2\ w_{ue}\ L$ kN (b) A_{st1} $N1$- # mm (c) $V_{ur.min}$ kN (d) V_{uD} kN (e) V_{uc} kN (f) $V_{us} = V_{uD} - V_{uc}$ kN (g) Des. Stirr. # - s mm (h) Min. Stirr. ϕ - s mm	 38.00 2-16 51.16 - - - – ϕ6-150	 42.33 2-16 51.16 - - - – ϕ6-150	 19.30 2-12 45.39 - - - – ϕ6-150	 30.10 2-12 45.39 - - - – ϕ6-150	 22.72 2-12 45.39 - - - – ϕ6-150	 57.00 2-16 51.16 39.61 - - – ϕ6-150	 47.00 3-12 49.57 - - - – ϕ6-150	 10.10 2-10 42.16 - - - – ϕ6-150
10.	*Load transferred to column at each end*	*C21&C22* *38.00*	*B13&C19* *42.33*	*C27&C28* *19.30*	*C22&C23* *30.10*	*B16a* *22.72*	*C23&C24* *57.0*	*B15* *@42.32*	*B16* *@5.42*

Explanatory Notes : *(Reference number corresponds to the serial numbers given in above Tables.)*

4. The slab loads are slab end shears given in *Fig. 9.4.2*

4(c) Load due to *RCC* balcony parapet 80 *mm* thick 1.7 *m* high = 3 x 1.7 = 5.1 *kN/m*

7. Top straight bars are only anchor bars are normally 2 Nos. #10*mm* for light beams and 2 Nos. #12 for heavy beams. In practice, anchor bar diameter is kept lower to the diameter of main bar. 8 *mm* diameter may be used as anchor bars in case of internal light beams. In case of spandrel or edge beam, since one face is exposed, 8 *mm* bars, if provided, are subjected to compressive strain at mid-span, and therefore they have a tendency to buckle between two stirrups which are normally spaced at 200 *mm* for 200 *mm* wide beam. Hence, 8 *mm* bars should not be provided as anchor bars for outer beams.

   Designer has to use his judgement and decide which bars are to be bent up to provide for partial fixity between beam and column. Normally 33% of the mid-span steel should be available at top of support. If top anchor bars are able to provide this percentage, then there is no need of bent up bars.

7. Selection of Number-Diameter bar combination of bars for a given section, ratio of b_f/b_w, and required $M_{u.max}$ is made from *Table F-3B* such that M_{ur} provided is greater than $M_{u.max}$ and for rectangular section from *Table F-2B*.

9.(b) A_{st1} represent the area of bottom straight bars continued up to the support.

9.(c) $V_{ur.min}$ = shear resistance of R.C. member
= shear resistance of concrete V_{uc} corresponding to A_{st1}+ shear resistance of minimum stirrups ($V_{usv.min}$).

The values of V_{uc}, $V_{usv.min}$ and $V_{ur.min}$ are obtained from *Table F-2B*.

This is obtained directly from *Table F-2.*

9.(d) $V_{uD} = V_{u.max} - w_{ue}(b_s/2 + d)$ where, b_s = breadth of support = 200 *mm*

9.(e) V_{uc} = shear resistance of concrete for given A_{stl} obtained from *Table F-2B*

9.(g) Design of stirrups is done using *Table F-7*

9.(h) The spacing of minimum stirrups can be obtained from *Table F-6*. But the maximum spacing shall be limited to 0.75*d* or 300 *mm* whichever is less *i.e.* 200*mm* in this case.

10. @ These loads are getting duplicated, and hence their contribution has not been taken. That is why column loads for *B17* and *B18* are less by 3.9 x 2.4/2 = 4.68 *kN*.
Since beams are safe for deflection and bond the calculations have not been given.

(c) *Design of Roof Beam : Category - II and III :*

	Description			Category -II : One End S.S. and other continuous - UDL				Category - III : Both Ends continuous - UDL		
1.	Beam Mark			B7	B8	B11	B2a	B9	B10	B20
2.	Span	L	m	3.4	3.2	3.15	3.2	2.6	2.5	2.4
3.	Section : Width :	b	mm	200	200	200	200	200	200	200
	Depth	d	mm	300	300	300	300	300	300	300
	Slab thickness	D_f	mm	100	100	100	100	100	100	100
4.	Equivalent UDL forB.M									
	(a) Slab Right			S4	S5	S1	S2	S4	S4	-
	Load	w_{u1}	kN/m	11.70	9.60	9.45	7.60	@3.90	@3.90	-
	(b) Slab Left			S4	S5	-	-	-	-	-
	Load	w_{u2}	kN/m	11.70	9.60	-	-	-	-	-
	(c) Wall	w_{uw}	kN/m	-	-	3.00	5.10	3.00	3.00	3.00
	(d) Self	w_{us}	kN/m	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	(e) **Total**	w_u	**kN/m**	**24.9**	**20.70**	**13.95**	**14.2**	**8.4**	**8.4**	**4.5**
5.	Maximum Moment									
	$M_{u.max}$		kN.m	28.8	21.2	13.84	14.54	4.73	4.37	2.16
6.	Section at Mid-span			T	T	L	L	R	R	R
	L_o/L		mm	0.7	0.7	0.7	0.7	-	-	-
	L_o		mm	2380	2240	2205	2240	-	-	-
	$b_f = (L_o/6+6D_f)+b_w$	T	mm	1197	1173	-	-	-	-	-
	$b_f = (L_o/12+3D_f)+b_w$	L	mm	-	-	683	687	200	200	200
	b_f/b_w			6.0	5.86	3.41	3.43	1.00	1.00	1.00
7.	Main Steel :									
	(a) At Mid-span									
	Top		N - # mm	2-12	2-12	2-10	2-10	2-10	2-10	2-10
	Bottom Bent		N - # mm	2-10	1-10	-	-	-	-	-
	Bottom Straight		N - # mm	2-12	2-12	2-10	2-10	2-10	2-10	2-10
	Provided A_{st}		sq. mm	305	305	157	157	157	157	157
	Effective depth		mm	268	268	269	269	269	269	269
	M_{ur} provided		kN.m	36.15	28.87	14.97	14.97	14.32	14.32	14.32

c) Design of Roof Beam : Category - II and III Continued ...

	Discription	Category - II : One End S.S. and other continuous - UDL				Category - III : Both Ends continuous - UDL		
	Beam Mark	B7	B8	B11	B2a	B9	B10	B20
7.	(b)At Continuous End							
	Top N - #mm	2-10+2-12	1-10+1-12	2-10	2-10	2-10	2-10	2-10
	Bottom N - # mm	2-12	2-12	2-10	2-10	2-10	2-10	2-10
	A_{st} provided mm^2	305	305	157	157	157	157	157
	Effective depth mm	268	268	269	269	269	269	269
	M_{ur} provided kN.m	31.56	25.99	14.32	14.32	14.32	14.32	14.32
8.	Equivalent UDL for Shear							
	(a) Slab Right	S4	S5	S1	S2	S4	S4	-
	Load kN/m	11.7	7.2	7.10	7.60	@3.9	@3.9	-
	(b) Slab Left	S4	S5	-	-	-	-	-
	Load kN/m	11.7	7.2	-	-	-	-	-
	(c) Wall kN/m	-	-	3.00	5.10	3.00	3.00	3.00
	(d) Self kN/m	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	(e) **Total** w_{ue} **kN/m**	**24.90**	**15.9**	**11.6**	**14.2**	**8.40**	**8.40**	**4.5**
9	Design for Shear							
	(a) At Continuous End							
	$V_{u.max}$ kN	50.80	30.53	21.92	27.26	10.92	10.50	5.4
	A_{st1} N-# mm	2-10+2-12	2-12+1-10	2-10	2-10	2-10	2-10	2-10
	$V_{ur.min}$ kN	50.92	48.42	42.16	42.16	42.16	42.16	42.16
	V_{uD} kN	-	-	-	-	-	-	-
	Des. Stirrups ϕ - s mm	-	-	-	-	-	-	-
	Min.stirrups ϕ - s mm	6-150	6-150	6-150	6-150	6-150	6-150	6-150
	(b) At Discontinuous End							
	$V_{u.max}$ kN	38.10	22.90	16.44	20.45	10.92	10.50	5.40
	A_{st1} N-# mm	2-12	2-12	2-10	2-10	2-10	2-10	2-10
	$V_{ur.min}$ kN	45.39	45.39	42.16	42.16	42.16	42.16	42.16
	Des. Stirrups #- s mm	-	-	-	-	-	-	-
	Min.Stirrups ϕ -s mm	6-150	6-150	6-150	6-150	6-150	6-150	6-150
10.	*Load transferred to Column*	*C14*	*C14*	*C21*	*B-15a*	*C19&C13*	*C19&C21*	*B15a*
	(a) Continuous end kN	**50.80**	**30.53**	**21.92**	**27.26**	**@5.85**	**@5.62**	**5.40**
		C13	*C15*	*C27*	*B14*	-	-	-
	(b) Discontinuous end kN	**38.10**	**22.90**	**16.44**	**20.45**	-	-	-

@ These loads are getting duplicated and hence their contribution in column loads has not been taken.
$10.92 - 3.9 \times 2.6/2 = 5.85$ kN for B9, and $10.50 - 3.9 \times 2.5/2 = 5.62$ kN for B10.

Explanatory Notes *(Reference number corresponding to serial numbers given in above Table)*

4.(c) Beam B20 : This beam at roof level will carry grill for height of 2m imposing load 3 kN/m.

7. Main steel is obtained from *Table F-3B* for flange section and from *Table-F-2B* for rectangular section

9. For *Category II* - At discontinuous end, $V_{u.max} = 0.60\ w_u L$, At discontinuous end, $V_{u.max} = 0.45\ w_u L$

For *Category III* - $V_{u.max} = 0.5\ w_u L$ at both ends.

Shear design is carried out using *Table F-6 and F-7*

(d) Category - IV : Miscellaneous Roof Beams

(1) Beam B14 - B13

		B14		B13	
1.	*Beam Mark*	*B14*		*B13*	
2.	*Span* L m	3.55		4.7	
3.	Section : Width b mm	200		200	
	Depth D mm	380		380	
	Depth of Slab D_f mm	100		100	
4.	*Equivalent UDL for B.M.*				
	(a) Slab Right	*S2*		*S5*	
	Load w_{u1} kN/m	-		11.95	
	(b) Slab Left	*S1*		*S4*	
	Load w_{u2} kN/m	9.45		@3.9	
	(c) Wall w_{uw} kN/m	3.00		-	
	(d) Self w_{us} kN/m	2.10		2.10	
	(e) Total Load w_u kN/m	**14.55**		**17.95**	
	(f) Beam Reaction from	***B2a***		***B6***	
	Point Load kN	**20.45**		**42.33**	
	Distance from nearest column	1.47		2.2	
5.	*Maximum Moments* $M_{u.max}$ Value in kN.m	A 14.7 *B14*	B −48.2	+44.4 *B13*	C −61.8
6.	*Section type at mid-span*	F1.	Rect.	F1.	Rect.
	$b_f = L_o/12 + 3D_f + b_w$ mm	707		774	
	b_f / b_w	3.5		3.87	
7.	*Main Steel* : Top N - #	2#12	2#12+2#12	2#12	2-12+2#16(1.5m)
	Bottom N - #	2#12	2#12	2#12+2#12	2#12
	Provided A_{st} mm^2	223	456	456	628
	Effective depth mm	348	348	348	346
	M_{ur} provided kN.m	27.93	49.13	54.89	63.64
8.	*Equivalent UDL for Shear*				
	(a) Slab Right	*S2*		*S5*	
	Load w_{u1} kN/m	-		9.26	
	(b) Slab Left	*S1*		*S4*	
	Load w_{u2} kN/m	7.10		@3.90	
	(c) Wall kN/m	3.00		-	
	(d) Self kN/m	2.1		2.1	
	(e) Total kN/m	**12.2**		**15.26**	
	(f) Beam reaction from	***B2a***		***B6***	
	Point Load kN	**20.45/1.47m**		**42.33/2.2m**	
9.	*Design for Shear*	*AB*	*BA*	*BC*	*CB*
	V_{umax} kN	20.9	52.8	56.1	57.9
	A_{st1} N#	2#12	4#12	4#12	2#12+1#16
	V_{urmin} kN	55.77	64.75	64.75	63.05
	Min. Stirrups φ-s	φ6−150	φ6−150	φ6−150	φ6−150
10.	***Load transferred to column***	***C28***	***C22***		***C14***
		20.9	***93.5(47.2 + 46.3)***		***49.4***

Explanatory Notes to Design Step No.

(5), (8), (9) Calculations of Bending Moment and Shears for Beam *B13 - B14*

Since live load is small compared to dead load, various loading cases need not be considered. The details loading are shown in *Fig. 8.5.(a)*

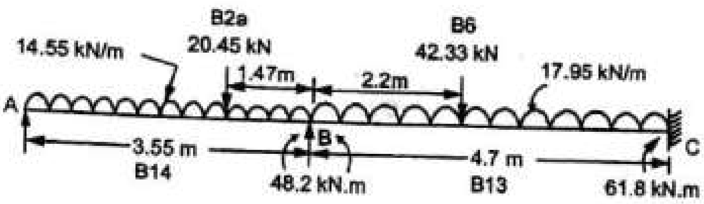


Fig. 9.5.1 Loading for Calculation of Bending Moment Beam B14-B13

M_{FBA} $= 14.55 \times 3.55^2/8 + (20.45 \times 2.08 \times 1.47^2/3.55^2)/2 + 20.45 \times 2.08^2 \times 1.47/3.55^2 = 37.0\ kNm$

M_{FBC} $= 17.95 \times 4.7^2/12 + 42.33 \times 2.2 \times 2.5^2/4.7^2$ $= 59.4\ kN.m$

M_{FCB} $= 17.95 \times 4.7^2/12 + 42.33 \times 2.2^2 \times 2.5/4.7^2$ $= 56.2\ kN.m$

Distribution Factors :

Joint	*Member*	*RSF*	*Sum*	*D.F.*
B	BA	3EI/3.55	1.696EI	0.5
	BC	4EI/4.7		0.5

Distribution Table :

Joint	*A*	*B*		*C*
Member	*AB*	*BA*	*BC*	*CB*
D.F	–	0.5	0.5	–
FEM	0	37.0	−59.4	56.2
Balance	–	11.2	11.2	5.6
Final Moment	0	48.2	−48.2	61.8

Span Moments :

Span AB :

V_{AB} $= 14.55 \times 3.55/2 + 20.45 \times 1.47/3.55 - 48.2/3.55$ $= 20.7\ kN$

x_{max} $=$ $20.7/14.55$ $= 1.423\ m < 2.08\ m$

M_{umax} $= 20.7 \times 1.423/2 - 0$ $= 14.7\ kN.m$

V_{BA} $= 14.55 \times 3.55/2 + 20.45 \times 2.08/3.55 + 48.2/3.55$ $= 51.4\ kN$

V_{BC} $= 17.95 \times 4.7/2 + 42.33 \times 2.5/4.7 - (61.8 - 48.2)/4.7$ $= 61.8\ kN$

V_{CB} $= 17.95 \times 4.7/2 + 42.33 \times 2.2/4.7 + (61.8 - 48.2)/4.7$ $= 64.9\ kN$

x_{max} $= 64.9/17.95 = 3.61\ m$ from $B > 2.5\ m$

∴ Maximum moment occurs under the point load

$M_{u.max}$ $= 64.9 \times 2.5 - 17.95 \times 2.5^2/2 - 61.8$ $= 44.4\ kN.m$

The moments have been entered at *Step No. 5*

Distance of point of inflection from *C* :

$64.9x - 17.95\, x^2/2 - 61.8 = 0$ $\therefore x = 1.122$ from *C*

Actual point of curtailment of 2-#16 from *C* = $1.122 + 0.346$ = say 1.5 *m* from *C*

Distance of point of inflection from *B* towards *C*

$61.8x - 17.95\, x^2/2 - 48.2 = 0$ $\therefore x = 0.91\ m$

7. The details of main steel obtained from *Table F2B* and *Table F3B*

Actual point of curtailment = 0.91 + 0.348 = 1.26 *m* on right and up to concentrated load at distance of 1.47 *m* on left

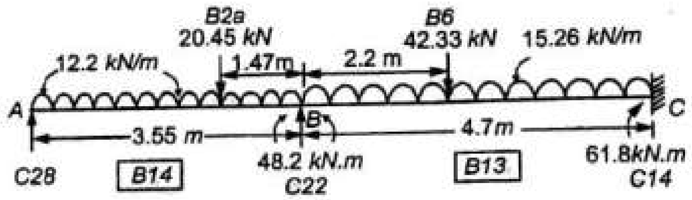


Fig. 9.5.2 Loading for calculation of Shear - Beam B14-B13

V_{AB} = 12.2 x 3.55/2 + 20.45 x 1.47/3.55 – 48.2/3.55 = 16.5 *kN*

V_{BA} = 12.2 x 3.55/2 + 20.45 x 2.08/3.55 + 48.2/3.55 = 47.2 *kN*

V_{BC} = 15.26 x 4.7/2 + 42.33 x 2.5/4.7 – (61.8 – 48.2)/4.7 = 55.48 *kN*

V_{CB} = 15.26 x 4.7/2 + 42.33 x 2.2/4.7 + (61.8 – 48.2)/4.7 = 58.57 *kN*

The shears have been entered at *Step No. 9*

Loads transferred to Columns

Load Transferred to Column *C28* = 16.5 *kN*

Load Transferred to Column C22 = 47.2 + (55.48 – 3.9 x 4.7/2)= 47.2 + 46.3 = 93.5 *kN*

Load Transferred to Column *C14* = 58.57 – 3.9 x 4.7/2 = 49.4 *kN*

These loads have been entered at *Step No. 10*

The details of curtailment of bars have been worked out as per Codal provisions.

The reinforcement details have been shown on Drawing sheet.

***Note :** Slab load of 3.9 kN/m in span BC is duplicated and hence deducted in obtaining column loads.*

(2) Roof Beam B15a - B15 - B16 - B16a

1.	*Beam Mark*	*B15a*	*B15*	*B16*	*B16a*
2.	*Span L m*	1.47	4.7	4.7	1.47
3.	Section :				
	Width *b mm*	200	200	200	200
	Depth *D mm*	*600	*600	*600	*600
	Depth of Slab D_f *mm*	100	100	100	100
4.	*Equivalent UDL for*	*B.M.*			
	(a) Slab Right	**S4*	***S4(2.7m)&S4(2.0m)*	*S4*	-
	Load w_{u1} *kN/m*	11.7	11.7	11.7/(2.1*m*)	-
	(b) Slab Left		*S5*	*S5*	*S2*
	Load w_{u2} *kN/m*		11.95	11.95	-
	(c) Wall w_{uw} *kN/m*	**13.5	**13.5(2.7*m*)	3(2.6*m*)	3.0
	(d) Self w_{us} *kN/m*	3.8	3.8	3.8	3.8
	(e) **Total** *(see Fig.9.5.4)*				
	w_u *kN/m*	**29.0**	**40.95(2.7*m*)/27.45(2*m*)**	**27.45(2.1*m*)/18.75(2.6*m*)**	**6.8**
	(f) **Beam Reaction**				
	from	***B2a+B20***	***B18***	***B17***	***B2***
	Point Load *kN*	**32.66**	**42.32**	**5.42**	**22.72**
	Distance from				
	nearest column *m*	1.47	2.0	2.1	1.47

Beam B15a - B15 - B16 - B16a

			B15a	B15		B16	B16a
5.	*Maximum Moments*		A B	F	C	G	D E
	$M_{u.max}$	kN.m	79.3	+68.9	−71.4	+10.9	−40.7
6.	*Section type at Mid-span*		Rect.	L	Rect.	L	Rect.
	$b_f = L_o/12 + 3D_f + b_w$	mm		774		774	
	b_f/b_w			3.87		3.87	
7.	*Main Steel*						
	Top	N - #	2-12+1-16	2-12	2-12+1-16	2-12	2-12
	Bottom	N - #	2-12	3-12	2-12	2-12	2-12
	Provided A_{st}	mm^2	427	339	427	339	226
	Effective depth	mm	566	568	566	568	568
	M_{ur} provided	kN.m	80.42	68.4	80.42	44.44	44.44
8.	*Equivalent UDL for Shear*						
	(a) Slab Right		*S4*	*S4*	*S4*		
	Load w_{u1}	kN/m	11.7	11.7	11.7(2.1*m*)		-
	(b) Slab Left		*S2*	*S5*	*S5*		*S2*
	Load w_{u2}	kN/m	-	9.26	9.26		-
	(c) Wall	kN/m	**13.5	**13.5(2.7*m*)	*3.0(2.6*m*)		3.0
	(d) Self	kN/m	3.8	3.8	3.8		3.8
	(e) Total	**kN/m**	**29.0**	**38.26/24.76(2*m*)**	**24.76/16.06**		**6.8**
	(f) Beam reaction from		***B2a+B20***	***B18***	***B17***		***B2***
	Point Load	**kN**	**32.66**	**42.32**	**5.42**		**22.72**
	Distance from nearest col.		1.47	2.0	2.1		1.47
9.	*Design for Shear*						
	$V_{u.max}$	kN	75.3 | 103.8		91.3 | 61.5		37.7 | 32.7
	A_{stl}		2#12+1#16		2#12+1#16		2#12
	$V_{ur.min}$		93.61 | 93.61		93.61 | 93.61		82.43 | 82.43
	V_{uD}	kN	- | 78.24				
	Design Stirrups		-		-		-
	Min. Stirrups ϕ-s		6-150 | 6-150		6-150 | 6-150		6-150 | 6-150
10	***Load transferred to column kN***		***C23* 179.1 (75.3 + 103.8)**		***C15* 152.8 (91.3 + 61.5)**		***C7* 70.4 (37.7 + 32.7)**

Explanatory Notes :

*******B15a* to *B15* of length 4.17 *m* (= 1.47 + 2.7) carries wall 2*m* high and cap slab of stair case.

*From *B17* up to the end of *B2* load due to parapet is taken

For a beam *AB* partially loaded by UDL the fixed end moments are given by :

$M_{FAB} = wa^2(3a^2 - 8aL + 6L^2)/12L^2$, $M_{FBA} = wa^3(4L - 3a)/12L^2$ see Fig. 9.5.3 and Table D-1(12)

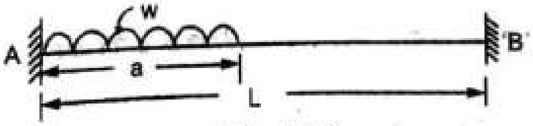


Fig. 9.5.3

The loading beam for calculation of bending moments is shown in *Fig. 9.5.4* as per *Step 4.*

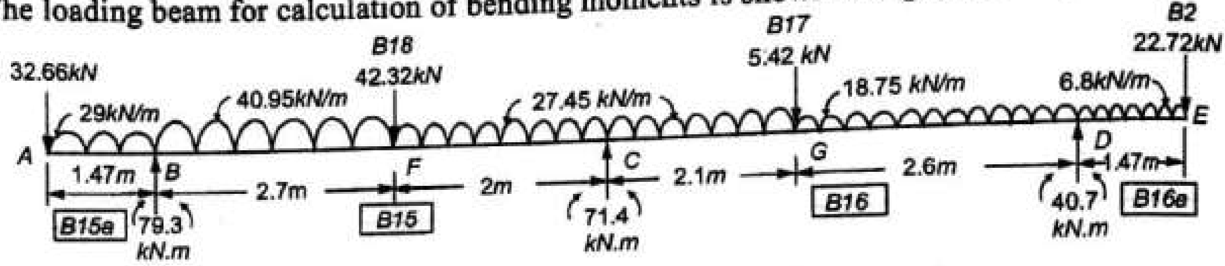


Fig. 9.5.4 Loading for calculation of bending moment

For BF , $w' = (40.95 - 27.45) = 13.50$ *kN/m* , $a = 2.7$ and $L = 4.7$
For CG , $w' = (27.45 - 18.75) = 8.7$ *kN/m* , $a = 2.1$ and $L = 4.7$

Fixed End Moments :

$M_{FBA} = 32.66 \times 1.47 + 29 \times 1.47^2/2$
$= 79.3$ *kN.m*

$M_{FBC} = 42.32 \times 2.7 \times 2^2/4.7^2 + 13.5 \times 2.7^2 \times (3 \times 2.7^2 - 8 \times 2.7 \times 4.7 + 6 \times 4.7^2)/(12 \times 4.7^2) + 27.45 \times 4.7^2/12$
$= 90.84$ *kN.m*

$M_{FCB} = 42.32 \times 2.7^2 \times 2/4.7^2 + 13.5 \times 2.1^3 \times (4 \times 4.7 - 3 \times 2.1)/ (12 \times 4.7^2) + 27.45 \times 4.7^2/12$
$= 89.17$ *kN.m*

$M_{CD} = 5.42 \times 2.1 \times 2.6^2/4.7^2 + 8.7 \times 0.447^2 \times (3 \times 0.447^2 - 8 \times 0.447 + 6) \times 4.7^2/12 + 18.75 \times 4.7^2/12$
$= 47.67$ *kN.m*

$M_{FDC} = 5.42 \times 2.1^2 \times 2.6/4.7^2 + 8.7 \times 0.447^3(4 - 3 \times 0.447) \times 4.7^2/12 + 18.75 \times 4.7^2/12$
$= 41.1$ *kN.m*

$M_{FDE} = 22.72 \times 1.47 + 6.8 \times 1.47^2/2$
$= 40.7$ *kN.m*

Distribution Factors : $dCB = dCD = 0.5$ as spans and end conditions are same.

Moment Distribution	Joint	B		C		D	
	Member	BA	BC	CB	CD	DC	DE
Distribution factors		0	1	0.5	0.5	1	0
Initial Fixed End Moments		79.3	−90.84	89.17	−47.67	41.1	−40.7
Distributed moments						−0.4	
Carry over moments			11.54	5.77	−0.20		
		79.3	−79.3	94.94	−47.87	40.7	−40.7
Distributed moments				−23.54	−23.53		
Final support moments		79.3	−79.3	71.40	−71.4	40.7	−40.7

Span Moments :

$V_{BA} = 32.66 + 29 \times 1.47$
$= 75.3$ *kN*

$V_{BC} = 42.32 \times 2/4.7 + 27.45 \times 4.7/2 + 13.50 \times 2.7 \times (1.35 + 2)/4.7 + (79.3 - 71.4)/4.7$
$= 110.2$ *kN*

$V_{CB} = 42.32 \times 2.7/4.7 + 27.45 \times 4.7/2 + 13.50 \times 2.7^2/(2 \times 4.7) - (79.3 - 71.4)/4.7$
$= 97.6$ *kN*

$V_{CD} = 5.42 \times 2.6/4.7 + 18.75 \times 4.7/2 + 8.7 \times 2.1 \times (1.05 + 2.6) / 4.7 + (71.4 - 40.7)/4.7$
$= 67.8$ *kN*

$V_{DC} = 5.42 \times 2.1/4.7 + 18.75 \times 4.7/2 + 8.7 \times 2.1^2/(2 \times 4.7) - (71.4 - 40.7)/4.7$
$= 44.0$ *kN*

$V_{DE} = 6.8 \times 1.47 + 22.72 = 32.7$ *kN*

Span BC :

x_{max} $= 110.2/40.95 = 2.69\ m < 2.7\ m$

M_{umax} $= 110.2 \times 2.69/2 - 79.3 = 68.9\ kN.m$

Span CD :

x_{max} $= \dfrac{44.00}{18.75} = 2.347\ m < 2.6\ m$

M_{max} $= 44 \times 2.347/2 - 40.7 = 10.9\ kN.m$

The moments have been entered at *Step - 5* and Main steel entered in *Step - 7* from *Table F-2B* and *Table F-3B*.

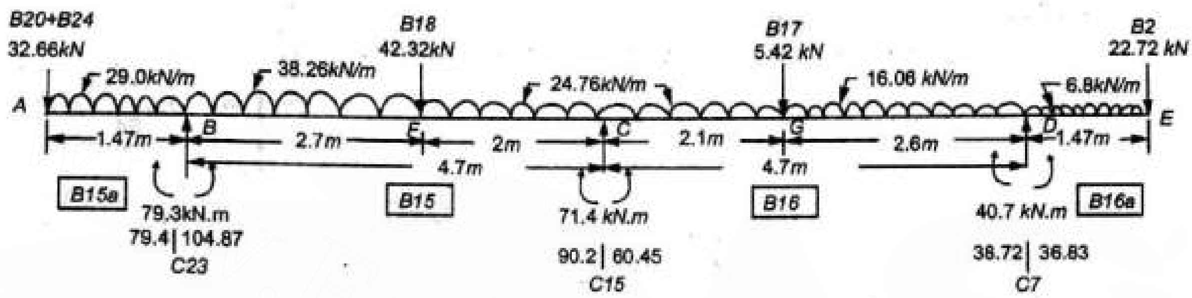


Fig. 9.5.5 Loading for Calculation of Shear Beam 15a-15-16-16a

Beam Shear :

The loading diagram is shown in *Fig. 9.5.5* as per *Step - 9*

V_{BA} $= 32.66 + 29 \times 1.47 = 75.3\ kN$

V_{BC} $= 38.26 \times 2.7 \times (2.7/2 + 2)/4.7 + 42.32 \times 2/4.7 + (79.3 - 71.4)/4.7 + 24.76 \times 2/4.7$
$= 103.8\ kN$

V_{CB} $= 38.26 \times 2.7\ (2.7/2)/4.7 + 42.32 \times 2.7/4.7 - (79.3 - 71.4)/4.7 + 24.76 \times 2 \times 3.7/4.7$
$= 91.3\ kN$

V_{CD} $= 24.76 \times 2.1 \times (2.1/2 + 2.6)/4.7 + 16.06 \times 2.6^2/(2 \times 4.7) + 5.42 \times 2.6/4.7 + (71.4 - 40.7)/4.7$
$= 61.5\ kN$

V_{DC} $= 24.76 \times 2.1^2/(2 \times 4.7) + 5.42 \times 2.1/4.7 + 16.06 \times 2.6 \times (1.3 + 2.1)/4.7 - (71.4 - 40.7)/4.7$
$= 37.7\ kN$

V_{DE} $= 6.8 \times 1.47 + 22.72$
$= 32.7\ kN$

The values of shear are entered at *Step - 9* and shear design is carried out as per *Table F-2B*

Load Transferred to Column *C23* $= (75.3 + 103.8)$ $= 179.1\ kN$

Load Transferred to Column *C15* $= 91.3 + 61.5$ $= 152.8\ kN$

Load Transferred to Column *C7* $= 37.7 + 32.7$ $= 70.4\ kN$

These details are entered at *Step - 5*

The details of curtailment of bars have been worked out as per Codal provisions.
The reinforcement details have been shown on Drawing sheet.

NOTES :

1. For One - Way Slab :

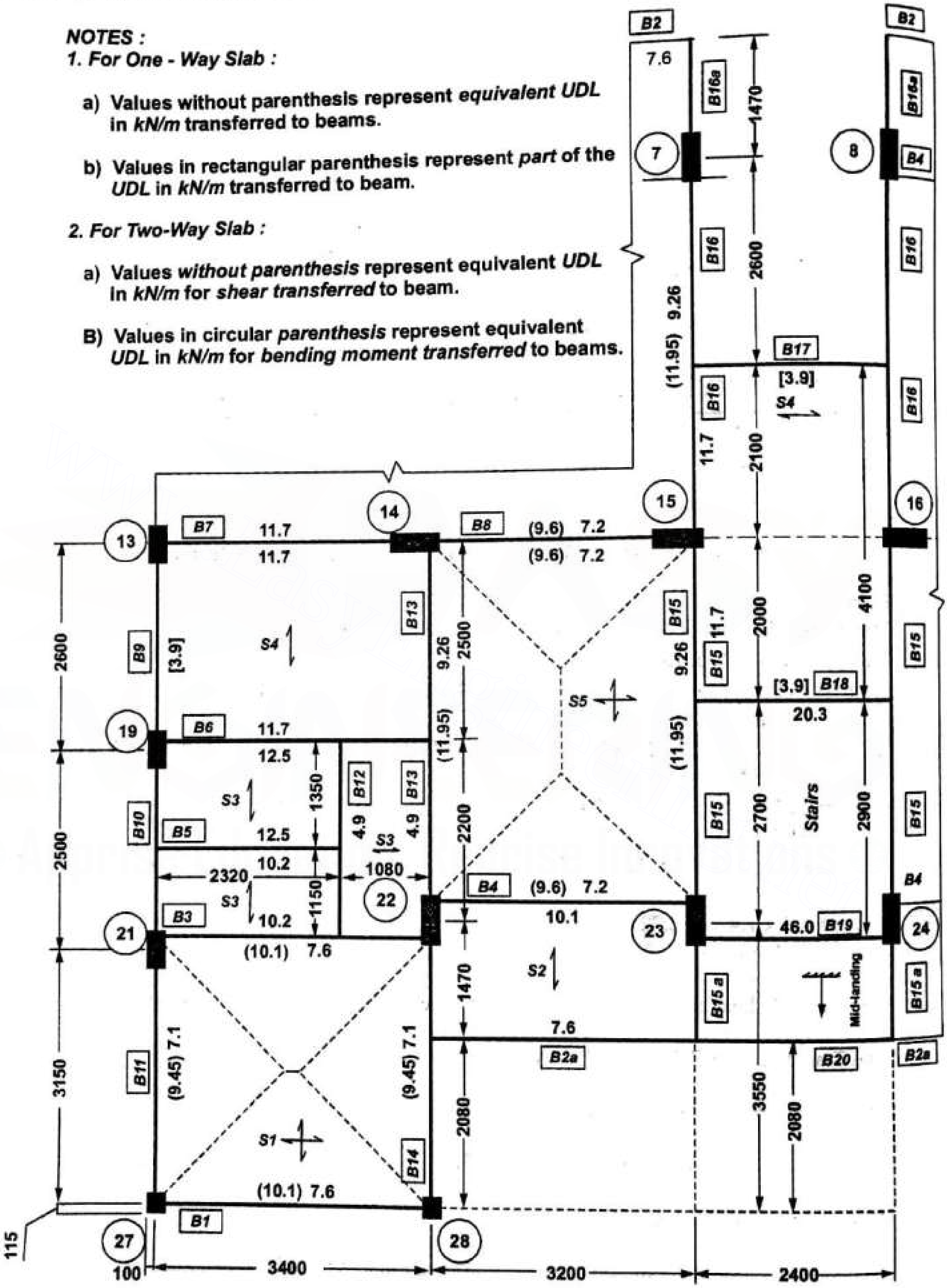
a) Values without parenthesis represent *equivalent UDL* in *kN/m* transferred to beams.

b) Values in rectangular parenthesis represent *part* of the *UDL* in *kN/m* transferred to beam.

2. For Two-Way Slab :

a) Values *without parenthesis* represent equivalent *UDL* in *kN/m* for *shear transferred* to beam.

B) Values in circular *parenthesis* represent equivalent *UDL* in *kN/m* for *bending moment transferred* to beams.



FLOOR PLAN

Fig. 9.5.5 Plan Showing Floor Slab End Shear Transferred to Beams

9.5.2 Floor Beams

(A) Categorisation and Grouping of Beams

Category - I (a) Simply supported (SS) at both ends-UDL				*Category - I(b)* S.S. at both ends UDL& Point Load			*Category -II* SS at one end and Continuous at other - UDL		*Category - III* Continuous at both ends		*Category - IV* Miscellaneous		
Beam	*Span*	Beam	*Span*	Beam	*Span*	Cross	Beam	*Span*	Beam	*Span*	Beam	*Span*	Cross
B1	3.4	*B11*	3.15	*B3*	3.4	*(B12)*	*B7*	3.4	*B20*	2.4	*B13*	4.7	*(B6)*
B2	3.2	*B17*	2.4	*B6*	3.4	*(B12)*	*B8*	3.2			*B14*	3.35	*(B2a)*
B4	3.2	*B18*	2.4	*B10**	2.5	*(B5)*	*B9*	2.5			*B15a*	1.47	*(B2a,20)*
B5#	2.32	*B19*	2.4	*B12*	2.5	*(B5)*	*B2a*	3.2			*B15*	4.7	*(B18)*
											B16	4.7	*(B17)*
											B16a	1.47	*(B2)*

***Note :** *Slabs S3 for W.C. and bathroom both, are at different levels and sunk.*

*Beam *B10* is provided inverted (*i.e.* above-floor level) to facilitate taking outlet for drainage. Therefore, there will be no continuity between *B9, B10, B11*. End conditions of these beams will be different from those of roof beams.

#Beam *B5* will be provided at floor level. Depths of beams *B5, B6* for the part where the latrine seat is provided is taken equal to 600 *mm* while for rest of its length, it will be 450 *mm*. However, the beam is designed assuming $D = 450mm$. The minor loading differences are ignored.

Now-a-days commodes are provided in toilets and hence there is no need to provide beams of different depths or even sunk the floor for laterine.

(B) Design of Floor Beams

Category - I(a) :Simply Supported Floor Beam : UDL only

Beams have not been taken serially but in descending order of span and loading so that heavier beams are designed first.

					B4	*B1*	*B11*	*B2*	*B19*	*B18*	*B17*	*B5*
1.	*Beam mark*				*B4*	*B1*	*B11*	*B2*	*B19*	*B18*	*B17*	*B5*
2.	*Span*		L	*m*	3.2	3.4	3.15	3.2	2.4	2.4	2.4	2.32
3.	*Section*	Width	b	*mm*	200	200	200	200	200	200	200	200
		Depth	D	*mm*	380	380	380	380	380	380	380	450
	Depth of Slab		D_f	*mm*	100	120	120	100	140	100	-	-
4.	*Equivalent UDL for B.M.*											
	(a) Slab Right				*S5*	*S1*	*S1*	-	*Stair*	*S4*	-	*S3*
	Load		w_{u1}	*kN/m*	9.60	10.1	9.45	-	46.0	@3.9	-	12.50
	(b) Slab Left				*S2*	-	-	*S2*	-	*Stair*	*S4*	*S3*
	Load		w_{u2}	*kN/m*	10.1	-	-	7.60	-	20.3	@3.9	10.2
	(c) Wall		w_{uw}	*kN/m*	18.2	18.2	18.2	5.10	-	-	3.0	7.5
	(d) Self		w_{us}	*kN/m*	2.1	2.1	2.1	2.1	2.1	2.1	2.1	3.4
	(e) Total		w_u	***kN/m***	**40.0**	**30.4**	**29.75**	**14.8**	**48.1**	**26.3**	**9.0**	**33.6**
5.	*Maximum Moments* $M_{u.max}$			*kN/m*	51.2	43.92	36.9	18.9	34.6	18.9	6.5	22.6

Category - I(a) :Simply Supported Floor Beam : UDL only continued ...

Step	Beam mark	B4	B1	B11	B2	B19	B18	B17	B5
6.	Section at Mid-span	T	L	L	L	L	L	R	R
	$b_f = (L_o/6 + 6D_f) + b_w$ mm	1333	-	-	-	-	-	-	-
	$b_f = (L_o/12 + 3D_f) + b_w$ mm	-	843	823	767	820	700	200	200
	b_f / b_w	6.7	4.2	4.1	3.8	4.1	3.5	1.00	1.00
7.	*Main Steel*								
	Top N-# mm	2-10	2-10	2-10	2-10	2-10	2-10	2-10	2-10
	Bottom Bent N-# mm	1-12	2-10	1-10	-	1-10	-	-	-
	Bottom Straight N-# mm	3-12	2-12	2-12	2-10	2-12	2-10	2-10	2-12
	M_{ur} Provided kN.m	55.6	46.80	37.4	19.53	37.4	19.51	18.86	32.20
	A_{st} Provided sq.mm	452	383	305	157	305	157	157	270
	Effective Depth mm	348	348	348	349	348	349	349	418
8.	*Equivalent UDL for Shear :*								
	(a) Slab Right	S5	S1	S1	-	Stair	S4	-	S3
	Load kN/m	7.20	7.6	7.10	-	46.0	@3.9	-	12.50
	(b)Slab Left kN/m	S2	-	-	S2	-	Stair	S4	S3
	Load kN/m	10.1	-	-	7.6	-	20.3	@3.9	10.2
	(c)Wall kN/m	18.2	18.2	18.2	5.1	-	-	3.00	7.5
	(d)Self kN/m	2.1	2.1	2.1	2.1	2.1	2.1	2.1	3.4
	(f) **Total** kN/m	**37.6**	**27.9**	**27.4**	**14.8**	**48.1**	**26.3**	**9.0**	**33.6**
9.	*Shear :*								
	$V_{u.max}$ kN	60.2	47.4	43.2	23.7	57.7	31.6	10.8	39.0
	A_{st1} N1-# mm	3-12	2-12	2-12	2-10	2-12	2-10	2-10	2-12
	$V_{ur.min}$ kN	60.81	55.77	55.77	51.89	55.77	51.89	51.89	64.50
	Min. Stirrups N-ϕ mm	6-150	6-150	6-150	6-150	6-150	6-150	6-150	6-150
10.	***Load transferred to Columns at each end*** **kN**	***C22&23*** ***60.2***	***C27&28*** ***47.4***	***C21,C27*** ***43.2***	***B16a*** ***23.7***	***C23&24*** ***57.7***	***B15*** ***26.9***	***B16*** ***5.4***	***B10,B12*** ***39.0***

@ Column load = $V_{u.max} - 3.9 \times 2.4/2$

Step	Category	*I(b) S.S. UD Load and Point Load*				*II One end S.S and Other Conti.- UDL*				*III Both ends Conti.- UDL*
1.	*Beam Mark*	*B6*	*B3*	*B10*	*B12*	*B7*	*B8*	*B9*	*B2a*	*B20*
2.	*Span* L m	3.4	3.4	2.5	2.5	3.4	3.2	2.6	3.2	2.4
3.	*Section :*									
	Width b mm 200	200	200	200	200	200	200	200	200	200
	Depth D mm	380	450	380	380	380	380	380	380	380
	Depth of Slab D_f mm	100	120	100	100	100	100	100	100	100
4.	*Load :*									
	(a) Slab Right	S4	S3	-	S3	S4	S5	S4	S2	-
	Load w_{u1} kN/m	11.7	10.2	-	4.9	11.7	9.60	@3.9	7.6	-

Category of Floor Beam Ia, II, II, III - continued.....

	Beam Mark		B6	B3	B10	B12	B7	B8	B9	B2a	B20
	(b) Slab Left :		S3	S1	-	-	S4	S5	-	-	-
	Load	w_{u2} kN/m	12.5	10.1	-	-	11.7	9.6	-	-	-
	(c) Wall	w_{u2} kN/m	10.0	10.0	18.2	7.5	10.0	10.0	18.2	5.1	3.0
	(d) Self	w_{us} kN/m	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.1
	(e) Total	w_u **kN/m**	**36.3**	**32.4**	**20.3**	**14.5**	**35.5**	**31.3**	**24.2**	**14.8**	**5.1**
	(f) Supported Beam		**B12**	**B12**	**B5**	**B5**	-	-	-	-	-
	Beam Shear	**kN**	**36.0**	**39.2**	**39.0**	**39.0**	-	-	-	-	-
	Distance from nearest support		1.08	1.08	1.15	1.15	-	-	-	-	-
5.	*Maximum Moment*										
	$M_{u.max}$	kN.m	73.6	70.2	40.0	35.5	41.0	32.0	16.4	15.2	2.5
6.	*Section at Mid-span*		L	L	R	L	T	T	R	L	R
	$b_f = (L_o/6+6D_f)+b_w$	mm	-	-	-	-	1196	1173	-	-	-
	$b_f = (L_o/12+3D_f)+b_w$	mm	698	758	200	640	-	-	200	686	200
	b_f/b_w		3.5	3.8	1.00	3.2	6.0	5.9	1.00	3.4	1.00
7.	*Main Steel :*										
	Top St.	N-#mm	2-12	2-12	2-10	2-10	2-12	2-12	2-10	2-10	2-10
	Bottom Bent	N-# mm	1-12	1-12	2-10	1-10	2-10	1-12	-	-	-
	Bottom Straight	N-# mm	2-16 +1-12	2-16 +1-12	2-12	2-12	2-12	2-12	2-10	2-10	2-10
	M_{ur} Provided	kN.m	74.1	74.7	42.6	37.2	47.2	41.8	18.86	19.5	18.86
	Provided A_{st}	sq. mm	628	628	427	304	383	339	157	157	157
	Effective Depth	mm	346	346	348	348	348	348	349	349	349
	(b) At Continuous End		-	-	-	-	2-12	1-12	-	-	-
	Top	N-# mm	-	-	-	-	+2-10	+2-12	2-10	2-10	2-10
	Bottom	N-# mm	-	-	-	-	2-12	2-12	2-10	2-10	2-10
	M_{ur} Provided	kN.m	-	-	-	-	42.62	38.29	18.86	18.86	18.86
	A_{st} Provided	sq. mm	-	-	-	-	383	339	157	157	157
	Effective Depth	mm	-	-	-	-	348	348	349	349	349
8.	*Equivalent UDL for Shear*										
	(a) Slab Right		S4	S3	-	S3	S4	S5	S4	S2	-
	Load	kN/m	11.7	10.2	-	4.9	11.7	7.2	@3.9	7.6	-
	(b) Slab Left		S3	S1	-	-	S4	S5	S4	S2	-
	Load	kN/m	12.5	7.6	-	-	11.7	7.2	-	-	-
	(c) Wall	kN/m	10.0	10.0	18.2	7.5	10.0	10.0	18.2	5.1	3.0
	(d) Self	kN/m	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.1	2.1
	(e) Total	**kN/m**	**36.3**	**29.9**	**20.3**	**14.5**	**35.5**	**26.5**	**24.2**	**14.8**	**5.1**
	(f) Supported beam		**B12**	**B12**	**B5**	**B5**	-	-	-	-	-
	Beam Shear		**36.0**	**39.2**	**39.0**	**39.0**	-	-	-	-	-
	Dist. from nearest support		1.08	1.08	1.15	1.15	-	-	-	-	-
9.	*Shear at continuous end : Right*										
	$V_{u.max}$	kN	86.0	77.6	43.3	36.0	72.4	50.9	37.8	28.4	6.2
	A_{st1}	N1-# mm	2-16 +1-12	2-16 +1-12	2-12	2-12	2-12 +2-10	2-12 +1-12	2-10	2-10	2-10

Category of Beam Ia, II, II - continued.....

Step	Beam Mark		B6	B3	B10	B12	B7	B8	B9	B2a	B20
	$V_{ur.min}$	kN	66.32	76.58	55.7	55.7	62.44	60.8	51.89	51.89	51.89
	V_{uD}	kN	69.8	64.3	-	-	56.5	-	-	-	-
	V_{uc}	kN	38.64	-	-	-	-	-	-	-	-
	V_{us}	kN	47.36	-	-	-	-	-	-	-	-
	Des. Stirrups	ϕ - mm	#8@250	-	-	-	-	-	-	-	-
	L_{s1}	mm	540	-	-	-	-	-	-	-	-
	Min. Stirrups	ϕ - s mm	6-150	6-150	6-150	6-150	6-150	6-150	6-150	6-150	6-150
	(b) At Discontinuous End : Left										
	$V_{u.max}$	kN	73.0	63.3	46.5	39.2	54.3	-	28.3	21.3	6.2
	A_{st1}	N1-# mm	1-12 +2-16	1-12 +2-16	2-12	2-12	2-12	2-12	2-10	2-10	2-10
	$V_{ur.min}$	kN	66.32	76.58	55.77	55.77	55.77	55.77	51.89	51.89	51.89
	V_{uD}	kN	56.8	-	-	-	-	-	-	-	-
	Min. Stirrups	ϕ - s mm	6-150	6-150	6-150	6-150	6-150	6-150	6-150	6-150	6-150
10.	***Load Transferred to Column -Continuous end (Right end)***		***B13***	***C22***	***C19***	***B6***	***C14***	***C15***	***C13***	***B15a***	***B15***
	Load in	***kN***	***86.0***	***77.6***	***43.3***	***36.0***	***72.4***	***38.2***	***@31.7***	***28.4***	***6.2***
	Discontinuous end (Left end)		***C19***	***C21***	***C21***	***B3***	***C13***	***C14***	***C19***	***B14***	***B15a***
	Load in	***kN***	***73.0***	***63.3***	***46.5***	***39.2***	***54.3***	***50.9***	***@23.7***	***21.3***	***6.2***

Note : *(i) Left or right end of the beam is decided by looking at the plan from below or from right.*
(ii) Bent up bars are not used as shear reinforcement.
(iii) Column load on C13 = (18.2 + 2.1) x 2.6 x 0.6 = 31.7 kN
Column load on C19 = (18.2 + 2.1) x 2.6 x 0.45 = 23.7 kN

Category - IV Miscellaneous Floor Beams (i) Beam B14 - B13

1. Beam Mark			B14	B13
2. Span	L	m	3.55	4.7
3. Section :				
Width	b	mm	200	200
Depth	D	mm	380	380
Depth of Slab	D_f	mm	120	100
4. Loads :				
(a) Slab Right				S5
Load	w_{u1}	kN/m	-	11.95
(b) Slab Left			S1	S3
Load	w_{u2}	kN/m	9.45	4.9
(c) Wall	w_{us}	kN/m	18.2	10.0
(d) Self	w_{us}	kN/m	2.1	2.1
(e) **Total**	w_u	**kN/m**	**29.8**	**28.95**
(f) Pt. Load from beam			B2a	B6
Value		kN	21.3	86.0
Distance from nearest column		m	1.47	2.2

ii

Category - IV Miscellaneous Beams continued....

Beam Mark	B14			B13	
5. *Maximum Moments* $M_{u.max}$	+24.2 kN.m		−84.1 kN.m	+93.7 kN.m	−89.3 kN.m
6. *Section type : at Mid-Span*		L	R	L	R
$b_f = (L_o/12 + 3D_f) + b_w$		767	200	774	200
b_f/b_w		3.8		4.0	1
7. *Main Steel :* Top	2-#12	2#12	3#12 +2#20	2-#12	3#12+2-#20
Bottom	2#12	2#12	2#12	3#12 +3#20	2#12
Provided Steel Top	226	226	968	226	968
Bottom	226	226	339	968	427
M_{ur} Provided kN.m	-	27.9	89.23	103.65	89.23
Effective depth	348	348	324	324	324
8 *Shear*					
(a) Slab Right : Load w_{u1}		-		9.26	
(b) Slab Left w_{u2}		7.1		4.9	
(c) Wall kN/m		18.2		10.0	
(d) Self kN/m		2.1		2.1	
(e) **Total** kN/m		**27.4**		**26.26**	
(f) Point Load from m		*B2*		*B6*	
Dist. from nearest support		1.47		2.2	
9. Design Shear					
$V_{u.max}$ kN	33.8	84.8	106.3		103.1
$V_{ur.min}$	55.77	71.9	71.9		71.9
V_{uD}	-	73.2	95.1		92.0
V_{uc}		46.14	46.14		46.14
$V_{us} = V_{uD} - V_{uc}$	-	27.1	48.96		45.86
V_{us}/d		83.6	151		142
Design stirrups		φ6@140	#8@240		#8@250
L_{s1} m		1.0m	1.3 m		1.2m
Minimum Stirrups	φ6-150		φ6-150		φ6-150
10 *Load transferred to Column* kN	*C28* 33.8		*C22* 191.1 (84.8 + 106.3)		*C14* 103.1

*Spacing limited to 0.75 d

Explanatory Notes : Calculation of Bending Moments and Shear in Beam B-13 and B-14

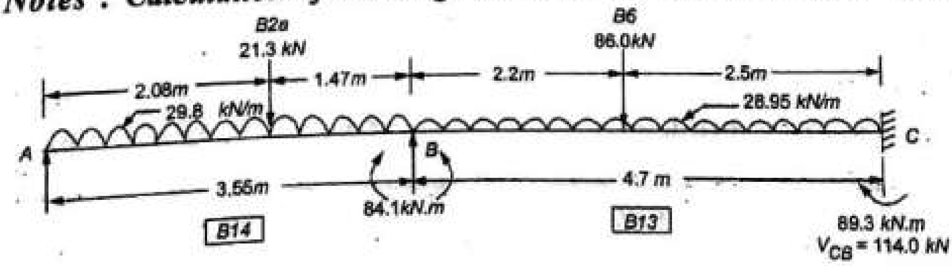


Fig. 9.5.6 Loading for Calculation of Bending Moment

Explanatory Notes Beam - B14 - B13 continued...

The beam has support symmetry and hence end is assumed to be fixed and distribution will be carried out for half the frame. The load of 4.9 *kN/m* transferred from *S3* is assumed over the whole span for computational simplicity.

Joint	Member	R.S.F.	SUM	DF
B	BA	3EI/3.55	1.696EI	0.5
	BC	4EI/4.7		0.5

Fixed End Moment :

M_{FBA} $= 29.8 \times 3.55^2/8 + 21.3 \times 2.08 \times 1.47^2/(2 \times 3.55^2) + 21.3 \times 2.08^2 \times 1.47/3.55^2$
$= 61.4$ *kN.m*

M_{FBC} $= 28.95 \times 4.7^2/12 + 86.0 \times 2.2 \times 2.5^2/4.7^2 = 106.8$ *kN.m*

M_{FCB} $= 28.95 \times 4.7^2/12 + 86.0 \times 2.2^2 \times 2.5/4.7^2 = 100.3$ *kN.m*

Distribution Table

Joint	A	B		C
Member	AB	BA	BC	CB
D. F.	–	1/2	1/2	–
FEM	0	61.4	–106.8	100.3
Balance -	–	22.7	22.7	–
	–	–	–	11.3
Final Moments	0	84.1	–84.1	111.6

Reactions :

V_{AB} $= 29.8 \times 3.55/2 + 21.3 \times 1.47/3.55 - 84.1/3.55 = 38.0$ *kN*

V_{BA} $= 29.8 \times 3.55/2 + 21.3 \times 2.08/3.55 + 84.1/3.55 = 89.0$ *kN*

V_{BC} $= 28.95 \times 4.7/2 + 86.0 \times 2.5/4.7 - (111.6 - 84.1)/4.7 = 108.0$ *kN*

V_{CB} $= 28.95 \times 4.7/2 + 86.0 \times 2.2/4.7 + (111.6 - 84.1)/4.7 = 114$ *kN*

Span Moments

Span AB :From joint A,

x_{max} $= 38.0/29.8 = 1.275\ m < 2.08\ m$

M_{max} $= 38 \times 1.275/2 - 0 = 24.2$ *kN.m*

From joint C

x_{max} $= 114/28.95 = 3.937\ m > 2.5m$ ∴ Maximum B.M.occurs under point load.

Since the moment at support C is high, 20 % redistribution will be carried out.

M_{max} $= 114 \times 2.5 - 28.95 \times 2.5^2/2 - 111.6 = 83$ *kN.m*

Redistribution of Moments :

Joint	A		B		C
Moments before redistribution	–	+24.2	–84.1	83.0	–116.6
Redistribution at *C* by 20%		–	–	11.2	22.3
Design Moments after redistribution		+24.2	–84.1	94.2 (approx.)	–89.3

Calculations revised after redistribution of Moments in Span BC :

V_{BC} $= 28.95 \times 4.7/2 + 86 \times 2.5/4.7 - (89.3 - 84.1)/4.7 = 112.7$ *kN*

V_{CB} $= 28.95 \times 4.7/2 + 86 \times 2.2/4.7 + (89.3 - 84.1)/4.7 = 109.4$ *kN*

Explanatory Notes for beam B14-B13 continued...

M_{max} = 109.4 x 2.5 – 28.95 x 2.5²/2 – 89.3 = 93.7 kN.m

These moments have been entered at *Step 5*

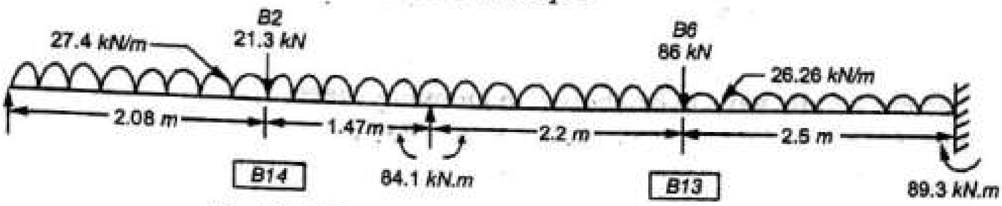


Fig. 9.5.7 Loading for Calculation of Shear

Design of Reinforcement at Support C

b_f = 200 mm, D = 380mm , (for two rows of bars) d' = 56mm , d = 380 – 56 = 324 mm

M_u = 89.3 kNm

dM = 0.2 $\quad \therefore k_{u.limit} = 0.6 - 0.2 = 0.4 < 0.48$

$\therefore x_{u.limit} = 0.4 \times d = 0.4 \times 324 = 129.6\ mm$

$M_{ur.limit} = 0.36 \times 20 \times 200 \times 129.6 \times (324 - 0.416 \times 129.6) \times 10^{-6} = 50.4\ kN.m < M_u\ (= 89\ kN.m)$

∴ Section is doubly reinforced

$$A_{st1} = \frac{50.4 \times 10^6}{0.87 \times 415 \times (324 - 0.416 \times 129.6)} = 516.8\ mm^2$$

$M_{u2} = 89.3 - 50.4 = 38.9\ kN.m$

$$A_{st2} = \frac{38.9 \times 10^6}{0.87 \times 415 \times (344 - 36)} = 349.8\ mm^2$$

Total area of tension steel = $A_{st1} + A_{st2} = 516.8 + 349.8 = 866.6\ mm^2$

$d'/d = 34/324$ = say 0.1 for which $f_{sc} = 348/N/mm^2$ *(Table 4.2.2)*

$$\therefore \quad A_{sc} = \frac{0.87 \times 415 \times 349.8}{(348 - 0.446 \times 20)} = 373\ mm^2 \quad \textit{(Eq. 4.2.3c)}$$

∴ Provide 2#20 + 3#12 at top of support and 1#16 + 2#12 at bottom.

Area provided = 968 mm^2 at top and 427 mm^2 at bottom

For remaining sections the No-Dia. will be selected from *Table F-2B* and *F-3B*

Point of Contraflexure from C : (see Fig. 9.5.6)

$109.4x - 28.95\ x^2/2 - 89.3 = 0$

$\therefore x^2 - 7.56x + 6.17 = 0 \quad \therefore x = 0.93\ m$

Actual point of curtailment = 0.93 + 0.344 = say 1.3 $m > L_d$

Point of contraflexure from B (Span BC) :

$112.7x - 28.95x^2/2 - 84.1 = 0$

$\therefore \quad x^2 - 7.78x + 5.81 = 0 \quad \therefore x = 0.84\ m$

Actual point of curtailment = 0.84 + 0.344 say 1.2 m

Point of contraflexure from B (Span BA)

$89x - 29.8\ x^2/2 - 84.1 = 0 \quad \therefore x = 1.18\ m$

Actual point of curtailment = 1.18 + 0.344 say 1.53 m

Calculation of Shear in Beam B14 - B13 (Fig. 9.5.7)

$$V_{AB} = 27.4 \times 3.55/2 + \frac{21.3 \times 1.47}{3.55} - \frac{84.1}{3.55} = 33.8\ kN$$

$$V_{BA} = 27.4 \times 3.55/2 + \frac{21.3 \times 2.08}{3.55} + \frac{84.1}{3.55} = 84.80\ kN$$

$$V_{BC} = 26.26 \times 4.7/2 + 86 \times 2.5/4.7 - (89.3 - 84.1)/4.7 = 106.3\ kN$$

$$V_{CB} = 26.26 \times 4.7/2 + 86 \times 2.2/4.7 + (89.3 - 84.1)/4.7 = 103.1\ kN$$

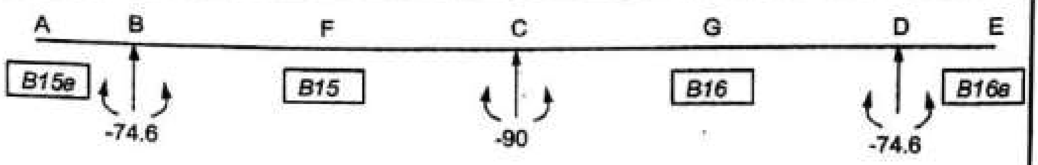
The details of curtailment of bars have been worked out as per Codal provisions.

The reinforcement details have been shown on Drawing sheet.

Miscellaneous Floor Beam - B15a - B15 - B16 - B16a

1. *Beam Mark*	*B15a*	*B15*	*B16*	*B16a*
2. *Span L m*	1.47	4.7	4.7	1.47
3. *Section :*				
Width b *mm*	200	200	200	200
Depth D *mm*	600	600	600	600
Depth of Slab D_f *mm*	100	100	100	100
4. *Loads*				
(a) Slab Right		*S4*	*S4*	-
Load w_{u1} *kN/m*	-	11.7(2*m*)	11.7(2.1*m*)	-
(b) Slab Left				
Load w_{u2} *kN/m*	-	11.95	11.95	-
(c) Wall w_{uw} *kN/m*	18.2	18.2	18.2	18.2
(d) Self w_{us} *kN/m*	3.8	3.8	3.8	3.8
(e) **Total** w_u ***kN/m***	**22.0**	**33.95/45.65**	**45.65/33.95**	**22.0**
(f) Point Load From	*B2a + B20*	*B18*	*B17*	*B2*
Magnitude *kN.m*	34.6 *kN*	26.9	5.4	23.7

	A	B	F	C	G	D	E
5. *Maximum Moments* $M_{u.max}$ *kN.m*		-74.6	+52.75	-90	+37.8	-58.6	
6. *Section type at Mid-span*			*R*	*L*	*R*	*L*	*R*
b_f			774		774		
b_f / b_w			3.87		3.87		
7. *Main Steel*							
Top		2-12+1-16	2-12	2-16+2-12	2-12	2-12+1-10	
Bottom		2-10+1-12	2-10+1-12	2-10+1-12	2-10+1-12	2-10+1-12	
M_{ur} Provided		80.42	54.64	113.52	54.64	58.98	
Effective depth		566	566	566	566	566	
8. *Equivalent UDL for Shear*							
(a) Slab Right							
Load w_{u1} *kN/m*		-	11.7(2*m*)		11.7(2.1*m*)		-
(b) Slab Left							
Load w_{u2} *kN/m*		-	9.26		9.26		-
(c) Wall *kN/m*		18.2	18.2		18.2	18.2	
(d) Self *kN/m*		3.8	3.8		3.8	3.8	
(e) **Total** ***kN/m***		**22.0**	**31.26/42.96**		**42.96/31.26**	**22**	
(f) Point Load from		*B2a + B20*	*B18*		*B17*	*B2*	
kN		34.6	26.9		5.4	23.7	



	B	C	D
9. ***Design for Shear***			
$V_{u.max}$ kN	66.9 | 86.6	110.6 | 102.2	74.7 | 56.0
A_{stl}	2-#12+1#16	2#16+2#12	2#12+1#10
$V_{ur.min}$	93.61	101.73	87.49
V_{uD}	-	81.98	-
Min. Stirrups ϕ-s	ϕ6-150	ϕ6-150	ϕ6-150
10. ***Load transferred to column kN***	***C23*** ***153.5*** ***(66.9+86.6)***	***C15*** ***212.8*** ***(110.6 +102.2)***	***C7*** ***130.7*** ***(74.7 + 56.0)***

Explanatory Notes : Calculation of Bending Moment and Shear Force in Beam *B15a-B15-B16-B16a*
The loading on beam for computation of shear is shown in *Fig. 9.5.9*

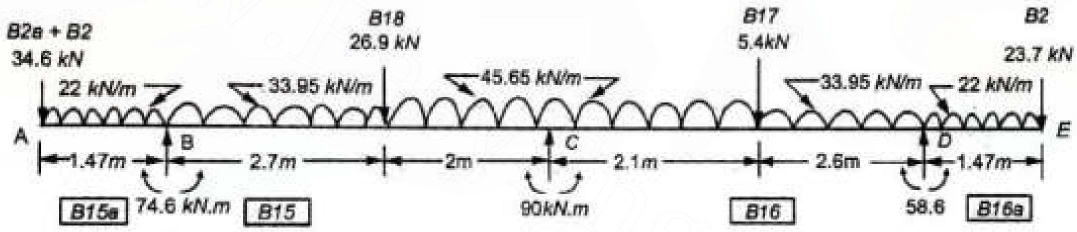


Fig. 9.5.8 Loading for Calculation of B.M.

The distribution factors will remain the same equal to 0.5 at joint *C*

Fixed End moments : Span BC

M_{FBA} = 22 x 1.47²/2 + 34.6 x 1.47 = 74.6 *kN.m*

For *Span BC* :

a = 2 *m* from *C*

w' = 45.65 – 33.95 = 11.7 *kN/m*

M_{FBC} = 33.95 x 4.7²/12 + 11.7 x 2³ (4 x 4.7 – 3 x 2)/(12 x 4.7²) + 26.9 x 2.7 x 2²/4.7²
= 80.2 *kN.m*

M_{FCB} = 33.95 x 4.7²/12 + 11.7 x 2²(3 x 2² – 8 x 2 x 4.7 + 6 x 4.7²)/(12 x 4.7²) + 26.9 x 2.7² x 2/4.7²
= 92.4 *kN.m*

For *Span CD* ,

a = 2.1*m* from *C*

w' = 45.65 – 35.95 = 11.7 *kN/m*

M_{FCD} = 33.95 x 4.7²/12 + 11.7 x 2.1²x(3 x 2.1² – 8 x 2.1 x 4.7 + 6 x 4.7²)/(12 x 4.7²) + 5.4 x 2.1 x 2.6²/4.7²
= 79.0 *kN.m*

M_{FDC} = 33.95 x 4.7²/12 + 11.7 x 2.1³ x (4 x 4.7 – 3 x 2.1)/(12 x 4.7²) + 5.4 x 2.1² x 2.6/4.7²
= 70.4 *kN.m*

M_{FDE} = 22 x 1.47²/2 + 23.7 x 1.47 = 58.6 *kN.m*

Explanatory Notes for Floor Beam B15a-B16-B16a continued...

Moment Distribution :

Joint	B		C		D	
Member	BA	BC	CB	CD	DC	DE
D. F.	0	1	0.5	0.5	1	0
FEM	74.6	−80.2	92.4	−79	70.4	−58.6
		5.6	2.8	−5.9	−11.8	–
		–	−5.2	−5.1		
Final Moments	74.6	−74.6	90.0	−90.0	58.6	−58.6

Span Moments :

Span BC

V_{BC} = 33.95 x 4.7/2 + 11.7 x 2 x 1/4.7 − (90 − 74.6) / 4.7 + 26.9 x 2/4.7
= 93 kN

x_{max} = 93/33.95 = 2.73m > 2.7 m ∴ Maximum Moment occurs under point load.

M_{max} = 93 x 2.7 - 33.95 x 2.7^2/2 − 74.6 = 52.75 kN.m

V_{CB} = 33.95 x 4.7/2 + 11.7 x 2 x 3.7/4.7 + (90 − 74.6) / 4.7 + 26.9 x 2.7/4.7
= 116.9 kN

Span CD

V_{CD} = 45.65 x 2.1 x (2.1/2 + 2.6) / 4.7 + 5.4 x 2.6/4.7 + (90 − 58.6)/4.7 + 33.95 x 2.6^2/(2 x 4.7)
= 108.5 kN

V_{DC} = 45.65 x 2.1^2/(2 x 4.7) + 5.4 x 2.1/4.7 − (90 − 58.6)/4.7 + 33.95 x 2.6 x (2.1 + 1.3)/4.7
= 81kN.m

x_{max} = 81/33.95 = 2.38 m < 2.6 m

M_{max} = 81 x 2.38/2 − 58.6 = 37.8 kN.m

The moments have been entered at *Step - 5*

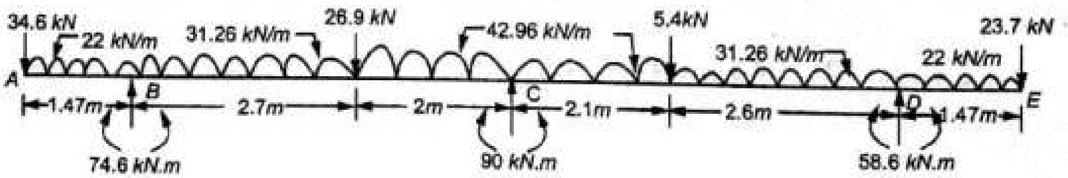


Fig. 9.5.9 Loading for Calculation of Shear

Calculation of Shear

V_{BA} = 22 x 1.47 + 34.6
= 66.9 kN

V_{BC} = 31.26 x 2.7 x 3.35/4.7 + 42.96 x 2^2/ (2 x 4.7) + 26.9 x 2/4.7 − (90 − 74.6)/4.7
= 86.6 kN

V_{CB} = 31.26 x 2.7^2/(2 x 4.7) + 42.96 x 2 x 3.7/4.7 + 26.9 x 2.7/4.7 + (90 − 74.6)/4.7
= 110.6 kN

V_{CD} = 31.26 x 2.6 x 1.3/ 4.7 + 42.96 x 2.1x (2.1/2 x 2.6)/4.7 + 5.4 x 2.6/4.7 + (90 − 58.6)/4.7
= 102.2 kN

V_{DC} = 31.26 x 2.6 x (2.1+2.6/2)/ 4.7 + 42.96 x 2.1^2/(2 x 4.7) + 5.4 x 2.1/4.7 − (90 − 58.6)/4.7
= 74.7 kN

V_{DE} = 22 x 1.47 + 23.7 = 56.0 kN

The shear forces are entered at *Step - 9*

The details of curtailment of bars have been worked out as per Codal provisions.
The reinforcement details have been shown on Drawing sheet.

9.5.3 Concluding Remarks

The design of residential building, which is difficult to divide into planer frames, has been illustrated using design aids. In this connection the following points may be noted.

(1) The effect of continuity of slab and beam has been taken into account and the values of the shearing forces have been worked out using different coefficients at continuous and discontinuous ends. Due to this the reader may feel the procedure to be little clumsy at an outset but the load is transferred from one element to the other in the same manner. However, the calculations can be simplified if the same coefficient of 0.5 is used at both ends of beam/slab.

(2) Also different coefficients for finding out equivalent UDL for Bending Moment and Shear force have been used for transferring the load of a two- way slab to beam. For computational simplification one may use the EUDL for bending moment to compute the values of shearing forces, but due to this the load gets over estimated to the extent of about 30%. Therefore, it is up to the designer how much he desires to err on the safer side by simplifying the calculations.

(3) It may be mentioned that the depth of the slab is normally governed by the deflection criteria while the beam is safe against deflection. As such the deflection check may not be applied to beams.

(4) The bond requirements are normally satisfied and hence the detailed calculations for development length required and development length provided have not been made. But it should be ensured that the bond length of $L_d/3$ is provided from the face of support as specified by the Code.

9.5.4 Plinth Beams

Category - *I : B1, B3, B3, B6*
- *II : B7, B8, B11*
- *III B9, B10, B19, B21*
- *IV : B13, B14, B15, B16, B15a, B16a*

Beams *B19* and *B21* of size 200 x 300 *mm* do not carry any wall load. They are provided only to interconnect columns to impart rigidity to the frame. Beams *B2, B5, B12, B17, B18* and *B20* are not provided

Step	Category	I				II			III	
	B.M. Coefficient	1/8				1/10			1/12	
1.	*Beam Mark*	*B1*	*B3*	*B6*	*B4*	*B7*	*B8*	*B11*	*B9*	*B10*
2.	*Span* *L m*	3.4	3.4	3.4	3.2	3.4	3.2	3.15	2.6	2.5
3.	*Section* *b mm*	200	200	200	200	200	200	200	200	200
	D mm	380	380	380	380	380	380	380	380	380
4.	*Wall Load*	21.0	11.7	11.7	21.0	11.7	11.7	21.0	21.0	21.0
	Self	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2
	Total	24.2	14.9	14.9	24.2	14.9	14.9	24.2	24.2	24.2
5.	*Maximum Moment* $M_{u.max}$ *kN.m*	35.0	21.5	21.5	31.0	17.2	15.2	24.0	13.6	12.6
6.	Main Steel :									
	Top *N-# mm*	2-10	2-10	2-10	2-10	2-10	2-10	2-10	2-10	2-10
	Bottom bent *N-# mm*	1-12	-	-	1-12	-	-	-	-	-
	Bottom Straight *N-#mm*	2-12	2-12	2-12	2-10	2-10	2-10	2-12	2-10	2-10
	Provided A_{st} mm^2	339	226	226	270	157	157	226	157	157
	Provided *d*	344	345	345	344	345	345	344	345	345
	Provided M_{ur} *kN.m*	38.29	26.49	26.49	31.19	18.86	18.86	26.49	18.86	18.86

Design of Plinth Beams continued....

7.	*Shear* : (a) *S.S. end*									
	$V_{u.max}$ kN	41.1	25.3	25.3	38.7	22.8	21.5	34.3	31.5	30.3
	A_{st1} N-# mm	2-12	2-12	2-12	2-10	2-10	2-10	2-10	2-10	2-10
	$V_{ur.min}$ kN	55.77	55.77	55.77	51.89	51.89	51.89	51.89	51.89	51.89
	Stirrups ϕ - s	-	-	-	-	6-150	6-150	6-150	6-150	6-150
	(b) Continuous End									
	$V_{u.max}$ kN	-	-	-	-	30.4	28.6	45.8	31.5	30.3
	A_{st1} N-# mm	-	-	-	-	2-10	2-10	2-12	2-10	2-10
	$V_{ur.min}$ kN	-	-	-	-	51.89	51.89	55.77	51.89	51.89
	Stirrups ϕ - s	6-150	6-150	6-150	6-150	6-150	6-150	6-150	6-150	6-150
8.	***Load transferred to column*** kN	***C27***	***C21***	***C19***	***C22***	***C13***	***C14***	***C27***	***C19***	***C21***
		41.1	***25.3***	***25.3***	***38.7***	***22.8***	***28.6***	***34.3***	***31.5***	***30.3***
		C28	***C22***	***B13***	***C23***	***C14***	***C15***	***C21***	***C13***	***C19***
		41.1	***25.3***	***25.3***	***38.7***	***30.4***	***21.5***	***45.8***	***31.5***	***30.3***

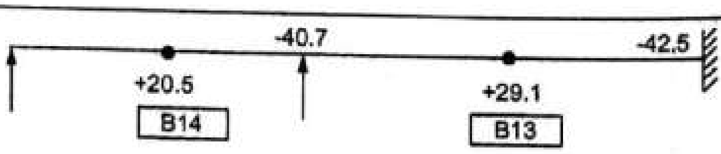
Explanatory Notes to Step Nos. in above Design

(4) Self weight rectangular beam of width 200 *mm* (225 *mm* with plaster) and depth 380 *mm*

$$= 25 \times 0.225 \times 0.38 \times 1.5 = 3.2 \ kN/m$$

(4) In the case of beams *B1, B4, B9, B10* and *B11* the load due to 200 *mm* (225 *mm* with plaster) thick brick wall comprises of floor wall of 3*m* height plus plinth wall 0.45 *m* height less depth beam 0.38 *m*. Thus, the load due to outer walls will be 21 *kN/m* (= 20 x 0.225 x 3.1 x 1.5). The inner walls of 100 *mm* thick provided on beams *B3, B6, B7* and *B8* will also be provided at ground level for giving rigidity. The load on these beams will be 11.7 *kN/m* (= 20 x 0.125 x 3.1 x 1.5)

Design of Plinth Beams : Category - IV - Beam B14 - B13

1.	*Beam Mark*	*B14*	*B13*
2.	*Span* L m	3.55	4.7
3.	*Section* : Width b mm	200	200
	Depth D mm	380	380
	Assumed d mm	346	346
4.	*Loads* : Wall kN/m	21.0	11.7
	Self kN/m	3.2	3.2
	Total kN/m	**24.2**	**14.9**
	Point Load from	-	*B6*
	Value kN	-	25.3
	Distance from nearest column m	-	2.2
5.	*Maximum Moments* $M_{u.max}$ kN.m		
6.	*Main Steel* : Top	(2#12) (2#12) 2#12 + 1#16	2#12 1#16 + 2#12
	Bottom	(2#12) 2#12 1#10+2#12	(2#12)
	M_{ur} Provided kN.m	26.49 46.50	34.79 46.50

Design of Plinth Beams : Category - IV - Beam B14 - B13 continued...

7.	*Design for Shear*				
	V_{umax}	kN	31.5	54.4 | 48.1	47.2
	A_{st1}	-	2#12	2#12+1#16	2#12+1#16
	$V_{ur.min}$	kN	55.77	63.65	63.65
	Minimum Stirrups	ϕ - s	6-150	6-150	6-150
8.	*Load Transferred to column*		**C28** 31.5	**C22** 102.5/(54.4 | 48.1)	**C14** 94.4(47.2 | 47.2)

Explanatory Notes :

Calculation of Bending Moment and Shear for Beam B13-B14

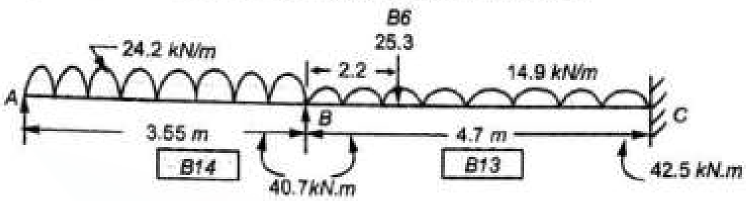


Fig. 9.5.10 Loading on Plinth Beam

Fixed End Moments :

M_{FBA} = $24.2 \times 3.55^2/8 = 38.1$ kN.m , M_{FBC} = $14.9 \times 4.7^2/12 + 25.3 \times 2.2 \times 2.5^2/4.7^2 = 43.2$ kN.m

M_{FCB} = $14.9 \times 4.7^2/12 + 25.3 \times 2.2^2 \times 2.5/4.7^2 = 41.3$ kN.m

The distribution factors will be the same equal to 0.5 as obtained earlier

Moment Distribution :

Joint	A	B		C
Distribution factors		0.5	0.5	
Fixed End Moments		38.1	–43.2	41.3
Distributed & Carry over moments	0	2.6	2.5	1.2
Final Moments	0	40.7	–40.7	42.5

Span Moments :

Span AB :

V_{AB} = $24.2 \times 3.55/2 - 40.7/3.55 = 31.5$ kN , V_{BA} = $24.2 \times 3.55/2 + 40.7/3.55 = 54.4$ kN

x_{max} = $31.5/24.2 = 1.30$ m , M_{max} = $31.5 \times 1.30/2 = 20.5$ kN

Span BC

V_{BC} = $14.9 \times 4.7/2 + 25.3 \times 2.5/4.7 - (42.5 - 40.7)/4.7 = 48.1$ kN

V_{CB} = $14.9 \times 4.7/2 + 25.3 \times 2.2/4.7 + (42.5 - 40.7)/4.7 = 47.2$ kN

x_{max} = $48.1/14.9 = 3.2\ m > 2.2\ m$ $\therefore$ Maximum moments occurs under point load

M_{max} = $48.1 \times 2.2 - 14.9 \times 2.2^2/2 - 40.7 = 29.1$ kN.m

The values of moments are entered at *Step - 5*.

The details of curtailment of bars have been worked out as per Codal provisions.

The reinforcement details have been shown on Drawing sheet.

Miscellaneous Plinth Beam - B15a - B15-B16-B16a

1.	*Beam Mark*		*B15a*	*B15 & B16*
2.	*Span*	*L* m	1.47	4.7
3.	*Section* : Width *b*	mm	200	200
	Depth *D*	mm	380	380
	Assumed *d*	mm	344	344

Miscellaneous Plinth Beam - B15a - B15-B16-B16a continued...

No.	Item		Support B (A–B–F)	Span BC	Support C	Span CD	Support D (D–E)
4.	*Loads* : Wall	*kN/m*		21.0		21.0	
	Self	*kN/m*		3.2		3.2	
	Total	*kN/m*		24.2		24.2	
5.	*Maximum Moments* $M_{u.max}$	*kN.m*	A, B: 26.1 kN.m	F: +27.3 *kN.m*	C: -53.9 kN.m	G: +27.3*kN.m*	D, E: 26.1 kN.m
7.	*Main Steel* : Top		2#12	2#12	3#12 + 1#16	(2#12)	2#12
	Bottom bent						
	Bottom Straight		(2#12)	2#12	(2#12)	2#12	(2#12)
8.	M_{ur} Provided *kN.m*		BA | BC 26.49		CB | CD 56.51		DC | DE 26.49 | 26.49
9.	*Shear*	$V_{u.max}$ kN	35.6 | 50.9		62.8 | 62.8		50.9 | 35.6
		$V_{ur.min}$ kN	55.77		67.02		55.77
		ϕ - *s*	6-150		6-150		6-150
10.	*Load transferred to column*	kN	***C23*** ***86.5*** ***(35.6 + 50.9)***		***C15*** ***125.6*** ***(62.8 + 62.8)***		***C7*** ***86.5*** ***(50.9 + 35.6)***

Explanatory Notes

The load on plinth beam is shown in *Fig. 9.5.11*

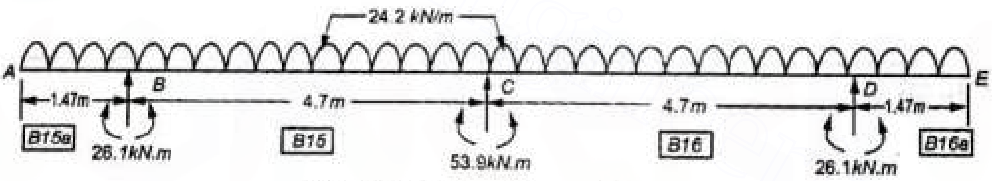


Fig. 9.5.11 Loading on Beam

Since the beam has even span support symmetry the moment distribution will be carried out assuming *C* as fixed.

Fixed End Moments :

M_{FBA} = $24.2 \times 1.47^2/2$ = 26.1 *kN.m* , M_{FBC} = $24.2 \times 4.7^2/12 = 44.6$ *kN.m*

Moment Distribution

Joint	*A*	B		*C*
Member	-	*BA*	*BC*	*CB*
D.F.		0	1	
FEM	-	26.1	– 44.6	44.6
Balance		-	18.5	9.3
Final Moments	0	26.1	–26.1	53.9

Calculation of Shear and Span Moments

V_{BA} = 1.47 x 24.2 = 35.6 *kN* , V_{BC} = 24.2 x 4.7/2 – (53.9 – 26.1)/4.7 = 50.9 *kN*

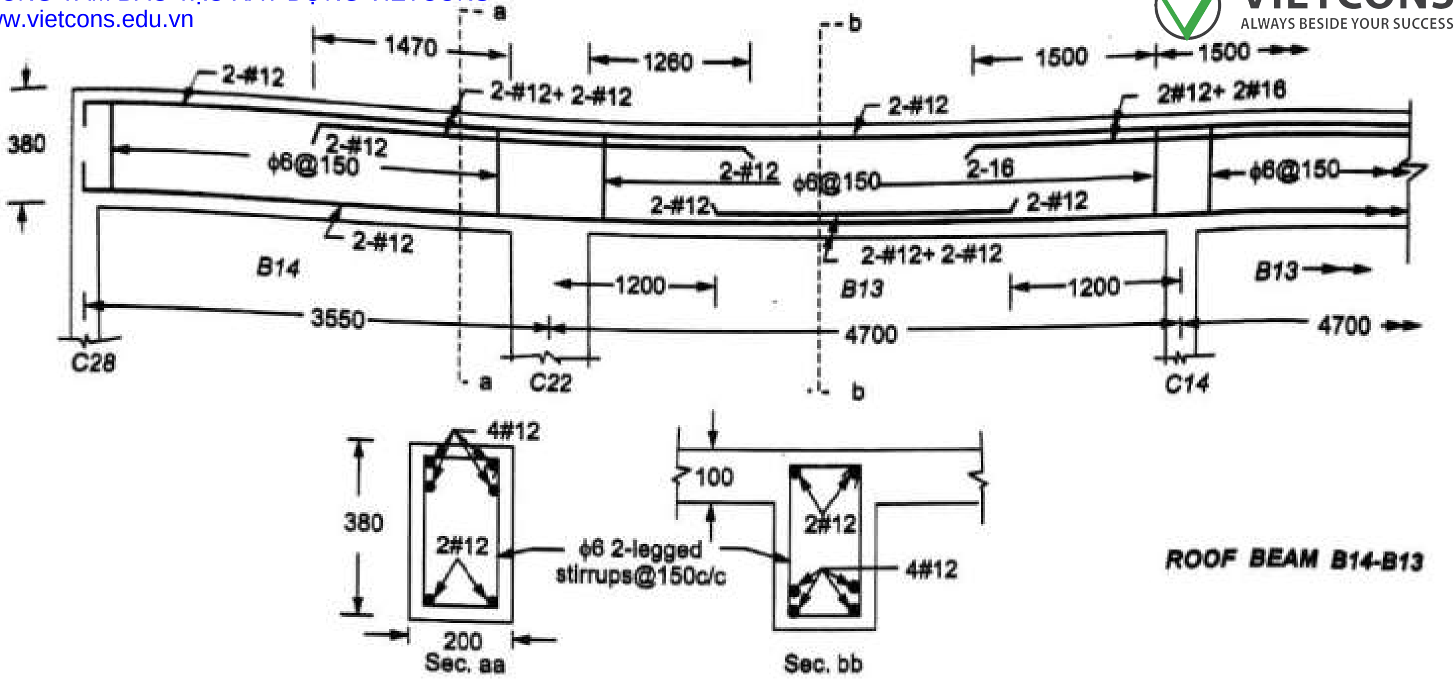
V_{CB} = 24.2 x 4.7/2 + (53.9 – 26.1)/4.7 = 62.8 *kN*

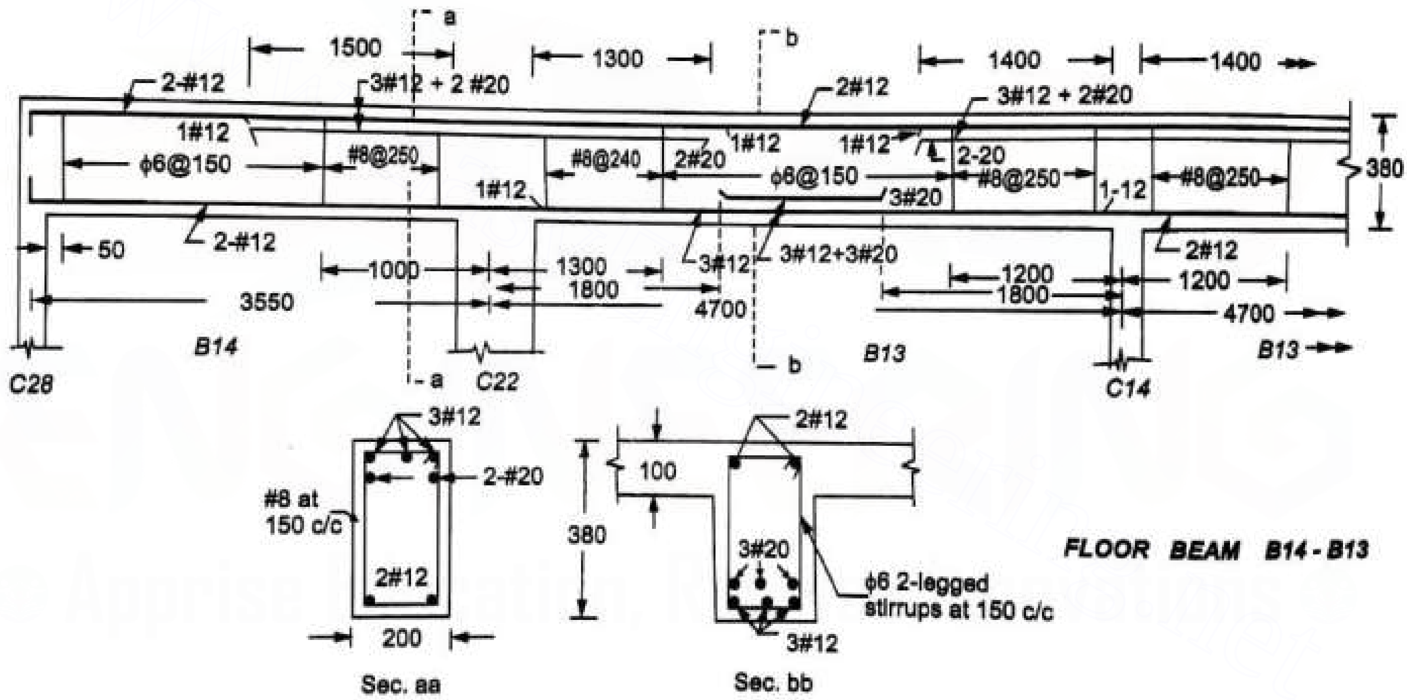
Span BC

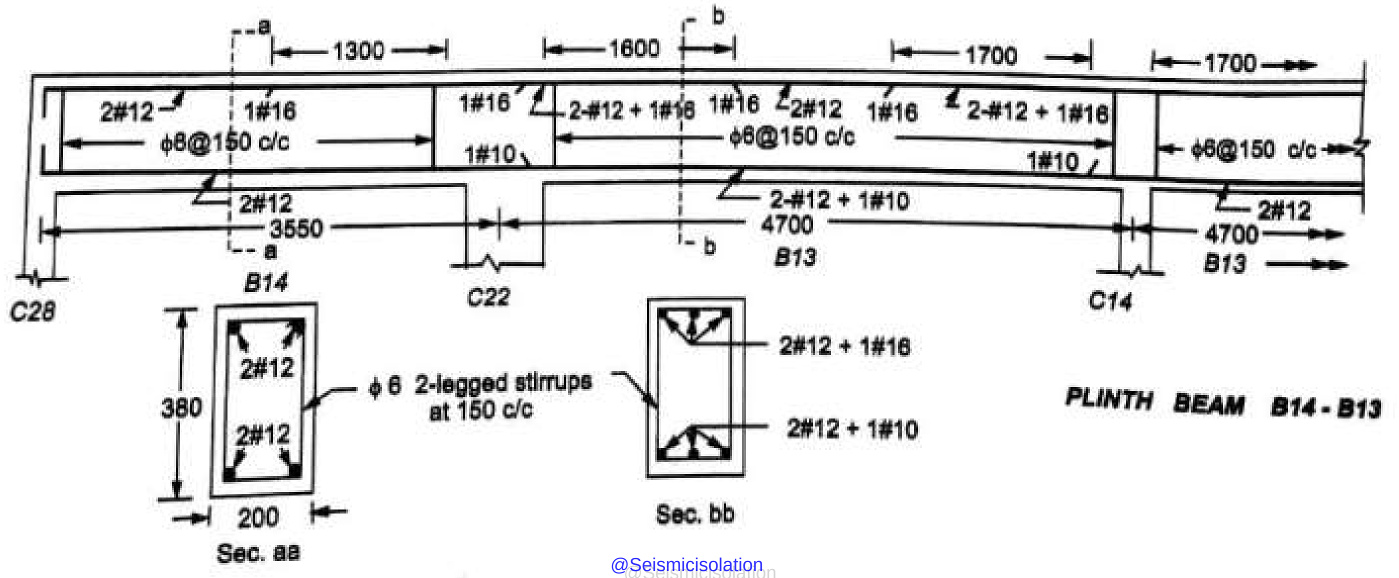
x_{max} = 50.9/24.2 = 2.1 *m* , M_{max} = 50.9 x 2.1/ 2 – 26.1 = 27.3 *kN.m*

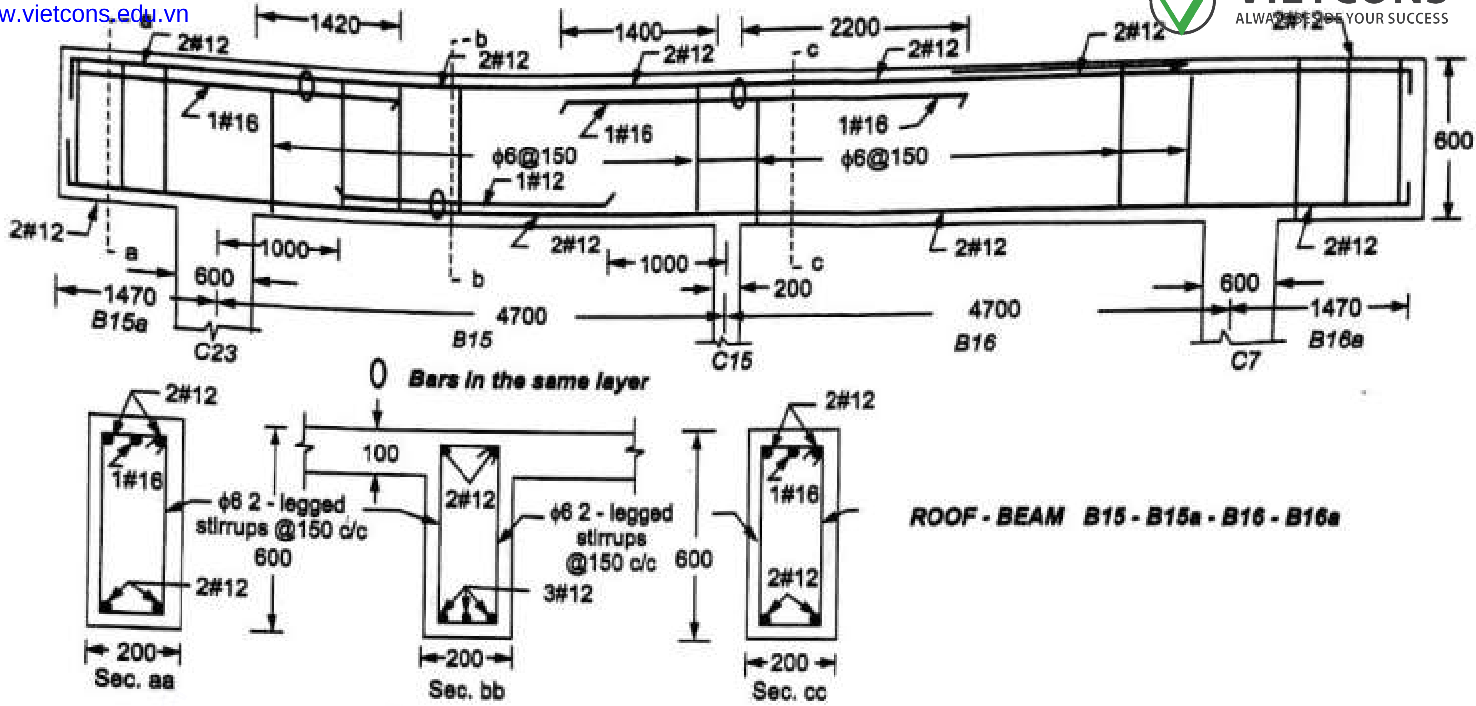
The values of the moment and Shear are entered at *Step -5* and *Step - 9* respectively.

The main steel is designed using *Table F-2B*

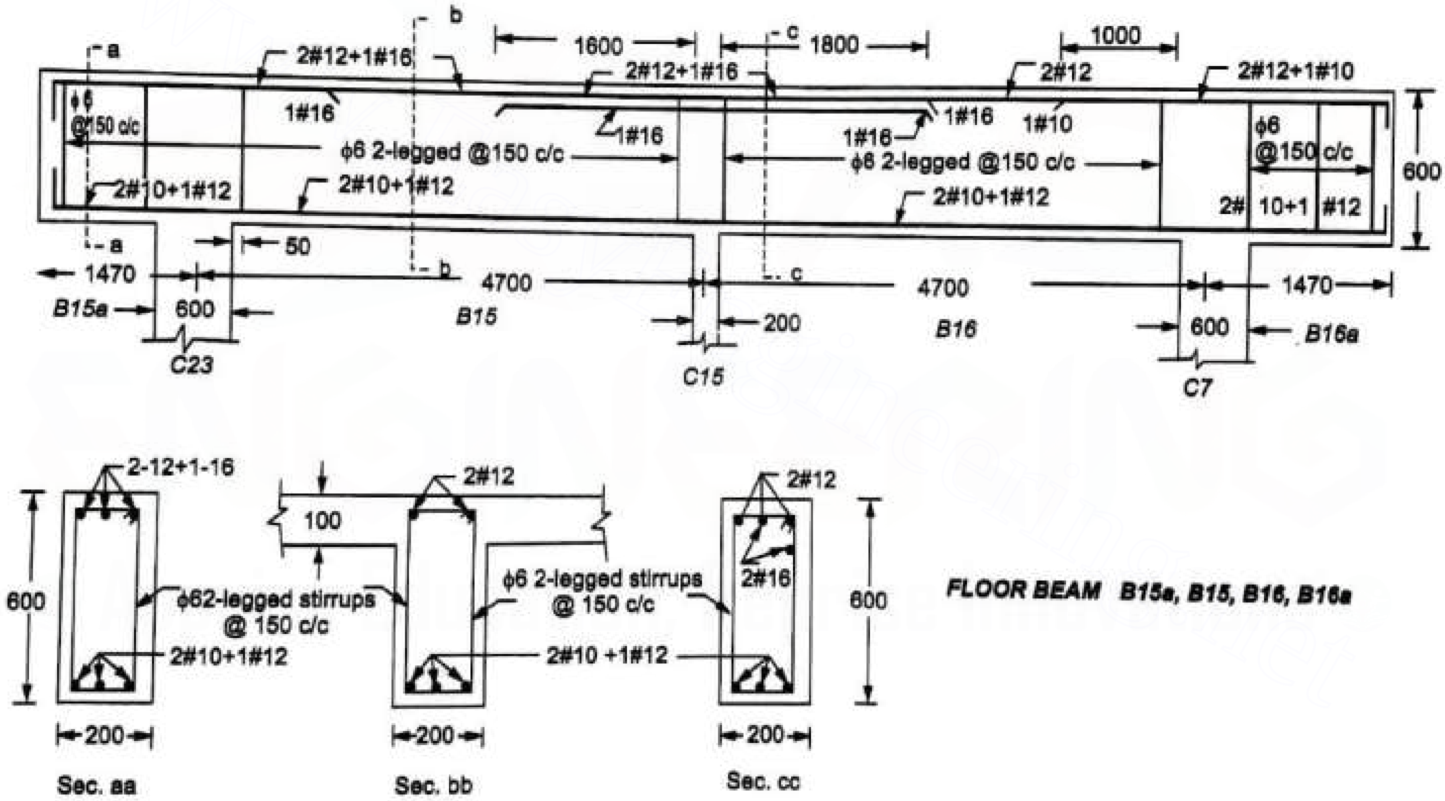

1470
1260
1500
1500
2-#12
2-#12+ 2-#12
2-#12
2#12+ 2#16
380
2-#12
ϕ6@150
2-#12
ϕ6@150
2-16
ϕ6@150
2-#12
2-#12
2-#12
2-#12+ 2-#12
B14
1200
B13
1200
B13
3550
4700
4700
C28
C22
C14
4#12
380
2#12
200
Sec. aa
100
2#12
4#12
ϕ6 2-legged
stirrups@150c/c
Sec. bb
ROOF BEAM B14-B13


1500
1300
1400
1400
2-#12
3#12 + 2 #20
2#12
3#12 + 2#20
1#12
ϕ6@150
#8@250
#8@240
2#20
1#12
1#12
2-20
#8@250
#8@250
1#12
ϕ6@150
3#20
1-12
380
50
2-#12
3#12
3#12+3#20
2#12
1000
1300
1800
1200
1800
1200
3550
4700
4700
B14
B13
B13
C28
C22
C14
3#12
2-#20
#8 at
150 c/c
2#12
200
Sec. aa
2#12
100
380
3#20
3#12
ϕ6 2-legged
stirrups at 150 c/c
Sec. bb
FLOOR BEAM B14-B13

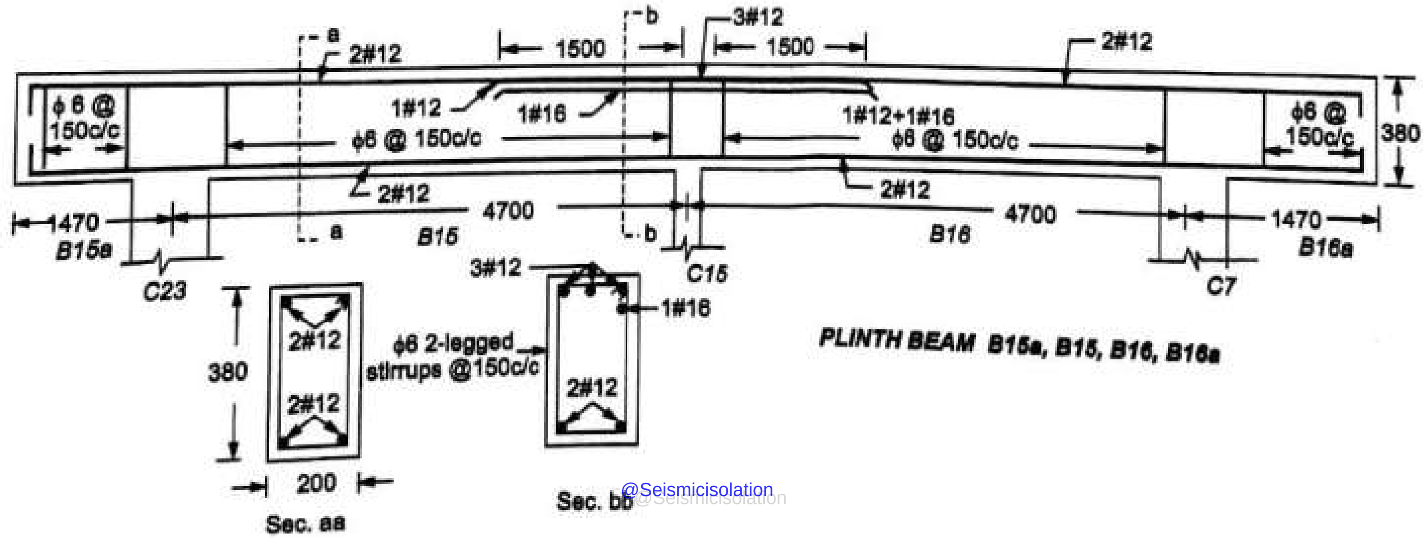

1300
1600
1700
1700
2#12
1#16
1#16
2-#12 + 1#16
1#16
2#12
1#16
2-#12 + 1#16
ϕ6@150 c/c
1#10
ϕ6@150 c/c
1#10
ϕ6@150 c/c
2#12
2-#12 + 1#10
2#12
3550
4700
4700
B14
B13
B13
C28
C22
C14
2#12
380
2#12
200
Sec. aa
ϕ 6 2-legged stirrups
at 150 c/c
2#12 + 1#16
2#12 + 1#10
Sec. bb
PLINTH BEAM B14-B13



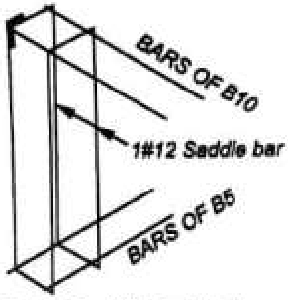
ROOF - BEAM B15 - B15a - B16 - B16a



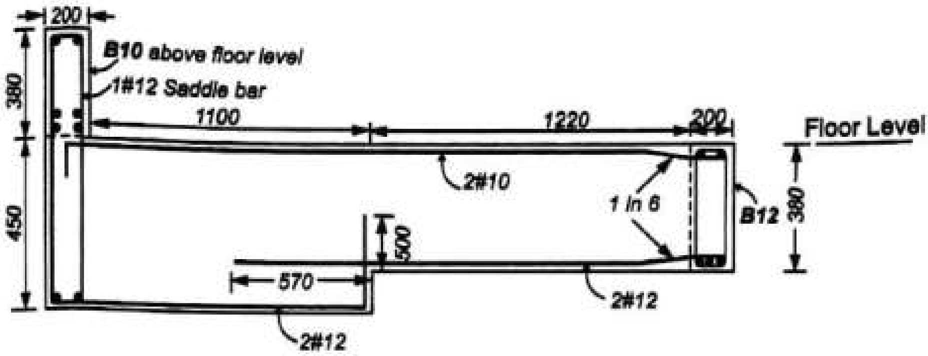
FLOOR BEAM B15a, B15, B16, B16a



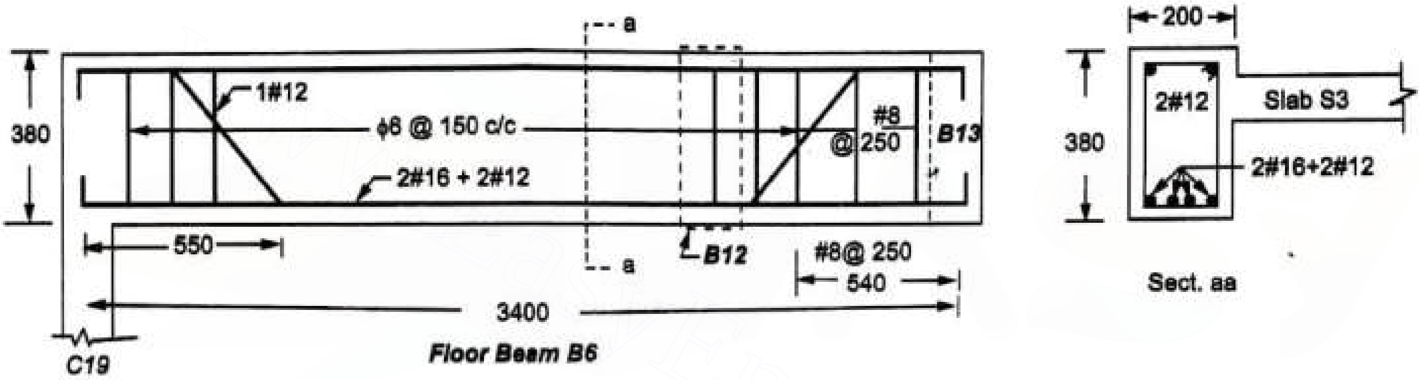
PLINTH BEAM B15a, B15, B16, B16a



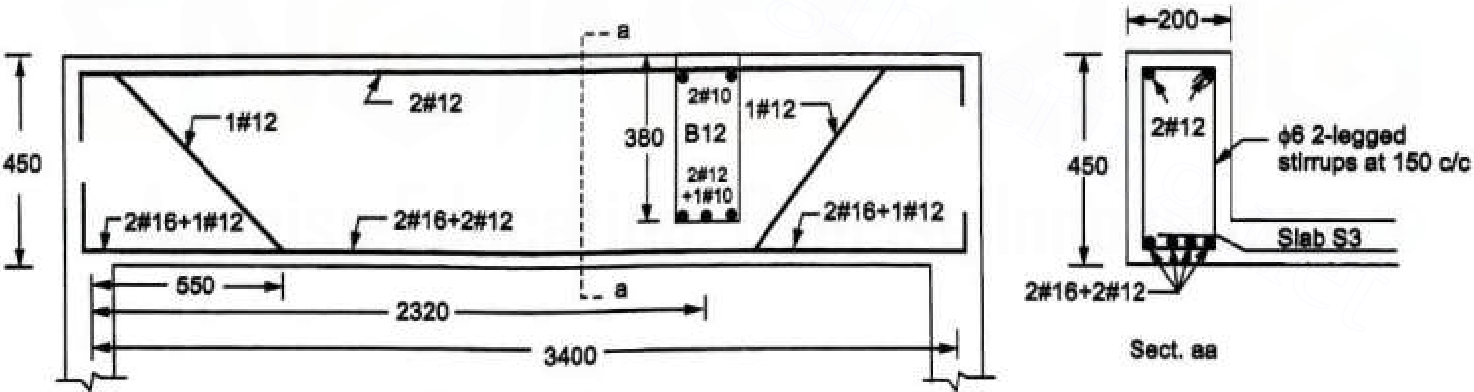
Typical Saddle Bar Details



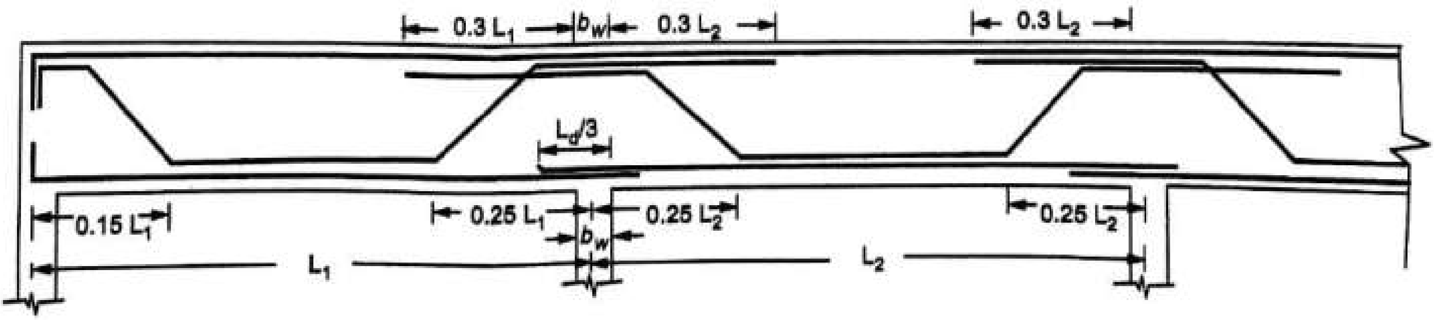
FLOOR BEAM B5



Floor Beam B6



Floor Beam B3



***Note :** Detailing Applicable for Continuous Beams of Approximately Equal Spans Not Differing by more than 15 % of the Longest*

Typical Details of Bar Bending for Beams

SCHEDULE OF BEAMS

Beam Nos.	*Size* B D mm mm	*Steel at Bottom*		*Top Steel*	*Stirrups*	*Remarks*
		Straight	*Bent*			
(A) ROOF BEAMS						
B9, B10, B11, B17, B20, B2a	200 x 300	2#10	–	2#10	ϕ 6 @150	
B3, B6, B19	200 x 300	2#16	–	2#10	ϕ 6 @150	
B1, B2	200 x 300	2#12	–	2#10	ϕ 6 @150	
B4	200 x 300	2#12	1#10	2#10	ϕ 6 @150	
B7, B8	200 x 300	2#12	1#10	2#12	ϕ 6 @150	
B18	200 x 300	3#12	–	2#10	ϕ 6 @150	
B13, B14, B15a, B15, B16, B16a						*See Drawing*
(B) FLOOR BEAMS						
B1, B10	200 x 380	2#12	2#10	2#10	ϕ 6 @150	
B2, B2a, B9, B17, B18, B20	200 x 380	2#10	–	2#10	ϕ 6 @150	
B3	200 x 450	2#16+1#12	1#12	2#12	ϕ 6 @150	*See Drawing*
B4	200 x 380	3#12	1#12	2#10	ϕ 6 @150	
B5	200 x 450	2#12	–	2#10	ϕ 6 @150	*See Drawing*
B6	200 x 380	2#16+1#12	1#12	2#12	ϕ 6 @150	*See Drawing*
B7	200 x 380	2#12	2#10	2#12	ϕ 6 @150	
B8	200 x 380	2#12	1#12	2#12	ϕ 6 @150	
B11, B12, B19	200 x 380	2#12	1#10	2#10	ϕ 6 @150	
B13, B14, B15a, B15, B16a, B16	–	–	–	–	–	*See Drawing*
(C) PLINTH BEAMS						
B1	200 x 380	2#12	1#12	2#10	ϕ 6 @150	
B3, B6, B11	200 x 380	2#12	–	2#10	ϕ 6 @150	
B4,	200 x 380	2#10	1#12	2#10	ϕ 6 @150	
B7, B8, B9, B10	200 x 380	2#10	–	2#10	ϕ 6 @150	
B13, B14, B15a, B15, B16, B16a	-					*See Drawing*

Notes : 1 Grade of concrete M20
Grade of Steel Fe415 denoted by #
Grade of Steel Fe250 denoted by ϕ

2. Environmental Exposure Condition : Mild
Nominal Cover for Beam 20*mm*
3. First stirrup shall be provided at a distance of 50 *mm* from the face of column
4. All laps shall be staggered and not more than 50% bars to be lapped at any given section.
5. Minimum spacing between any two longitudinal bars in beam shall be 25 *mm*
6. Fire rating considered is for 1 hour maximum.
7. Do not scale the drawing.

	STRUCTURES PUBLICATIONS		
	Four Storeyed Residential Building		
	Structural Details of Beams		
PROJECT NO.	3	STRUCTURAL ENGR.	
PLOT NO.		DRG. NO.	
S. NO.		DRG. BY	
DATE		SIGN	

9.6 DESIGN OF COLUMNS

9.6.1 Categorization of Columns

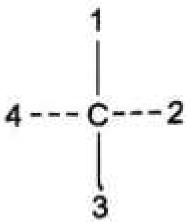
Category - I : Axially Loaded Columns : *C14*

Category - II : Columns subjected to axial compression and uniaxial bending : *C7, C13, C15, C19, C21, C22* and *C23.*

Category - III : Columns under axial compression and biaxial bending : *C27, C28.*

9.6.2 Assessment of Loads on Columns

The actual load on column is obtained from the load transferred by beams to supporting columns. The total load acting on any column is the algebraic sum of the shears at the end of all beams meeting at the column. The load transferred by each beam to column is obtained from design of beams and is shown in *Fig. 9.6.1.* The beam directions are taken as per adjacent figure. The load on each column is calculated as detailed below.



COLUMNS IN TOP STOREY

Loads transferred by Roof Beams

Cate-gory	*Column Mark*	*Beam end Forces in kN from directions*				*Total kN*	*Tank Load kN*	*Total Rounded* P_r *kN*
		1	2	3	4			
I	C14	49.4	30.5	49.4	50.8	180.1	-	181
II	C7	32.7	-	37.7	30.1	100.5	-	101
	C13	5.9	38.1	5.9	-	49.9	-	50
	C15	61.5	-	91.3	22.9	175.7	105	281
	C19	5.9	42.3	5.6	-	53.8	-	54
	C21	5.6	38.0	21.9	-	65.5	-	66
	C22	46.3	30.1	47.2	38.0	161.6	-	162
	C23	103.8	57.0	75.3	30.1	266.2	105	372
III	C27	16.4	19.3	-	-	35.7	-	36
	C28	20.9	-	-	19.3	40.2	-	41

The floor load transferred from each beam is obtained from *Step No. 10* of design of floor beam and is shown in *Fig. 9.6.1.* The load on each column is calculated as detailed below

COLUMNS IN INTERMEDIATE STOREY

Loads transferred by Floor Beams

Cate-gory	*Column Mark*	*Beam end Forces in kN from directions*				*Total kN*	*Total Rounded* P_f *kN*
		1	2	3	4		
I	C14	103.1	50.9	103.1	72.4	329.5	330
II	C7	56.0	-	74.7	60.2	190.9	191
	C13	31.7	54.3	31.7	-	117.7	118
	C15	102.2	-	110.6	38.2	251.0	251
	C19	23.7	73.0	43.3	-	140.0	140
	C21	46.5	63.3	43.2	-	153.0	153
	C22	106.3	60.2	84.8	77.6	328.9	329
	C23	86.6	57.7	66.9	60.2	271.4	272
III	C27	43.2	47.4	-	-	90.6	91
	C28	33.8	-	-	47.4	81.2	82

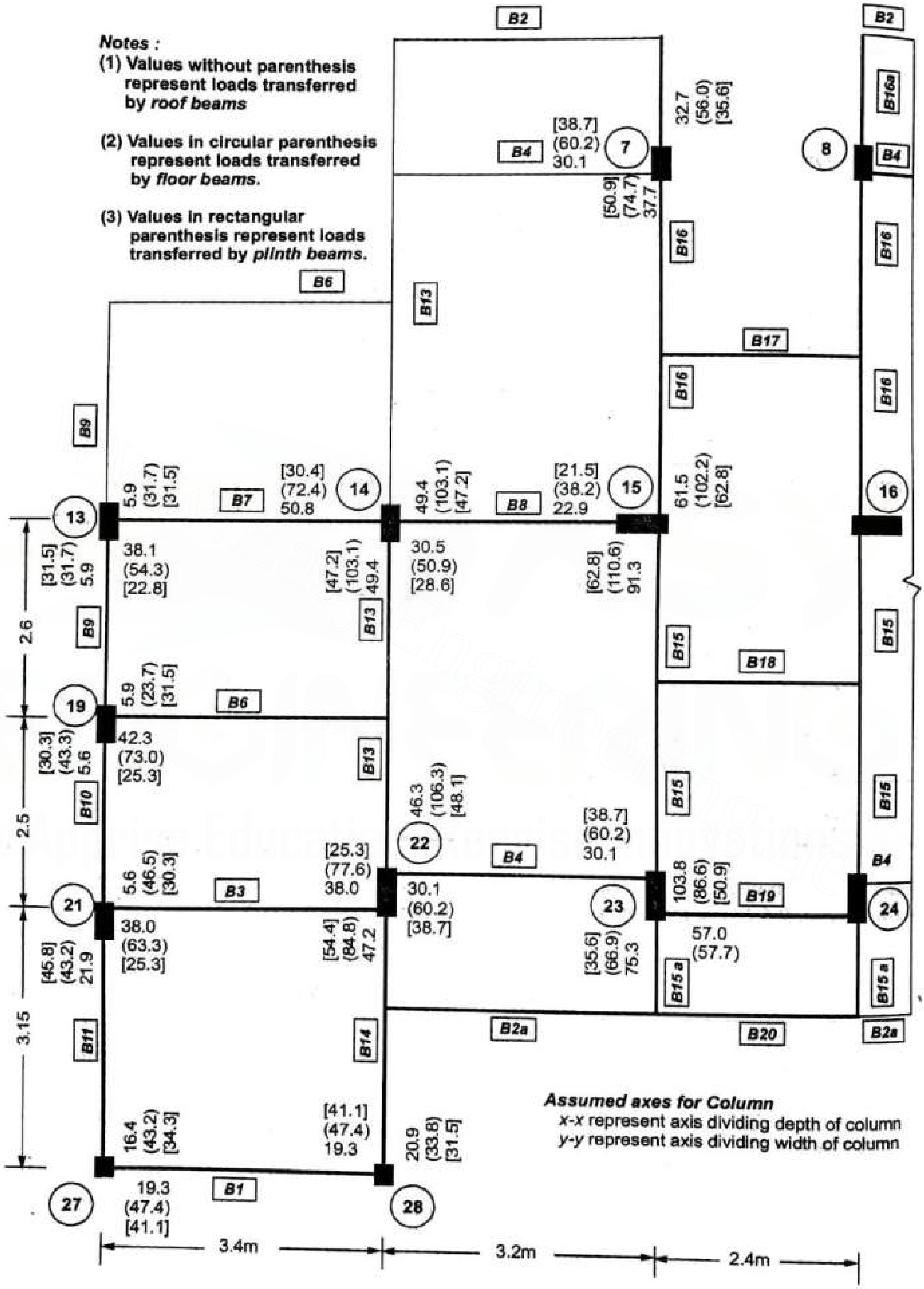


Fig. 9.6.1 Plan showing Beam end Shear transferred to columns

On the same lines the loads transferred from plinth beams is obtained from beam end shears as per *Step No. 8* of design of plinth beams and is shown in *Fig. 9.6.1 (c)*

COLUMNS IN FIRST STOREY

Loads transferred by Plinth Beams

Category	Column Mark	Beam end Forces in kN from directions				Total kN	Total Rounded P_f kN
		1	2	3	4		
I	C14	47.2	28.6	47.2	30.4	153.4	154
II	C7	35.6	-	50.9	38.7	125.2	126
	C13	31.5	22.8	31.5	-	85.8	86
	C15	62.8	-	62.8	21.5	147.1	148
	C19	31.5	25.3	30.3	-	87.1	88
	C21	30.3	25.3	45.8	-	101.4	102
	C22	48.1	38.7	54.4	25.3	166.5	167
	C23	50.9	-	35.6	38.7	125.2	126
III	C27	34.3	41.1	-	-	75.4	76
	C28	31.5	-	-	41.1	72.6	73

9.6.3 Determination of Effective Length and Slenderness

The unsupported lengths of the columns is first computed from the floor to floor height.

		Top Storey	Middle Storey	Bottom Storey
Floor to floor height	=	3000 *mm*	3000 *mm*	*3450 *mm*
Depth of shallowest beam	=	300 *mm*	380 *mm*	380 *mm*
Unsupported Length L	=	2700 *mm*	2620 *mm*	3070 *mm*

For bottom storey the unsupported length will be more by 450 *mm*. , because of height of plinth above ground level.

Since all columns are laterally supported by walls, they can be assumed to be braced and column ends not free to sway. When exact calculations for rotation release factors β_1 and β_2 are not done, L_{eff} for top storey can be taken equal to 1.2L when its top end is not held in position. However, it is not totally free hence L_{eff} for top storey columns may be assumed to be equal to L and for intermediate storeys L_{eff} may be approximately taken equal to 0.8 L. Thus, depending on the ratio of L_{eff}/b for columns in different storeys having widths of 200 *mm* or 230 *mm* the column type is decided.

The allowance for slenderness has been approximately arrived at from reduction coefficient used in working stress method.

Reduction coefficient $C_r = 1.25 - L_{eff}/(48b) = 1.25 - (L_{eff}/b)/48$

$\therefore$ Allowance for slenderness in %= $(1/C_r - 1) \times 100$

The allowances for different columns have been worked out in the following tables.

Details of Column Type and Allowances for Slenderness for Columns in different storeys.

			Top Storey	Middle Storey	Bottom Storey
Column length	L_{cc}	*mm*	3000	3000	3450
Minimum beam depth	D	*mm*	300	380	380
Unsupported Length	L	*mm*	2700	2620	3070
	L_{eff}		2700	2096	2456
Width of column	b	*mm*	200/230	200/230	200/230
	L_{eff}/b		13.5/11.7	10.5/9.1	12.3/10.7
Column type			Long/Short	Short/Short	Long/Short
Allowance for slenderness %			say 4%/Nil	Nil/Nil	0.6%(neglected)/Nil

To account for the effect of slenderness these allowances have been taken in obtaining the equivalent loads on long columns.

9.6.4 Calculation of Column Loads in Each Storey

Loads obtained in each storey in *Sect. 9.6.2* are compiled and loads at each storey level are worked out.

(A) Loads in each Storey in kN.

Storey	Category I	Category II							Category III	
	C14	C7	C13	C15	C19	C21	C22	C23	C27	C28
Top Storey P_r kN	181	101	50	281	54	66	162	372	36	41
Int. Storey P_f kN	330	191	118	251	140	153	329	272	91	82
1st Storey P_p kN	154	126	86	148	88	102	167	126	76	73

(B) Loads at each Floor Level in kN

Group	C14	C7	C13	C15	C19	C21	C22	C23	C27	C28
	I	II							III	
Top Storey P_r kN	181	101	50	281	54	66	162	372	36	41
3rd Storey $P_r + P_f$ kN	511	292	168	532	194	219	491	644	127	123
2nd Storey $P_r + 2P_f$ kN	841	483	286	783	334	372	820	916	218	205
1st Storey $P_r + 3P_f$ kN	1171	674	404	1034	474	525	1149	1188	309	287
Plinth $P_r + 3P_f + P_p$ kN	1325	800	490	1182	562	627	1316	1314	385	360

9.6.5 Calculations of Equivalent Design Axial Load

Since the design of column is assume and try process, initially the column section is selected for an equivalent axial load and then checked for the forces to which it is subjected. While selecting the section of the column the abrupt change in diameter of longitudinal steel should be avoided and the column may be increased/decreased in the module of 75 *mm* to 150 *mm*.

The equivalent axial design loads are computed by adding allowances for bending due to effect of fixity between beam and column, and allowances for slenderness. For columns subjected to uniaxial bending the allowance is taken equal to 15% and for biaxial bending 33% approximately. The allowance for slenderness is taken depending on the ratio of L_{eff} / b as detailed in *Sect. 9.6.3* The tentative column section is selected referring to *Table G-1* in Appendix.

Computation of Equivalent Axial Loads and Assumed Column Section

Description		C14	C15	C13, C19 C21	C22	C23	C7	C27, C28
(a) Column between Storey R-3								
P_r	kN	181	281	66	162	372	101	36
% Allowance for fixity		-	15%	15%	15%	15%	33%	
Allowance for fixity	kN	-	42	10	24	56	15	12
Total	kN	181	323	76	186	428	116	48
% Allowance for slenderness		4%	4%	4%	4%	4%	4%	4%
Value in	kN	8	13	3	8	17	5	2
Equivalent Axial Design Load	kN	189	336	79	194	445	121	50
Section $b \times D$ in	mm	230x200	230x200	200x230	200x600	200x600	200x230	200x230
N - #		4-#12	4-#12	4-#12	4-#16+ 2-#12	4-#16 +2-#12	4-#12	4-#12
Provided P_{ur}	kN	392	392	392	991	991	392	392

Computation of Equivalent Axial Load Continued...

Description	C14	C15	C13,C19,C21	C22	C23	C7	C27, C28
(b) Column between Storey 3-2							
P_f kN	330	251	153	329	272	191	91
Load on Column $P_r + P_f$ kN	511	532	219	491	644	292	127
% Allowance for fixity on P_f 0/15/33 on P_f	-	38	23	50	41	29	30
Equivalent Axial Design Load kN	511	570	242	541	685	321	157
Section $b \times D$	300 x 200	300 x 200	200 x 230	200 x 600	200 x 600	200 x 230	200 x 230
N - #	6-#12	4#16+2#12	4-#12	4-#16+2#12	4-#16+2#12	4#12	4-#12
Provided P_{ur} kN	531	607	392	991	991	392	392
ϕ - s							
(c) Column between Storey 2-1							
Load on Column $(P_r + 2P_f)$ kN	841	783	372	820	916	483	218
Allowance for fixity kN	-	38	23	50	41	29	30
Equivalent Axial Design Load kN	841	821	395	870	957	512	248
Section mm x mm	380 x 230	380 x 230	200 x 300	200 x 600	200 x 600	200 x 450	200 x 230
N - #	4#16+2#12	4#16+2#12	6-#12	4-#16+2-#12	4-#16+2-#12	4-#16	4-#12
Provided P_{ur}	880	880	531	991	991	750	392
ϕ - s							
(d) Column between Storey 1 - PL							
Load on Column $(P_r + 3P_f)$ kN	1171	1034	525	1149	1188	674	309
Allowance for fixity kN	-	38	23	50	41	29	30
Equivalent Axial Design Load kN	1171	1072	548	1199	1229	703	339
Section mm x mm	530 x 230	450 x 230	230 x 300	230 x 600	230 x 600	230 x 530	230 x 230
N - #	6#16+2#12	6-#16	6-#12	6-#16	4-#16+4-#12	4-#16+2#12	4-#12
Provided P_{ur} kN	1226	1038	662	1287	1299	1128	491
ϕ - s							
(e) Column between PL - Ft							
P_p kN	154	148	102	167	126	126	76
Load on Column $P_r + 3P_f + P_p$ kN	1325	1182	627	1316	1314	800	385
Allowance for fixity kN	-	22	16	25	19	19	25
Equivalent Axial Design Load kN	1325	1204	643	1341	1333	819	410
Section $b \times D$ mm x mm	600 x 230	600 x 230	230 x 300	230 x 600	230 x 600	230 x 600	230 x 230
N - #	6#16+2#12	6-#16+2#12	6-#12	6-#16+2-#12	6-#16+2-#12	6-#16	4-#12
Provided P_{ur} kN	1342	1342	662	1342	1342	1287	491
ϕ - s							

Note : *The width of the column in x - direction (i.e. horizontal direction) is written first and then in y - direction (i.e. vertical direction)*

Explanatory Notes :

@ Since the length of the wall between the centres of column is taken for computation of weight of wall, the self weight of column is not added.

\$ As the length of the column between ground level to top of footing is maximum 700 mm(= 1000–300) only the self weight of the column is neglected.

9.6.6 Check Column Section for Axial Load and Moment

(calculations are presented for typical columns from each group)

Once the tentative section is obtained based on equivalent axial load it is necessary to check the section for axial load and moment to which it is subjected. These checks are carried out in the following steps.

(a) The fixed end moments at beam-column junction are worked out first. The fixed end moments are taken equal to $w_u L^2 / 24$ at their discontinuous end of the beam and $w_u L^2 / 12$ at the continuous end. For continuous beams the values of the initial fixed end moments worked out earlier have been taken. *(eg. Beam B16-B15, B13-B14).*

(b) The fixed end moments are distributed between column and beams in proportion to their distribution factors.

(c) These moments are compared with minimum moments ($P_u \times e_{min}$) and the maximum value is taken. Finally the column is checked for axial load and the moments thus obtained.

The step wise calculations performed are as under :

(A) Calculation of storey wise stiffness of columns.
(B) Computations of floor wise stiffness of beam.
(C) Calculation of moments in each column.
(D) Summary of moments in each storey.
(E) Check the column section for combined axial load and bending.

9.6.7. Storey wise Stiffness of Columns

Note : (1) x-x is taken as major axis of bending dividing the Depth of the column.
(2) y-y is taken as minor axis of bending dividing the Width of the column.

	Category / Column Mark		III / C27	II / C23	II / C21	I / C14
	Storey					
a)	R - 3 : Section $b \times D$	mm	200 x 230	200 x 600	200 x 230	230 x 200
	L_{cc}	mm	3000	3000	3000	3000
	$I_x \times 10^{-6}$	mm^4	202.8	3600	202.8	202.8
	$I_x / L_{cc} = k_{r3x} \times 10^{-3}$	mm^3	67.6	1200	67.6	67.6
	$I_y \times 10^{6}$	mm^4	153.3	400	153.3	153.3
	$I_y / L_{cc} = k_{R3y} \times 10^{-3}$	mm^3	51.1	133.3	51.1	51.1
b)	3-2 : Section $b \times D$	mm	200 x 230	200 x 600	200 x 230	300 x 200
	L_{cc}	mm	3000	3000	3000	3000
	$I_x \times 10^{-6}$	mm^4	202.8	3600	202.8	450
	$I_x / L_{cc} = k_{32x} \times 10^{-3}$	mm^3	67.6	1200	67.6	150
	$I_y \times 10^{-6}$	mm^4	153.3	400	153.3	200
	$k_{32y} \times 10^{-3}$	mm^3	51.1	133.3	51.1	66.7
c)	2 - 1 : Section $b \times D$	mm	200 x 230	200 x 600	200 x 300	380 x 230
	L_{cc}	mm	3000	3000	3000	3000
	$I_x \times 10^{-6}$	mm^4	202.8	3600	450	1051.7
	$I_x / L_{cc} = k_{21x} \times 10^{-3}$	mm^3	67.6	1200	150	350.5
	$I_y \times 10^{-6}$	mm^4	153.3	400	200	385.3
	$k_{21y} \times 10^{-3}$	mm^3	51.1	133.3	66.7	128.4

(A) Storey wise Stiffness of Column continued...

Category / *Column Mark*		*III* *C27*	*II* *C23*	*C21*	*I* *C14*
Storey					
d) 1 - *PL* : Section $b \times D$	*mm*	230 x 230	230 x 600	230 x 300	530 x 230
L_{cc}	*mm*	3450	3450	3450	3450
$I_x \times 10^{-6}$	mm^4	233.2	4140	517.5	2853
$I_x / L_{cc} = k_{1px} \times 10^{-3}$	mm^3	67.6	1200	150	827
$I_y \times 10^{-6}$	mm^4	233.2	608.3	304.1	537.4
$k_{1py} \times 10^{-3}$	mm^3	67.6	176.3	88.1	155.7
e) *PL-FT* : Section $b \times D$	*mm*	230 x 230	230 x 600	230 x 300	600 x 230
L_{cc}	*mm*	1000	1000	1000	1000
$I_x \times 10^{-6}$	mm^4	233.2	4140	517.5	4140
$k_{PFx} \times 10^{-3}$	mm^3	67.6	4140	517.5	4140
$I_y \times 10^{-6}$	mm^4	233.2	608.3	304.1	608.3
$k_{PFy} \times 10^{-3}$	mm^3	67.6	608.3	304.1	608.3

9.6.8 Floor wise Stiffness of Beams

(A) Bending about x - axis dividing Depth of Column

Column Supporting Beam		*C27*		*C23*		*C21*		*C14*	
		Left	*Right*	*Left*	*Right*	*Left*	*Right*	*Left*	*Right*
(a) Roof Beams									
Beam Mark		-	**B11**	**B4**	-	**B11**	**B10**	**B7**	**B8**
Span L	*m*	-	3.15	3.2	-	3.15	2.5	3.4	3.2
Section $b \times D$	*mm*	-	200 x 300	200 x 300	-	200 x 300	200 x 300	200 x 300	200 x 300
Section type		-	*FL*	*FL*	-	*FL*	*FL*	*FL*	*FL*
$I_x \times 10^{-6}$	mm^4	-	900	900	-	900	900	900	900
$k_x \times 10^{-3}$	mm^3	-	285.7	281.2	-	285.7	360	264.7	281.2
Σk_x	mm^3	285.7	-	-	281.2	-	645.7	-	545.9
w_u	*kN/m*	-	13.95	21.2	-	13.95	8.4	24.9	20.7
M_{F1}, M_{F2}	*kN.m*	-	6.0	9.0	-	11.5	4.4	24.0	17.7
M_e		6.0	-	-	9.0	-	7.1	-	6.3
(b) 3rd Floor Beams		-	200 x 380	200 x 380	-	200 x 380	200 x 380	200 x 380	200 x 380
Section type $b \times D$		-	*FL*	*FL*		*FL*	*FR*	*FL*	*FL*
(c) 2nd Floor Beams Section type $b \times D$		-	200 x 380	200 x 380	-	200 x 380	200 x 380	200 x 380	200 x 380

Floor wise Stiffness of Beams : (Bending about x - axis dividing Depth of Column) Continued...

Column Supporting Beams		C27 Left	C27 Right	C23 Left	C23 Right	C21 Left	C21 Right	C14 Left	C14 Right
(d) 1st Floor Beam									
Beam Mark			B11	B15A	B15	B11	B10	B7	B8
Span L	m	-	3.15	1.47	4.7	3.15	2.5	3.4	3.2
Section b x D	mm	-	200 x 380	200 x 600	200x600	200 x 380	200 x 380	200 x 380	200x 380
Section type		-	FL	R	FL	FL	R	FL	FL
$I_x \times 10^{-6}$	mm^4	-	1829	3600	7200	1829	914	1829	1829
$k_x \times 10^{-3}$	mm^3	-	581	2449	1532	581	365.6	538	571.5
Σk_x	mm^3	-	581	3981	-	946.6	-	1109.5	-
w_u	kN/m	-	29.75	22.0	33.95/45.65	29.75	20.3+39(PL)	35.5	31.3
Point Load	kN			34.6	26.9				
M_{F1}, M_{F2}	$kN.m$	-	12.3	74.6	80.2	12.3	(5.3+6.3)	34.2	26.7
M_e		-	12.3	5.6	-	1.0		7.5	-
(e) Plinth Beam									
Beam Mark		-	B11	15A	B15	B11	B10	B7	B8
Span L	m	-	3.15	1.47	4.7	3.15	2.5	3.4	3.2
Section b x D	mm	-	200 x 380	200 x 380	200x380	200 x 380	200 x 380	200 x 380	200x 380
Section type		-	R	R	R	R	R	R	R
$I_x \times 10^6$	mm^4	-	914	914	914	914	914	914	914
$k_x \times 10^3$	mm^3	-	290	621.7	621.7	290	365.6	268.8	285.6
Σk_x	mm^3	-	290	816		655.6	-	554.4	-
w_u	kN/m	-	24.2	24.2	24.2	24.2	24.2	14.9	14.9
M_{F1}, M_{F2}	$kN.m$	-	10	26.1	26.1	20.0	12.6	14.3	12.7
M_e		-	10	18.5	-	7.4	-	1.6	-

(B) Bending about y - axis dividing width of Column

Column Supporting Beams		C27 Left	C27 Right	C23 Left	C23 Right	C21 Left	C21 Right	C14 Left	C14 Right
(a) Roof Beam									
Beam Mark		-	B1	B4	-	-	B3	B13	B13
Span L	m	-	3.4	3.2	-	-	3.4	4.7	4.7
Section b x D	mm	-	200 x 300	200 x 300		-	200 x 300	200 x 300	200 x 300
Section type		-	FL	FL	-	-	FL	FL	FL
$I_y \times 10^{-6}$	mm^4	-	900	3600	7200	-	900	900	900
$k_y \times 10^{-3}$	mm^3	-	265	281.2	-	-	265	191	191
Σk_y	mm^3	-	265	281.2	-	265	-	382	-
w_u	kN/m	-	13.61	21.2	-	-	25.35	17.95	17.95
Point Load	$kN.m$			-	-			42.33	42.33
M_{F1}, M_{F2}	$kN.m$	-	6.6	9.0	-		12.2	56.2	56.2
M_e		-	6.6	9.0	-	12.2	-	0.0	-

Floor wise Stiffness of Beams : (Bending about y - axis dividing width of Column) Continued...

Column Supporting Beam		C27		C23		C21		C14	
		Left	Right	Left	Right	Left	Right	Left	Right
(b) 3rd Floor Beams		-							
Section type $b \times D$		-	200 x 380	200 x 600	200 x 600	-	200 x 380	200 x 380	200 x 380
(c) 2nd Floor Beams									
Section $b \times D$		-	200 x 380	200 x 600	200 x 600	-	200 x 380	200 x 380	200 x 380
(d) 1st Floor Beams									
Beam Mark		-	B1	B4	-	-	B3	B13	B13
Span L	m	-	3.4	3.2	-	-	3.4	4.7	4.7
Section $b \times D$	mm	-	200 x 380	200 x 300	-	-	200 x 380	200 x 380	200x 380
Section type		-	FL	FL	-	-	FL	FL	FL
$I_y \times 10^6$	mm^4	-	1829	1829	-	-	1829	1829	1829
$k_y \times 10^{-3}$	mm^3	-	538	571.6	-	-	538	389	389
Σk_y	mm^3	-	538	571.6	-	538	-	778	-
w_u	kN/m	-	30.4	40		-	32.4	28.95	28.95
Point Load	kN	-	-	-	-		39.2	86	86
M_{F1}, M_{F2}	kN.m	-	14.6	17.1	-	-	(15.6+4.6=)20.2	100.3	100.3
M_e		-	14.6	17.1	-	20.2	-	0.0	-
(e) Plinth									
Beam Mark		-	B1	B4	-	-	B3	B13	B13
Span L	m	-	3.4	3.2	-	-	3.4	4.7	4.7
Section $b \times D$	mm	-	200 x 380	200 x 380	-	-	200 x 380	200 x 380	200x 380
Section type		-	R	R	-	-	R	R	R
$I_y \times 10^6$	mm^4	-	914	914	-	-	914	914	914
$k_y \times 10^6$	mm^3	-	269	285.6	-	-	269	194.4	194.4
Σk_y	mm^3	-	269	285.6	-	269	-	389	-
w_u	kN/m	-	24.2	24.2	-	-	14.9	14.9	14.9
M_{F1}, M_{F2}	kN.m	-	11.6	10.3	-	-	7.2	41.3	41.3
M_e		11.6	-	10.3	-	7.2	-	0.0	-

9.6.9 Calculation of Moments in Column at Each Floor Level

(i) Bending about x - axis		C27	C23	C21	C14
(a) Roof :					
Connected Beams		B11	B15a & B15	B11 and B10	B7and B8
$\Sigma k_b \times 10^3$	mm^3	285.7	3981	645.7	545.9
Column $k_{R3x} \times 10^3$	mm^3	67.6	1200	67.6	67.6
$\Sigma k_{R3x} + k_b/2 = \Sigma k \times 10^3$	mm^3	210.4	3191	390.5	340.6
$d_{col.R3}$		0.32	0.376	0.173	0.198
M_e	kN.m	6.0	11.54	7.1	6.3
$M_{col.R3}$	kN.m	2.84	4.34	1.23	1.25

Calculation of Moments in Column at Each Floor Level Continued...
Bending about x - axis

		C27	C23	C21	C14
(b) 3rd Floor $\Sigma k_b \times 10^3$	mm^3	581	3981	946.6	1109.5
Column $k_{R3x} \times 10^3$	mm^3	67.6	1200	67.6	67.6
Column $k_{32} \times 10^3$	mm^3	67.6	1200	67.6	150
$\Sigma k \times 10^3$	mm^3	425.7	4391	608.3	772.3
$d_{col.3R}$		0.159	0.273	0.11	0.09
$d_{col.\ 3\text{-}2}$		0.159	0.273	0.11	0.194
M_e	kN.m	12.3	5.6	1.0	7.5
$M_{col.\ 3R}$	kN.m	1.96	1.53	0.11	0.67
$M_{col.32}$	kN.m	1.96	1.53	0.11	1.46
(c) 2nd Floor					
$\Sigma k_b \times 10^3$	mm^3	581	3981	946.6	1109.5
Column $k_{32x} \times 10^3$	mm^3	67.6	1200	67.6	150
Column $k_{21.x} \times 10^3$	mm^3	67.6	1200	150	350.5
$\Sigma k \times 10^3$	mm^3	425.7	4391	608.31	1055.2
$d_{col.23}$		0.159	0.273	0.11	0.142
$d_{col.21}$		0.159	0.273	0.11	0.332
M_e	kN.m	12.3	5.6	1.0	7.5
$M_{col.23}$	kN.m	1.96	1.53	0.11	1.07
$M_{col.21}$	kN.m	1.96	1.53	0.11	2.49
(d) 1st Floor					
$\Sigma k_b \times 10^3$	mm^3	581	3981	946.6	1109.5
Column $k_{21x} \times 10^3$	mm^3	67.6	1200	150	350.5
Column $k_{1PL} \times 10^3$	mm^3	67.6	1200	150	827
$\Sigma k \times 10^3$	mm^3	425.7	4391	773.3	1732.2
$d_{col.12}$		0.16	0.273	0.2	0.2
$d_{col.PFT}$		0.16	0.273	0.2	0.48
M_e	kN.m	12.3	5.6	1.0	7.5
$M_{col.1\text{-}2}$	kN.m	2.0	1.53	0.2	1.5
$M_{col.1\text{-}P_L}$	kN.m	2.0	1.53	0.2	3.58
(e) PL-FT					
$\Sigma k_b \times 10^3$	mm^3	290	816	655.6	554.4
Column $k_{1PLx} \times 10^3$	mm^3	67.6	1200	150	827
Column $k_{PFx} \times 10^3$	mm^3	67.6	4140	517.5	4140
$\Sigma k \times 10^3$	mm^3	280.2	5748	995.3	5244.2
$d_{col.PL_1}$		0.24	0.209	0.15	0.157
$d_{col.PF_T}$		0.24	0.72	0.52	0.79
M_e	kN.m	10.0	18.5	7.4	1.6
$M_{col.P_L\text{-}1}$	kN.m	2.4	0.39	1.11	0.25
$M_{col.PL\ -\ F_T}$	kN.m	2.4	13.32	3.85	1.26

Calculation of Moments in Column at Each Floor Level Continued...

(ii) Bending about y - axis

		C27	C23	C21	C14
(a) Roof : Connected Beams		B1	B4	B3	B13&B13
Beam $\Sigma\, k_b \times 10^3$	mm^3	265	281.2	265	382
Column $k_{R3y} \times 10^3$	mm^3	51.1	133.3	51.1	51.1
$\Sigma k = \Sigma k_c + \Sigma k_b /2 \times 10^3$	mm^3	183.6	273.9	183.6	242.1
$\dfrac{\Sigma k_c}{(\Sigma k_c + \Sigma k_b /2)} =$	$d_{col.R3}$	0.278	0.487	0.278	0.21
M_e	$kN.m$	6.6	9.0	12.2	0.0
$M_{col.R3}$	$kN.m$	1.84	4.38	3.39	0
(b) 3rd Floor					
$\Sigma\, k_b \times 10^3$	mm^3	538	571.6	538	778
Column $k_{R3.y} \times 10^3$	mm^3	51.1	133.3	51.1	51.1
Column $k_{32.y} \times 10^3$	mm^3	51.1	133.3	51.1	66.7
$\Sigma k \times 10^3$	mm^3	371.2	552.4	371.2	506.8
$d_{col.3R}$		0.138	0.24	0.137	0.1
$d_{col.32}$		0.138	0.24	0.137	0.13
M_e	$kN.m$	14.6	17.1	20.2	0.0
$M_{col.3R}$	$kN.m$	2.0	4.1	2.77	0.0
$M_{col.32}$	$kN.m$	2.0	4.1	2.77	0.0
(c) 2nd Floor					
$\Sigma k_b \times 10^3$	mm^3	538	571.6	538	778
Column $k_{32y} \times 10^3$	mm^3	51.1	133.3	51.1	66.7
Column $k_{21.y} \times 10^3$	mm^3	51.1	133.3	66.7	128.4
$\Sigma k \times 10^3$	mm^3	371.2	552.4	386.8	584.1
$d_{col.23}$		0.138	0.24	0.132	0.114
$d_{col.21}$		0.138	0.24	0.17	0.22
M_e	$kN.m$	14.6	17.1	20.2	0.0
$M_{col.23}$	$kN.m$	2.00	4.1	2.67	0.0
$M_{col.21}$	$kN.m$	2.00	4.1	3.43	0.0
(d) 1st Floor					
$\Sigma k_b \times 10^3$	mm^3	538	571.6	538	778
Column $k_{21y} \times 10^3$	mm^3	51.1	133.3	66.7	128.4
Column $k_{1PL.y} \times 10^3$	mm^3	67.6	176.3	88.1	155.7
$\Sigma k \times 10^3$	mm^3	387.7	595.4	423.8	673.1
d_{col12}		0.132	0.220	0.158	0.19
$d_{col.1PL}$		0.174	0.296	0.21	0.23
M_e	$kN.m$	14.6	17.1	20.2	0.0
$M_{col.12}$	$kN.m$	1.93	3.76	3.2	0.0
$M_{col.1PL}$	$kN.m$	2.54	5.06	4.24	0.0

Calculation of Moments in Column at Each Floor Level Continued...

(ii) Bending about y - axis

		C27	C23	C21	C14
(e) PL-FT					
$\Sigma k_b \times 10^3$	mm^3	269	285.6	269	389
Column $k_{1PL.y} \times 10^3$	mm^3	67.6	176.3	88.1	155.7
Column $k_{PF.y} \times 10^3$	mm^3	67.6	608.3	304.1	608.3
$\Sigma k \times 10^3$	mm^3	269.7	927.4	526.7	958.5
$d_{col.PL1}$		0.25	0.19	0.167	0.162
$d_{col.PLF}$		0.25	0.656	0.577	0.635
M_e	kN.m	11.6	10.3	7.2	0.0
$M_{col.PL1}$	kN.m	2.9	1.96	1.2	0.0
$M_{col.PLF}$	kN.m	2.9	6.75	4.15	0.0

9.6.10 Summary of Moments in Each Floor

(i) Bending about y - Axis

Floor	Between		C27	C23	C21	C14
(a) Roof	(R-3)	kN.m	1.84	4.38	3.39	0.0
3rd Floor	(3-R)	kN.m	2.0	4.10	2.77	0.0
(b) 3rd Floor	(3-2)	kN.m	2.0	4.10	2.77	0.0
2nd Floor	(2-3)	kN.m	2.0	4.10	2.67	0.0
(c) 2nd Floor	(2-1)	kN.m	2.0	4.10	3.43	0.0
1st Floor	(1-2)	kN.m	1.93	3.76	3.2	0.0
(d) 1st Floor	(1-PL)	kN.m	2.54	5.06	4.24	0.0
Plinth	(PL-1)	kN.m	2.9	1.96	1.2	0.0
(e) Plinth	(PL-FT)	kN.m	2.9	6.75	4.15	0.0
Footing	(FT-PL)	kN.m	0.00	0.00	0.00	0.0

(ii) Bending about x - Axis

Floor	Between		C27	C23	C21	C14
(a) Roof	(R-3)	kN.m	2.84	4.34	1.23	1.25
3rd Floor	(3-R)	kN.m	1.96	1.53	0.11	0.67
(b) 3rd Floor	(3-2)	kN.m	1.96	1.53	0.11	1.46
2nd Floor	(2-3)	kN.m	1.96	1.53	0.11	1.07
(c) 2nd Floor	(2-1)	kN.m	1.96	1.53	0.11	2.49
1st Floor	(1-2)	kN.m	2.0	1.53	0.2	1.5
(d) 1st Floor	(1-PL)	kN.m	2.0	1.53	0.2	3.58
Plinth	(PL-1)	kN.m	2.4	0.39	1.11	0.25
(e) Plinth	(PL-FT)	kN.m	2.4	13.32	3.85	1.26
Footing	(FT-PL)	kN.m	0.00	0.00	0.00	0.0

The Ultimate Axial compression, ultimate moment about *x - axis*, dividing the depth of the column, ultimate moment about *y-axis* dividing width of the column are complied in *Table 9.6.1*

Table 9.6.1

Ultimate Axial Compression, Ultimate Moment about *x - axis*, Ultimate moment about *y-axis*

	C27			C23			C21			C14		
Level	P_u	M_{ux}	M_{uy}	P_u	M_{ux}	M_{uy}	P_u	M_{ux}	M_{uy}	P_u	M_{ux}	M_{uy}
Roof Level	36	2.84	1.84	372	4.34	4.38	66	1.23	3.39	181	1.25	0.0
Section Assumed	*200 x 230 4-#12*			*200 x 600 4-#16+2-#12*			*200 x 230 4-#12*			*200 x 230 4-#12*		
Top of 3rd Floor		1.96	2.0		1.53	4.10		0.11	2.77		0.67	0.0
3rd Floor *Bottom of 3rd Floor*	127	1.96	2.0	644	1.53	410	219	0.11	2.77	511	1.46	0.0
Section Assumed	*200 x 230 4-#12*			*200 x 600 4-#16+2-#12*			*200 x 230 4-#12*			*200 x 300 6-#12*		
Top of 2nd Floor		1.96	2.0		1.53	4.10		0.11	2.67		1.07	0.0
2nd Floor *Bottom of 2nd Floor*	218	1.96	2.0	916	1.53	4.10	372	0.11	3.43	841	2.49	0.0
Section Assumed	*200 x 230 4-#12*			*200 x 600 4-#16+2-#12*			*200 x 300 6-#12*			*230 x 380 4-#16+2-#12*		
Top of 1st Floor		2.0	1.93		1.53	3.76		0.2	3.2		1.5	0.0
1st Floor *Bottom of 1st Floor*	309	2.0	2.54	1188	1.53	5.06	525	0.2	4.24	1171	3.58	0.0
Section Assumed	*230 x 230 4-#12*			*230 x 600 4-#16+4-#12*			*230 x 300 6-#12*			*230 x 530 6-#16+2-#12*		
Top of Plinth		2.4	2.9		0.39	1.96		1.11	1.2		0.25	0.0
Plinth *Bottom of Plinth*	385	2.4	2.9	1314	13.32	6.75	627	3.85	4.15	1325	1.26	0.0
Section Assumed	*230 x 230 4-#12*			*230 x 600 6-#16+2-#12*			*230 x 300 6-#12*			*230 x 600 6-#16+2-#12*		
Footing	385	0.0	0.0	1314	0.0	0.0	627	0.0	0.0	1325	0.0	0.0

9.6.11 Design of Column for Axial Load and Moment in Each Storey

(a) Column Between Storey R-3

(i) Bending about y-axis

Column Nos.			C27, C28	C23	C19,C13,C21	C14
Section	$b \times D$	mm	200 x 230	200 x 600	200 x 230	200 x 230
No - Dia		N-#	4-#12	4-#16+2-#12	4-#12	4-#12
Area		mm^2	452.4	1030.4	452.4	452.4
p%		%	0.98	0.86	0.98	0.98
Floor to Floor height	L_{cc}	mm	3000	3000	3000	3000
Depth of Beam at Top		mm	300	600	300	300
Unsupported Length	L	mm	2700	2400	2700	2700
Minimum Eccentricity	$e_{.miny}$	mm	20	24.8	20	20
Axial Load	P_u	kN	**36**	**372**	**66**	**181**
$M_{uy.min}$		kN.m	0.72	7.44	1.32	3.62
e_{miny}/b			0.1	0.1	0.1	0.1
M_{uy} at top (R-3)		kN.m	1.84	4.38	3.39	0.0
M_{uy} at bottom (3-R)		kN.m	2.00	4.10	2.77	0.0
Initial Moment	M_i	kN.m	**2.00**	**7.44**	**3.39**	**3.62**
Effective Length	L_{eff}	mm	2700	2700	2700	2700
L_{eff}/b			13.5	13.5	13.5	13.5
Column Type			**Long**	**Long**	**Long**	**Long**
Additional moment due to Slenderness						
$P_{uz} = 0.45 f_{ck} A_g + (0.75 f_y - 0.45 f_{ck}) A_{sc}$		kN	550.7	1391	550.7	550.7
$P_{ub} = (k_1 + k_2 p / f_{ck}) f_{ck} bD$		kN	*152	*396	*152	*152
$k = \dfrac{P_{uz} - P_u}{P_{uz} - P_{ub}} \ngtr 1$			1	1	1	0.92
Additional Moment		kN.m				
$M_a = (P_u b/2000) \times (L_{eff}/b)^2 \times k$			0.656	6.78	1.21	3.03
Design Moment ($M_i + M_a$)		kN.m	**1.96**	**14.2**	**4.6**	**6.65**
d_c/b			0.26	0.26	0.26	0.26
$P_u/(f_{ck} \times b \times D)$			0.039	0.155	0.072	0.197
p/f_{ck}			0.049	0.043	0.049	0.049
$M_{ury} / (f_{ck} b^2 D)$	Chart 5-G		$0.067	$0.089	$0.008	$0.087
Provided	M_{ury}	kN.m	12.3	128.2	14.7	16.0(15.5)@

* value obtained by extrapolation

@ The exact value is 15.5 *kN.m* as per Software developed by the Author[9.1].

$ Values obtained from *Chart 5-G*

Column Between Storey R-3

(ii) Bending about x-axis

Column Nos.			C27	C23	C21	C14
Section	$b \times D$	mm	200 x 230	200 x 600	200 x 230	200 x 230
No - Dia		N-#12	4-#12	4-#16+2-#12	4-#12	4-#12
Area		mm^2	452.4	1030.4	452.4	452.4
$p\%$		%	0.98	0.86	0.98	0.98
Floor to Floor height	L_{cc}	mm	3000	3000	3000	3000
Depth of Beam at Top		mm	300	600	300	300
Unsupported Length	L	mm	2700	2400	2700	2700
Minimum Eccentricity	e_{minx}	mm	20	20	20	20
e_{minx}/D			0.087	0.041	0.087	0.087
Axial Load		kN	**36**	**372**	**66**	**181**
$M_{ux.min}$		kN.m	0.72	7.44	1.32	3.62
M_{ux} at top (R-3)		kN.m	2.84	4.34	1.23	1.25
M_{ux} at bottom (3-R)		kN.m	1.96	1.53	0.11	0.67
Initial Moment	M_i	kN.m	**2.84**	**7.44**	**1.32**	**3.62**
Effective Length	L_{eff}	mm	2700	2700	2700	2700
L_{effx}/D			11.7	4.0	11.7	11.7
Column Type			Short	Short	Short	Short
Design Moment	M_{ux}	kN.m	**2.84**	**7.44**	**1.32**	**3.62**
d_c/D			0.226	0.09	0.226	0.226
$P_u/(f_{ck} \times b \times D)$			0.039	0.155	0.072	0.197
p/f_{ck}			0.049	0.043	0.049	0.049
$M_{urx} / (f_{ck}/bD)^2$			0.071	0.105	0.80	0.092
Provided M_{urx} (Chart 4-G and 5-G)		kN.m	15.02	*151	$16.9	$19.4
P_u/P_{uz}			0.065	-	-	-
α_n			1	-	-	-
$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} = \left(\frac{2.84}{15.02}\right) + \left(\frac{2.656}{13.8}\right)$			@0.38	-	-	-

Explanatory Notes :

(1) @ For slender corner column *C27*, subjected to bi-axial bending, the modified initial moment obtained by using equation $(0.6\ M_{u2} - 0.4\ M_{u1}) + 0.4\ M_{u2}$ has not being worked out because the value is small compared to $M_{uy.min}$
Also the value obtained by interaction equation is far less than unity there is no scope to reduce either the size of the column *or* steel, since the section provided is smallest.

(2) * The large size of the column *C23* is governed by the requirements of layout and is provided with minimum reinforcement.

(3) $ In the case of column subjected to uniaxial bending if $e_{miny}/b > e_{minx}/D$ the design is governed by the minimum moment about *y-axis*.[9.2]

(b) Column Between Storey 3-2

(i) Bending about y-axis

Column Nos.			C27	C23	C21	C14
Section	$b \times D$	mm	200 x 230	200 x 600	200 x 230	200 x 300
No - Dia		N-#	4-#12	4-#16+2-#12	4-#12	6-#12
Area		mm^2	452.4	1030.4	452.4	678
$p\%$		%	0.98	0.86	0.98	1.13
Floor to Floor height	L_{cc}	mm	3000	3000	3000	3000
Depth of Beam at Top		mm	380	600	380	380
Unsupported Length	L	mm	2620	2400	2620	2620
Minimum Eccentricity	e_{miny}	mm	20	20	20	20
e_{miny}/b			**0.1**	**0.1**	**0.1**	**0.1**
Axial Load	P_u	kN	**127**	**644**	**219**	**511**
$M_{uy.min}$		kN.m	2.54	12.88	4.38	10.22
M_{uy} at top		kN.m	2.0	4.1	2.77	0.0
M_{uy} at bottom		kN.m	2.0	4.1	2.67	0.0
Initial Moment	M_i	kN.m	2.54	12.88	4.38	10.22
Effective Length	L_{eff}	mm	2096	1920	2096	2096
L_{effx}/b			10.5	9.6	10.5	10.5
Column Type			Short	Short	Short	Short
Design Moment	M_{uy}	kN.m	**2.54**	**12.88**	**4.38**	**10.22**
d_c/b			0.26	0.27	0.26	0.26
$P_u/(f_{ck} \times b \times D)$			0.138	0.268	0.238	0.426
p/f_{ck}			0.049	0.043	0.049	0.0565
$M_{ury} / (f_{ck}/b^2D)$	*Chart 5-G*		0.09	0.08	0.087	0.06
Provided	M_{ury}	kN.m	16.56	38.4	16.0	14.4

(ii) Bending about x-axis

Column Nos.			C27	C23	C21	C14
Section	$b \times D$	mm	200 x 230	200 x 600	200 x 230	200 x 300
No - Dia		N-#	4-#12	4-#16+2-#12	4-#12	6-#12
$p\%$		%				
Floor to Floor height	L_{cc}	mm	3000	3000	3000	3000
Depth of Beam at Top		mm	380	600	380	380
Unsupported Length	L	mm	2620	2400	2620	2620
Minimum Eccentricity	e_{minx}	mm	20	24.8	20	20
e_{minx}/D			**0.087**	**0.041**	**0.087**	**0.067**
Axial Load	P_u	kN	**127**	**644**	**219**	**511**
$M_{ux.min}$		kN.m	2.54	15.97	4.38	10.22
M_{ux} at top		kN.m	1.96	1.53	0.11	1.46
M_{ux} at bottom		kN.m	1.96	1.53	0.11	1.07
Initial Moment	M_i	kN.m	2.54	15.97	4.38	10.22
Effective Length	$L_{eff} = 0.8L$	mm	2096	1920	2096	2096
L_{effx}/D			9.1	3.2	9.1	6.9
Column Type			Short	Short	Short	Short

Note : Since $e_{miny}/b > e_{minx}/D$ therefore, ***bending about y-axis governs***[9.2].

(c) Column Between Storey 2-1

(i) Bending about y-axis

Column Nos.			C27	C23	C21	C14
Section	$b \times D$	mm	200 x 230	200 x 600	200 x 300	230 x 380
No - Dia		N-#	4-#12	4-#16 + 2-#12	6-#12	4-#16+2-#12
Area		mm^2	452.4	1030.4	678.6	1030.4
p%		%	0.98	0.86	1.13	1.18
Floor to Floor height	L_{cc}	mm	3000	3000	3000	3000
Depth of Beam at Top		mm	380	600	380	380
Unsupported Length	L	mm	2620	2400	2620	2620
Minimum Eccentricity	e_{miny}	mm	20	20	20	20
e_{miny}/b			0.1	0.1	0.1	0.1
Axial Load	P_u	kN	**218**	**916**	**372**	**841**
$M_{uy.min}$		$kN.m$	4.36	18.32	7.44	16.82
M_{uy} at top		$kN.m$	2.0	4.1	3.43	0.0
M_{uy} at bottom		$kN.m$	1.93	3.76	3.2	0.0
Initial Moment	M_i	$kN.m$				
Effective Length	L_{eff}	mm	2096	1920	2096	2096
L_{effx}/b			10.48	9.6	10.48	9.11
Column Type			Short	Short	Short	Short
Design Moment	M_{uy}	$kN.m$	4.36	18.32	7.44	16.82
d_c/b			0.26	0.27	0.26	0.23
$P_u/(f_{ck} \times b \times D)$			0.24	0.38	0.31	0.48
p/f_{ck}			0.049	0.043	0.056	0.059
$M_{ury}/(f_{ck} b^2 D)$			0.087	0.062	0.08	0.05
Provided	M_{ury}	$kN.m$	16.0	29.76	19.2	20.1

(ii) Bending about x-axis

Column Nos.			C27	C23	C21	C14
Section	$b \times D$	mm	200 x 230	200 x 600	200 x 300	230 x 380
No - Dia		N-#	4-#12	4-#16 + 2-#12	6-#12	4-#16+2-#12
Area		mm^2	452.4	1030.4	678.6	1030.4
Floor to Floor height	L_{cc}	mm	3000	3000	3000	3000
Depth of Beam at Top		mm	380	600	380	380
Unsupported Length	L	mm	2620	2400	2620	2620
Minimum Eccentricity	e_{minx}	mm	20	24.8	20	20
e_{minx}/D			0.087	0.04	0.087	0.0526
Axial Load	P_u	kN	**218**	**916**	**372**	**841**
$M_{ux.min}$		$kN.m$	4.36	22.71	7.44	16.82
M_{ux} at top		$kN.m$	1.96	1.53	0.11	2.49
M_{ux} at bottom		$kN.m$	1.95	1.53	0.2	1.5
Initial Moment	M_i	$kN.m$	4.36	22.71	7.44	16.82
Effective Length	L_{eff}	mm	2096	1920	2096	2096
L_{effx}/D			9.1	3.2	7.0	5.5
Column Type			Short	Short	Short	Short

Note : Since $e_{miny}/b > e_{minx}/D$ therefore, **bending about *y-axis* governs.**

(d) Column Between 1-PL

(i) Bending about y-axis

Column Nos.			C27	C23	C21	C14
Section $b \times D$		mm	230 x 230	230 x 600	230 x 300	230 x 530
No - Dia		N-#	4-#12	4-#16+4-#12	6-#12	6-#16+2-#12
Area		mm^2	452.4	1256.6	678.6	1432.5
$p\%$		%	0.85	0.91	0.98	1.17
Floor to Floor height	L_{cc}	mm	3450	3450	3450	3450
Depth of Beam at Top		mm	380	600	380	380
Unsupported Length	L	mm	3070	2850	3070	3070
Minimum Eccentricity	e_{miny}	mm	20	20	20	20
e_{miny}/b			0.087	0.087	0.087	0.087
Axial Load	P_u	kN	**309**	**1188**	**525**	**1171**
$M_{uy.min}$		kN.m	6.18	23.8	10.5	23.42
M_{uy} at top		kN.m	2.54	5.06	4.24	0.0
M_{uy} at bottom		kN.m	2.9	1.96	1.2	0.0
Initial Moment	M_i	kN.m	6.18	23.8	10.5	23.42
Effective Length	L_{eff}	mm	2456	2280	2456	2456
L_{effy}/b			10.7	9.9	10.7	10.7
Column Type			Short	Short	Short	Short
Design Moment	M_{uy}	kN.m	6.18	23.8	10.5	23.42
d_c/b			0.226	0.235	0.226	0.235
$P_u/(f_{ck} \times b \times D)$			0.292	0.43	0.38	0.48
p/f_{ck}			0.043	0.046	0.05	0.058
$M_{ury}/(f_{ck}\, b^2 D)$			0.079	0.058	0.069	0.047
Provided	M_{ury}	kN.m	19.2	36.8	21.9	26.4

(ii) Bending about x-axis

Column Nos.			C27	C23	C21	C14
Section	$b \times D$	mm	230 x 230	230 x 600	230 x 300	230 x 530
No - Dia		N-#	4-#12	4-#16+4-#12	6-#12	6-#16+2-#12
Area		mm^2	452.4	1256.6	678.6	1030.4
Floor to Floor height	L_{cc}	mm	3450	3450	3450	3450
Depth of Beam at Top		mm	380	600	380	380
Unsupported Length		mm	3070	2850	3070	3070
Minimum Eccentricity	e_{minx}	mm	20	25.7	20	23.8
e_{minx}/D			0.087	0.04	0.067	0.045
Axial Load	P_u	kN	**309**	**1188**	**525**	**1171**
$M_{ux.min}$		kN.m	6.18	30.53	10.5	27.86
M_{ux} at top		kN.m	2.0	1.53	0.2	3.58
M_{ux} at bottom		kN.m	2.4	0.39	1.11	0.25
Initial Moment	M_i	kN.m	6.18	30.53	10.5	27.86
Effective Length	L_{eff}	mm	2456	2280	2456	2456
L_{effx}/D			10.7	3.8	8.2	4.6
Column Type			Short	Short	Short	Short

***Note :** Since $e_{miny}/b > e_{minx}/D$ therefore, bending about y-axis governs.*

(e) Column Between PL-Ft

(i) Bending about y-axis

Column Nos.			C27	C23	C21	C14
Section	b x D	mm	230 x 230	230 x 600	230 x 300	230 x 600
No - Dia		N-#	4-#12	6-#16 + 2-#12	6-#12	6-#16+2-#12
Area		mm^2	452.4	1432.5	678.6	1432.5
p%		%	0.85	1.04	0.98	1.04
Floor to Floor height	L_{cc}	mm	1000	1000	1000	1000
Depth of Beam at Top		mm				
Unsupported Length	L	mm	1000	1000	1000	1000
Minimum Eccentricity	e_{miny}	mm	20	20	20	20
e_{miny}/b			0.087	0.087	0.087	0.087
Axial Load		kN	**385**	**1314**	**627**	**1325**
$M_{uy.min}$		kN.m	7.7	26.28	12.5	26.5
M_{uy} at top		kN.m	2.9	6.75	4.15	0.0
M_{uy} at bottom		kN.m	0.0	0.0	00	0.0
Initial Moment	M_i	kN.m	7.7	26.28	12.5	26.5
Effective Length	L_{eff}	mm	1000	1000	1000	1000
L_{effy}/b			4.35	4.35	4.35	4.35
Column Type			Short	Short	Short	Short
Design Moment	M_{uy}	kN.m	7.7	26.28	12.5	26.5
d_c/b			0.226	0.235	0.226	0.235
$P_u/(f_{ck} \times b \times D)$			0.36	0.476	0.454	0.48
p/f_{ck}			0.043	0.052	0.049	0.052
$M_{ur} / (f_{ck}\, b^2 D)$			0.68	0.046	0.05	0.045
Provided	M_{ury}	kN.m	16.6	29.20	15.87	28.56

Explanatory Notes :

It can be seen that the tentative section assumed has worked out to be safe. However, it may not be so for all cases and therefore detailed calculations should be carried out to check the assumed section.

All the values of moment of resistance of the section about y-axis have been checked with the results obtained from the software developed by the author[9.1]. They approximately tally with the exact values.

9.6.12 Approximate Method of Computation of Loads on Columns :

(a) When a structure cannot be divided into frame due to unsymmetrical positions of columns and when footing details are required prior to the design of structural elements, the approximate method of calculation of loads is used. Based on these loads the tentative size of the footing is designed.

This approximate method of computation of loads is illustrated below.

Unit ultimate Loads :

For residential building the unit ultimate loads are given in *Appendix - A-3*. They are reproduced for ready reference.

(a) R.C.C. slab and Beam

Roof / Floor Slab	9.0 kN/m^2
Cantilever balcony	12.0 kN/m^2
Simply Supported balcony	10.0 kN/m^2
Bath / W.C.	10.5 kN/m^2
Loft	1.5 $kN/m/m$
Grill	3.0 kN
R.C.C. parapet 80 mm thick 1.7 m high	5.1 kN
Stairs 25 x 0.14 x 1.2 + 25 x 0.167/2 + 1 + 3 =	15.4 kN/m

(b) Self weight of flanged beam - weight of rib (assuming D_f = 100 mm)

Size of beam	200 mm x 300 mm	1.5 kN/m
	200 mm x 380 mm	2.1 kN/m
	200 mm x 450 mm	2.6 kN/m
	200 mm x 600 mm	3.8 kN/m

(c) Brick walls

External wall 200 mm (225 mm with plaster) thick	6.75 $kN/m/m$
Internal wall 100 mm (125 mm with plaster) thick	3.75 $kN/m/m$

(a) Computations of End Reaction of Roof Beams

The loads transferred by one-way slab and two-way roof/floor slabs *S1* and *S5* have been calculated as under :

Type I :Refers to - Load transferred from one-way slab.

Type II: Refers to - Load transferred from two-way slab

Trapezoidal portion of floor load transferred from two-way slab :

- Trapezoidal portion of floor load transferred from slab *S1*

$$= \left[\frac{3.4 + (3.4 - 3.15)}{2} \times \frac{3.15}{2}\right] \times 9 \quad = 26\ kN \qquad \ldots(2a)$$

- Trapezoidal portion of floor load transferred from slab *S5*

$$= \left[\frac{4.7 + (4.7 - 3.2)}{2} \times \frac{3.2}{2}\right] \times 9 \quad = 45\ kN \qquad \ldots(2b)$$

Triangular portion of floor load transferred from two-way slab :

- Triangular portion of floor load transferred from two-way slab *S1*

$$= \left(\frac{1}{2} \times 3.4 \times \frac{3.15}{2}\right) \times 9 \quad = 24\ kN \qquad \ldots(2c)$$

- Triangular portion of floor load transferred from two-way slab *S5*

$$= \left(\frac{1}{2} \times 3.2 \times \frac{3.2}{2}\right) \times 9 \quad = 23\ kN \qquad \ldots(2d)$$

Computation of End Reaction of Floor Beams :

(1) The calculations the weight of full height of wall and centre to centre distance between the column

is taken. The self weight of beam *or* self weight of column is not worked out and the difference between the unit weight of concrete and that of masonry is neglected.

(2) All beams are assumed to be simply supported at their ends, neglecting the continuity effect.

(3) The additional load due to slenderness or effect of fixity is not considered.

All these approximations have been made to simplify the computations.

The details as per these assumptions have been worked out in the following table :

(a) Loads transferred from Roof slab :

Beam	Span m	Loads Transferred from Slab						Load Calculations	Loads Transferred to	
		Left	Type	Right	Type	Wall kN/m	Self wt kN/m	Values in kN (Rounded)	Left Col.No/kN (Direction) 1,2,3 or 4	Right Col.No/kN
B1	3.4	-	-	S1	2a	3	1.5	26 + 3 x 3.4 + 1.5 x 3.4=42	C27/21(2)	C28/21(4)
B3	3.4	S1	2a	S4	1	-	1.5	26 + 9x (3.4 x 2.5/2) + 1.5 x 3.4 = 70	C21/35(2)	C22/35(4)
B6	3.4	S4	1	S4	1	-	1.5	9x(3.4x2.5/2)+9 x (3.4x2.6/2)+1.5x3.4 = 38.25 + 39.78 +5.1 = 84	C19/42(2)	B13/42
B7	3.4	S4	1	S4	1	-	1.5	9 x 2 x (3.4 x 2.6/2) + 1.5 x 3.4 = 84	C13/42(2)	C14/42(4)
B2a	3.2	-	-	S2	1	3	1.5	10 x 3.2 x 1.68/2 + 5.1 x 3.2 + 1.5 x 3.2 = 48	B14/24	B15a/24
B4	3.2	S2	1	S5	2d	-	1.5	9 x 3.2 x 1.68/2 + 23+ 1.5 x 3.2 = 52	C22/26(2)	C23/26(4) C7/25(4)
B8	3.2	S5	2d	S5	2d	-	1.5	23 + 23 + 1.5 x 3.2 = 51	C14/26(2)	C15/26(4)
B20	2.4	-	-	-	-	Grill	1.5	3.0 x 2.4 + 1.5 x 2.4 = 11	B15a/6	
B19	2.4	Canti.	-	Stair	-	-	1.5	15.4 x 1.26 x 2.4 + 15.4 x 2.9/2 x 2.4 +1.5 x 2.4 = 104	C23/52(2)	-
B18	2.4	Stair	-	-	-	6.75	1.5	15.4 x 2.9/2 x 2.4 + 6.75 x 2 x 2.4 + 1.5 x 2.4 = 90	B15/45	
B17	2.4	-	-	-	-	3	1.5	3 x 2.4 + 1.5 x 2.4 = 11	B16/6	
B9	2.6	-	-	-	-	3	1.5	3 x 2.6 + 1.5 x 2.6 = 12	C19/6(1)	C13/6(3)
B10	2.5	-	-	-	-	3	1.5	3 x 2.5 + 1.5 x 2.5 = 12	C21/6(1)	C19/6(3)
B11	3.15	-	-	S1	2c	3	1.5	24 + 3 x 3.15 + 1.5 x 3.15 = 38	C27/19(1)	C21/19(3)
B13	4.7	B6	42	S5	2b	-	2.1	42 + 45 + 2.1 x 4.7 = 97	C22/49(1)	C14/49(1)
B14	3.55	S1	2c	B2a	24	3	2.1	24 + 24 + 3x 2 .08 + 2.1 x 3.55 = 62	C28/31(1)	C22/31(3)
B2	3.2	S2	1	-	-	3	1.5	10 x 1.68/2 x 3.2 + 5.1 x 3.2 + 1.5 x 3.2 = 48	B16a/24	-
B15a	1.47	B2a +B20	24 +6	S4	1	6.75	3.8	30 + 9 x 2.4/2 x 1.47 + 6.75 x 2 x1.47 + 3.8 x 1.47 = 72	-	C23/72(3)
B15	4.7	S5	2b	S4 B18	1 45	6.75	3.8	45 + 9 x 2.4/2 x 2.7 + 45 + 6.75 x 2 x 2.7 + 3.8 x 4.7 = 174	C23/87(1)	C15/87(3)
B16a	1.47	B2	24	-	-	3	3.8	24 + 3 x 1.47 + 3.8 x 1.47 = 34	-	C7/34(1)
B16	4.7	S5	2b	B17 S4	6 1	3	3.8	45 + 6 + 9 x 2.4/2 x 2.1+ 3 x 2.6 + 3.8 x 4.7=100	C15/50(1)	C7/50(3)

(b) Loads transferred from Floor slab

Beam	Span m	Loads Transferred from Slab Left	Type	Right	Type	Wall kN/m	Self wt kN/m	Load Calculations Values in kN (Rounded)	Loads Transferred to Left Col.No/kN (Direction) 1,2,3 or 4	Right Col.No/kN
B1	3.4	-	-	S1	2a	6.75	2.1	26+6.75x2.7x3.4+2.1x3.4=95	C27/48(2)	C28/48(4)
B5	2.32	S3	1	S3	1	3.75	3.5	18x(1.15+1.35)/2x2.32+3.75x2.1x2.32 +3.5x2.32=78	B10/39	B12/39
B12	2.5	B5	39	S3	1	3.75	2.1	39+9x2.5x1.08/2+3.75x2.7x2.5 +2.1x2.5=82	B3/41	B6/41
B3	3.4	S1	2a	S3 B12	1 41	3.75	2.1	26+18x1.15/2x2.32+3.75x2.7x3.4 +2.1x3.4+41=133	C21/66(2)	C22/66(4)
B6	3.4	S3 B12	1 41	S4	1	3.75	2.1	18x2.32x1.35/2+41+9x3.4x2.6/2 +3.75x2.7x3.4+2.1x3.4=150	C19/75(2)	B13/75
B7	3.4	S4	1	S4	1	3.75	2.1	9x3.4x2.6/2+9x3.4x2.6/2 +3.75x2.7x3.4+2.1x3.4=122	C13/61(2)	C14/61(4)
B2a	3.2	-	-	S2	1	3	2.1	10x3.2x1.68+3x1.7x3.2 +2.1x3.2=77	B14/38	B15a/38
B4	3.2	S2	1	S5	2d	6.75	2.1	9x3.2x1.68/2+23+6.75x2.7x3.2 +2.1x3.2=112	C22/56(2)	C23/56(4) C7/56(4)
B8	3.2	S5	2d	S5	2d	3.75	2.1	23+23+3.75x2.7x3.2+2.1x3.2=86	C14/43(2)	C15/43(4)
B20	2.4	-	-	-	-	Grill	2.1	5.1x2.4+2.1x2.4=18	B15a/9	
B19	2.4	Canti.	-	Stair	-	-	2.1	15.4x1.26x2.4+15.4x2.4x2.9/2 +2.1x2.4=106	C23/53(2)	-
B18	2.4	Stair	-	-	-	6.75	2.1	15.4x2.9/2x2.4+6.75x2x2.4 +2.1x2.4=92	B15/46	-
B17	2.4	-	-	-	-	3	2.1	3x2.4+2.1x2.4=12	B16/6	-
B9	2.6	-	-	-	-	6.75	2.1	6.75x2.7x2.6+2.1x2.6=53	C19/27(1)	C13/27(3)
B10	2.5	B5	39	-	-	6.75	2.1	39+6.75x2.7x2.5+2.1x2.5 =90	C21/45(1)	C19/45(3)
B11	3.15	-	-	S1	2c	6.75	2.1	24+6.75x2.7x3.15+2.1x3.15=88	C27/44(1)	C21/44(3)
B13	4.7	B6	75	S5	2b	3.75	2.1	75+45+3.75x2.7x4.7+2.1x4.7 9x1.08/2x2.2=189	C22/95(1)	C14/95(3) C14/95(1)
B14	3.55	S1 +B20	2c +6	B2a	38	6.75	2.1	24+38+6.75x2.7x3.55+2.1x3.55=134	C28/67(1)	C22/67(3)
B15a	1.47	B2a +B20	38 9	-	-	6.75	3.8	47+6.75x2.7x1.47+3.8x1.47=79	-	C23/79(3)
B2	3.2	S2	1	-	-	3	2.1	10x3.2x1.68/2+3x1.7x3.2 +2.1x3.2 = 50	B16a/25	-
B16a	1.47	B2	25	-	-	6.75	3.8	25+6.75x2.7x1.47+3.8x1.47=58	-	C7/58(1)
B16	4.7	S5	2b	B17	6	6.75	3.8	45+6+6.75x2.7x4.7+3.8x4.7=155	C15/78(1)	C7/78(3)
B15	4.7	S5	2b	B18	46	6.75	3.8	45+46+6.75x2.7x4.7+3.8x4.7=195	C23/98(1)	C15/98(3)

(c) Computation of End Reaction of Plinth Beam

The plinth will only carry wall loads and its self weight. Beams *B2, B5, B12, B17, B18* and *B20* have not been provided at plinth level. Beams *B20* and *B21* of size 230 *mm* x 300 *mm* have been provided to acts as stiffener for the column. Since they carry only self weight their end reaction being small is neglected.

All plinth beams have been provided at ground level to get lateral rigidity for the whole frame.

The beams supporting walls has width of 200 *mm* (225*mm* with plaster) and depth 380 *mm*.

The self weight of the beam = (1.5 x 0.23 x 0.38)x 25 = 3.2 *kN/m*

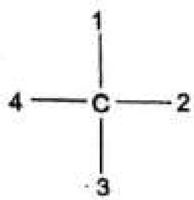
Load due to the external wall= 6.8 x (3.45–0.38) = 21 *kN/m* *(Table 9.3.2)*

Load due to internal wall = 3.8 x (3.45 – 0.38) = 11.7 *kN/m* *(Table 9.3.2)*

(c) The end reaction of plinth beams are worked out as under :

Beam No.	Span m	Loads Transferred from		Load Calculations	Loads Transferred to	
		Wall kN/m	Self wt. kN/m		Left Col.No/kN (Direction)	Right Col.No./kN (Direction)
B1	3.4	21	3.2	21 x 3.4 + 3.2 x 3.4 = 82	C27/41(3)	C28/41(4)
B3	3.4	11.7	3.2	11.7 x 3.4 + 3.2 x 3.4 = 51	C21/26(2)	C22/26(4)
B6	3.4	11.7	3.2	11.7 x 3.4 + 3.2 x 3.4 = 51	C19/26(3)	B13/26
B7	3.4	11.7	3.2	11.7 x 3.4 + 3.2 x 3.4 = 51	C13/26(2)	C14/26(4)
B4	3.2	21	3.2	21 x 3.2 + 3.2 x 3.2 = 78	C22/39(2)	C23/39(4) C7/39(4)
B8	3.2	11.7	3.2	11.7 x 3.2 + 3.2 x 3.2 = 48	C14/24(2)	C15/24(4)
B9	2.6	21	3.2	21 x 2.6 +3.2 x 2.6 = 63	C19/32(1)	C13/32(3) C13/32(1)
B10	2.5	21	3.2	21 x 2.5 + 3.2 x 2.5 = 61	C21/31(1)	C19/31(3)
B11	3.15	21	3.2	21 x 3.15 +3.2 x 3.15 = 76	C27/38(1)	C21/38(3)
B13	4.7	11.7 B13/26	3.2	11.7 x 4.7 +3.2 x 4.7 +26 = 96	C22/48(1)	C14/48(3) C14/48(1)
B14	3.55	21	3.2	21 x 3.55 +3.2 x 3.55 = 86	C28/43(1)	C22/43(3)
B15a	1.47	21	3.2	21 x 1.47 + 3.2 x 1.47 = 36	C23/36(3)	-
B15	4.7	21	3.2	21 x 4.7 +3.2 x 4.7 = 113	C23/57(1)	C15/57(3)
B16a	1.47	21	3.2	21 x 1.47 + 3.2 x 1.47 = 36	C7/36(1)	-
B16	4.7	21	3.2	21 x 4.7 + 3.2 x 4.7 = 113	C15/57(1)	C7/57(3)

Computation of Load on Column (Approximate Method)



The load on any column is computed from the end reactions of the beams on the same lines as detailed in *Sect. 8.6.2*. The beam directions are taken as per adjacent Figure.

The load computations for each column are given as under.

Columns in Top Storey

Loads Transferred by Roof Beams (Approximate method)

Category	Column Mark	Beam End Forces in kN from direction 1	2	3	4	Total kN	Tank Load kN	Total Rounded Pr kN
I	C14	49	26	49	42	166	-	166
II	C7	34	-	50	25	109	-	109
	C13	6	42	6	-	54	-	54
	C15	50	-	87	26	163	105	268
	C19	6	42	6	-	54	-	54
	C21	6	35	19	-	60	-	60
	C22	49	26	31	35	141	-	141
	C23	87	52	72	26	237	105	342
III	C27	19	21	-	-	40	-	40
	C28	31	-	-	21	52	-	52

Columns in Intermediate Storey

Loads Transferred by Floor Beams (Approximate method)

Category	Column Mark	Beam End forces in kN from direction 1	2	3	4	Total P_f kN
I	C14	95	43	95	61	294
II	C7	58	-	78	56	192
	C13	27	61	27	-	115
	C15	78	-	98	43	219
	C19	27	75	45	-	147
	C21	45	66	44	-	155
	C22	95	56	67	66	284
	C23	98	53	79	56	286
III	C27	44	48	-	-	92
	C28	67	-	-	48	115

Columns in First Storey

Loads Transferred by Plinth Beams (Approximate method)

Category	Column Mark	Beam End forces in kN from direction 1	2	3	4	Total P_p kN
I	C14	48	24	48	26	146
II	C7	36	-	57	39	132
	C13	32	26	32	-	90
	C15	57	-	57	24	138
	C19	32	26	31	-	89
	C21	31	26	38	-	95
	C22	48	39	43	26	156
	C23	57	-	36	39	132
III	C27	38	41	-	-	79
	C28	43	-	-	41	84

Calculation of column loads in each storey (Approximate Method)

Loads obtained in each storey are compiled and loads at each storey level are worked out.

(A) Loads in each storey in *kN* (Approximate method)

Storey	Category									
	I	*II*							*III*	
	C14	C7	C13	C15	C19	C21	C22	C23	C27	C28
Top Storey P_r *kN*	166	109	54	268	54	60	141	342	40	52
Int. Storey P_f *kN*	294	192	115	219	147	155	284	286	92	115
1st Storey P_p *kN*	146	132	90	138	89	95	156	132	79	84

(B) Loads in each storey in *kN* (Approximate method)

Storey	Category									
	I				*II*				*III*	
	C14	C7	C13	C15	C19	C21	C22	C23	C27	C28
Top Storey P_r *kN*	166	109	54	268	54	60	141	342	40	52
3rd Storey $P_r + P_f$ *kN*	460	301	169	487	201	215	425	628	132	167
2nd Storey $P_r + 2P_f$ *kN*	754	493	284	706	348	370	709	914	224	282
1st Storey $P_r + 3P_f$ *kN*	1048	685	399	925	495	525	993	1200	316	397
Plinth $P_r + 3P_f + P_p$ *kN*	1194	817	489	1063	584	620	1149	1332	395	481
Add 10%	119	82	49	106	58	62	115	133	40	48
Total Load (Apprr. Method)	***1313***	***899***	***538***	***1169***	***642***	***682***	***1264***	***1465***	***435***	***529***
Total Load (Exact Method)	***1325***	***800***	***490***	***1182***	***562***	***627***	***1316***	***1314***	***385***	***360***

Remarks : *Since approximate method does not take into account allowance for fixity, continuity effect, and slenderness, they have been increased arbitrarily by 10%.Depnding on the quality control expected, availability of skilled labour, material available the percentage may be increased. A comparison with load obtained by exact method will show that the loads arrived at by approximate method are in general on higher side and or in same cases they are less than the exact load still the differences between them is not more than 5%. Hence loads obtained by approximate method can be used for design of footing.*

Storey	*Column*	*Section*	*Main Steel*	*Lateral ties*
R-3	C14	200 x 230	4#12	ϕ 6 at 190
	C7	200 x 230	4#12	ϕ 6 at 190
	C15	200 x 230	4#12	ϕ 6 at 190
	C13,C19,C21	200 x 230	4#12	ϕ 6 at 190
	C22	200 x 600	4#16+2-#12	ϕ 6 at 190
	C23	200 x 600	4#16+2-#12	ϕ 6 at 190
	C27 and C28	200 x 230	4#12	ϕ 6 at 190

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Storey	Column	Section	Main Steel	Lateral ties
3-2	C14	200 x 300	6-#12	ϕ 6 at 190
	C7	200 x 230	4#12	ϕ 6 at 190
	C15	200 x 300	4#16+2#12	ϕ 6 at 190
	C13,C19,C21	200 x 230	4#12	ϕ 6 at 190
	C22	200 x 600	4#16+2-#12	ϕ 6 at 190
	C23	200 x 600	4#16+2-#12	ϕ 6 at 190
	C27 and C28	200 x 230	4#12	ϕ 6 at 190
2-1	C14	230 x 380	4-#16+2-#12	ϕ 6 at 190
	C7	230 x 450	4#16	ϕ 6 at 190
	C15	230 x 380	4#16+2#12	ϕ 6 at 190
	C13,C19,C21	200 x 300	6#12	ϕ 6 at 190
	C22	200 x 600	4#16+4-#12	ϕ 6 at 190
	C23	200 x 600	4#16+2-#12	ϕ 6 at 190
	C27 and C28	200 x 230	4#12	ϕ 6 at 190
1-PL	C14	230 x 530	6-#16+2-#12	ϕ 6 at 190
	C7	230 x 530	4#16+2-#12	ϕ 6 at 190
	C15	230 x 450	6#16	ϕ 6 at 190
	C13,C19,C21	230 x 300	6#12	ϕ 6 at 190
	C22	230 x 600	4#16+4#12	ϕ 6 at 190
	C23	230 x 600	4#16+4-#12	ϕ 6 at 190
	C27 and C28	230 x 230	4#12	ϕ 6 at 190
PL-Ft	C14	230 x 600	6-#16+2-#12	ϕ 6 at 190
	C7	230 x 600	6#16	ϕ 6 at 190
	C15	230 x 600	6#16+2-#12	ϕ 6 at 190
	C13,C19,C21	230 x 300	6#12	ϕ 6 at 190
	C22	230 x 600	6#16+2-#12	ϕ 6 at 190
	C23	230 x 600	6#16+2-#12	ϕ 6 at 190
	C27 and C28	230 x 230	4#12	ϕ 6 at 190

9.7 DESIGN OF FOOTINGS

9.7.1 Categorisation of Footings

All footings have been designed for axial loads as the bearing capacity of the soil is reasonably low.

9.7.2 Grouping of Footings

Footings of the columns having same sizes and variation of load of about 10% are grouped together and designed for the maximum load in that group.

Group	Column Nos.	Size *mm x mm*	Maximum ultimate Load *kN*	Design working load Rounded *kN*
I	C27, C28	230 x 230	385	260
II	C13, C19, C21	230 x 300	627	420
III	C7	230 x 600	800	540
IV	C14, C15, C22, C23	230 x 600	1325	890

The design of footing has been made using the software developed by the Author.[9.3]

9.7.3 Design of Footings

Step	Design Calculations		C27,C28	C13,C19,C21	C7	C14,C15,C22,C23
I	**Data :**					
	Maximum column load P_u		385	627	800	1325
	Design working load $= P = P_u / 1.5$		260	420	540	890
	Column Section	mm x mm	230 x 230	230 x 300	230 x 600	230 x 600
	Bearing capacity of soil	kN/m^2	300	300	300	300
	Material used	mm	M20, Fe415	M20, Fe415	M20,Fe415	M20, Fe415
	Offset at top of footing	mm	75	75	75	75
II	***Proportioning of Base size***					
	Area of footing required	m^2	0.95	1.54	1.98	3.26
	Area of footing provided	m^2	0.96	1.55	1.99	3.29
	Length of footing provided	mm^2	980	1280	1610	2010
	Breadth of footing provided	mm^2	980	1210	1240	1640
	Projection from column face	mm^2	375	490	505	705
	$w_u = P \times 1.5$/Area of footing	kN/m^2	406.1	406.7	405.7	404.98
III	***Depth of footing required from B.M. Considerations :***					
	M_{ux}	kN.m	27.98	59.1	64.1	165.05
	M_{uy}	kN.m	27.98	62.5	83.3	202.29
	Depth for B.M	mm	220	300	310	460
IV	***Depth of footing required from Two - way shear consideration :***					
	Perimeter at critical Sect B_2	mm	1688	2012	2720	3320
	Depth at peripheral Sect D_2	mm	184.3	222.1	240.9	345.6
	Area resisting shear A_2	mm^2	311098	446858	655330	1147376
	Shear resisted by concrete V_{uc2}	kN	347.8	499	647.2	1133.1
	Design shear V_{uD2}	kN	317.7	527	636.3	1069.8
	Depth for 2-way shear $(D_f)_{req}$	mm	260	310	330	480
V	***Depth increased to satisfy One-way shear requirements:***	$(D_f)_{prov.}$	300	400	400	550
			150	150	150	200
	Revised calculations for two -way shear					
		B_2 mm	1848	2412	3000	3600
		D_2 mm	211.5	281.4	281.2	391.9
		A_2 mm^2	390852	678673	843662	1411000
		V_{uc2} kN	437	758.8	833.2	1393.5
		V_{uD2} kN	303	482.6	595.6	1020.8
	Area of steel :					
	Area of steel provided along long direction		402	553	628	1099
	Area of steel provided along short direction		452	603	942	1335
VI	***Check for One-way Shear for Bending about y-axis***					
		A_y mm^2	142729	215927	279176	490128
		p_{ty}	0.28	0.28	0.337	0.272
		τ_{ucy} N/mm^2	0.39	0.376	0.408	0.372
		V_{ucy} kN	56.7	81.3	113.8	182.6
		V_{uDy} kN	55.3	79.1	111.0	179.1

Step	Design Calculations		C27,C28	C13,C19,C21	C7	C14,C15,C22,C23
VII	***Check for One-way shear for Bending about x-axis***					
		A_x mm^2	147729	208632	218265	404633
		p_{tx}	0.27	0.265	0.288	0.27
		τ_{ucx} N/mm^2	0.37	0.368	0.381	0.372
		V_{ucx} kN	55.0	76.82	83.2	150.6
		V_{uDx} kN	52.1	70.90	80.5	139.5
VIII	***Results***					
	Length of Footing	L_f mm	980	1280	1610	2010
	Breadth of Footing	B_f mm	980	1210	1240	1640
	Total depth of footing	D_f mm	300	400	400	550
	Minimum Depth of footing	$D_{f.min}$ mm	150	150	150	200
	Offset at top of footing	mm	75	75	75	75
	No.Dia of bars along long direction	$N_x - \#$	8-#8	11-#8	8-#10	14-#10
	No. Dia. of bars along short direction	$N_y - \#$	9 – #8	12-#8	12-#10	17–#10
	Clear dist. of bars along long direction	C_{Lx} mm	115	101	145	105
	Clear dist. of bars along Short direction	C_{Ly} mm	101	98	126	108

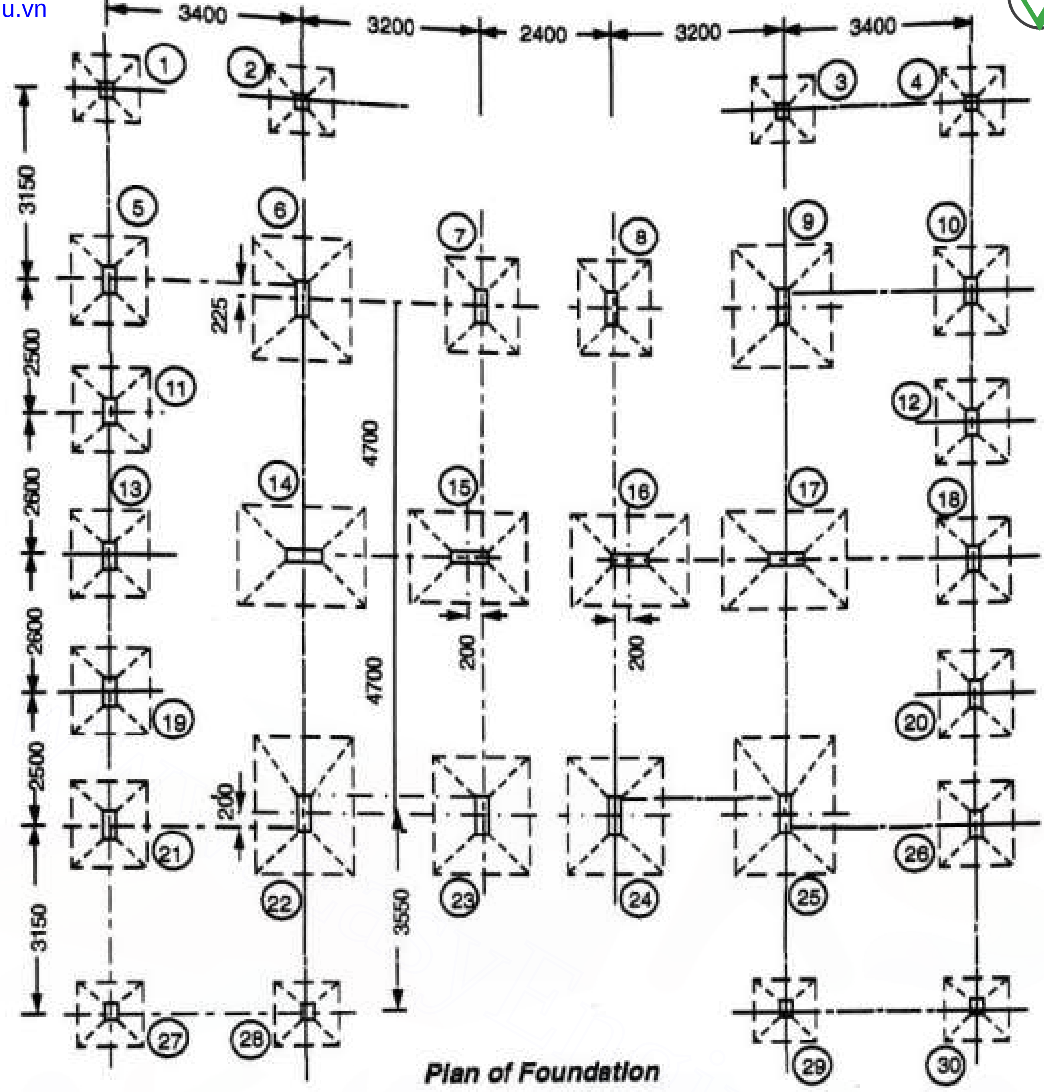
References

9.1 Software for design of axially loaded column, column subjected to uniaxial bending biaxial bending slender column. Structure Publications, Pune 411009

9.2 Shah, V. L., Karve, S. R., "Limit State theory and design of reinforced concrete ",Structure Publication, Pune 411009, Seventh Edition 2014, Sect. 11.6.1 and *Ex. 11.6.1(b)*,

9.3 Software No.3 for design of pad *or* sloped footing for Axially loaded column, Structure Publications, Pune 411009

9.4 Shah, V. L., Karve, S. R., "Handbook of Reinforce Concrete Design" Structure Publications, Pune 411009, Fifth Edition 2010.



Plan of Foundation

COLUMN N
PCC M10
RCC Footing
Column Between: Footing to Plinth
Plinth to 1st Floor
1st Floor to 2nd Floo
2nd Floor to 3rd Flo
3rd Floor to Roof

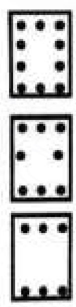
Notes :

1) Diameter spac

2) *N1 - D1 + N2*

N1- D1 Repre

N2 - D2 Repre



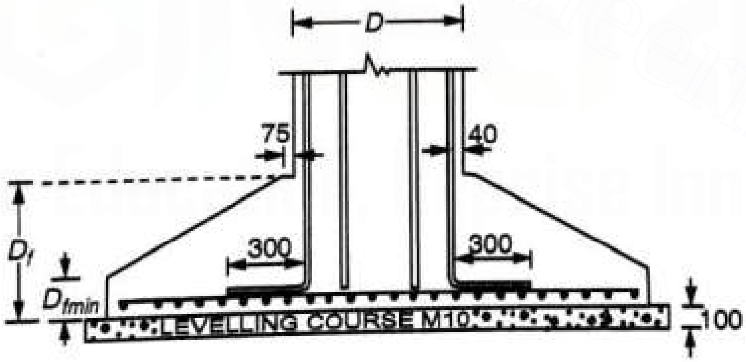
e.g. 6#16 + 4

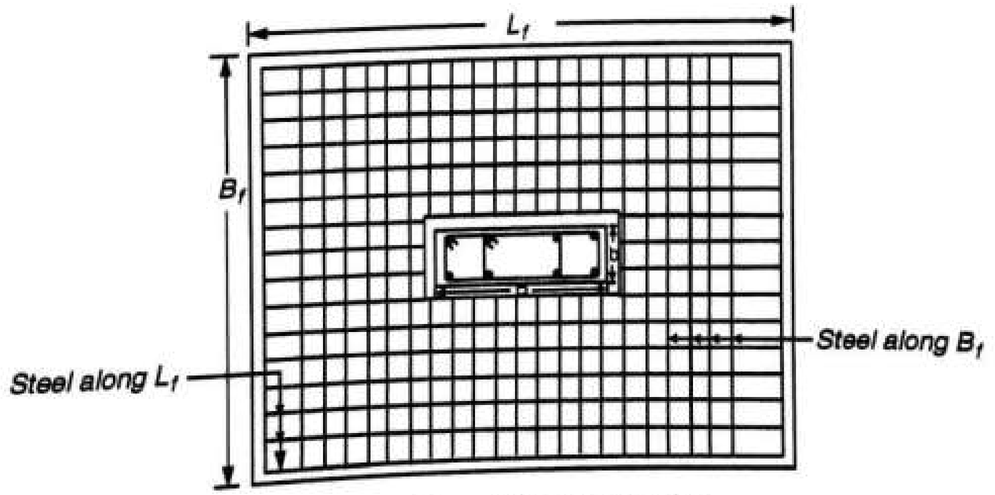
three No. of 1

third row equi

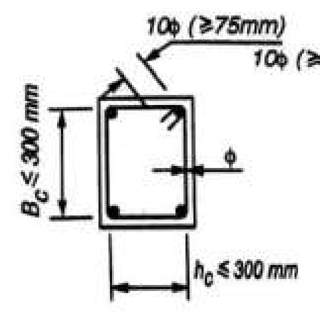
If *N2* = 2 then

If *N2* = 0 then



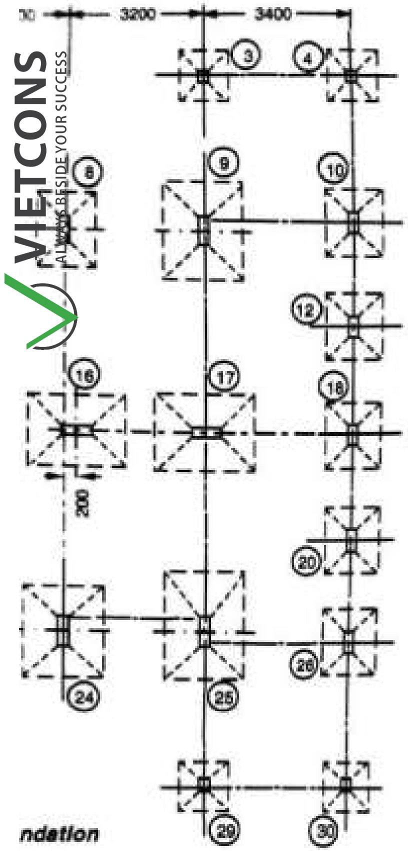


General Footing Details



h shall be larger of h_c and B_c

(a) Single hoop



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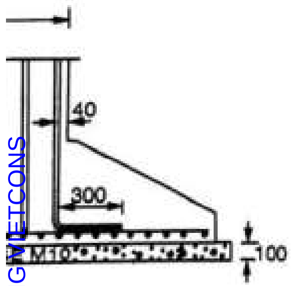
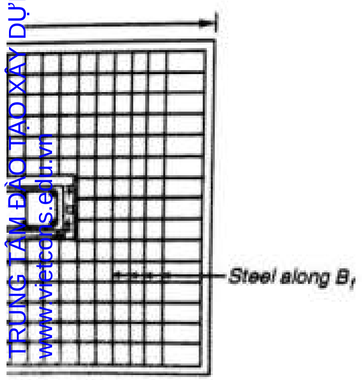


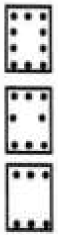


SCHEDULE OF COLUMNS

COLUMN NUMBERS		1,2,3,4,27, 28, 29, 30	5,10,11,12,13 18,19,20,21,26	6,9,22,25	23,24	7,8	14,17	15,16
PCC M10	Thickness 100 *mm*	1280 x 1280	1580 x 1510	2310 x 1940	2310 x 1940	1910 x 1540	2310 x 1940	2310 x 1940
RCC Footing	Size : $L_f \times B_f$	980 x 980	1280 x 1210	2010 x 1640	2010 x 1640	1610 x 1240	2010 x 1640	2010 x 1640
	Depth : D_f, D_{fmin}	300 x 150	400 x 150	550 x 200	550 x 200	400 x 150	550 x 200	550 x 200
	Steel : Along L_f N-#	8 - #8	11-#8	14-#10	14-#10	8-#10	14#10	14#10
	Along B_f N-#	9 - #8	12-#8	17-#10	17-#10	12-#10	17#10	17#10
Column Between: Footing to Plinth	Size : b x D	230 x 230	230 x 300	230 x 600	230 x 600	230 x 600	230 x 600	230 x 600
	Steel N-#	4#12	6#12	6#16+2#12	6#16+2#12	6-#16	6#16 + 2#12	6#16 + 2#12
Plinth to 1st Floor	Size : b x D	230 x 230	230 x 300	230 x 600	230 x 600	230 x 530	230 x 530	230 x 450
	Steel : N-#	4#12	6#12	4#16 + 4#12	4#16+4#12	4#16+2#12	6#16 + 2#12	6#16
1st Floor to 2nd Floor	Size : b x D	200 x 230	200 x 300	200 x 600	200 x 600	230 x 450	230 x 380	230 x 380
	Steel : N-#	4#12	6#12	4#16 +4#12	4#16+2#12	4#16	4#16 + 2#12	4#16 + 2#12
2nd Floor to 3rd Floor	Size : b x D	200 x 230	200 x 230	200 x 600	200 x 600	200 x 230	200 x 300	200 x 300
	Steel : N-#	4#12	4#12	4#16+2#12	4#16+2#12	4#12	6#12	4#16 + 2#12
3rd Floor to Roof	Size : b x D	200 x 230	200 x 230	200 x 600	200 x 600	200 x 230	200 x 230	200 x 230
	Steel : N-#	4#12	4#12	4#16+2#12	4#16 + 2#12	4#12	4#12	4#12

Notes :

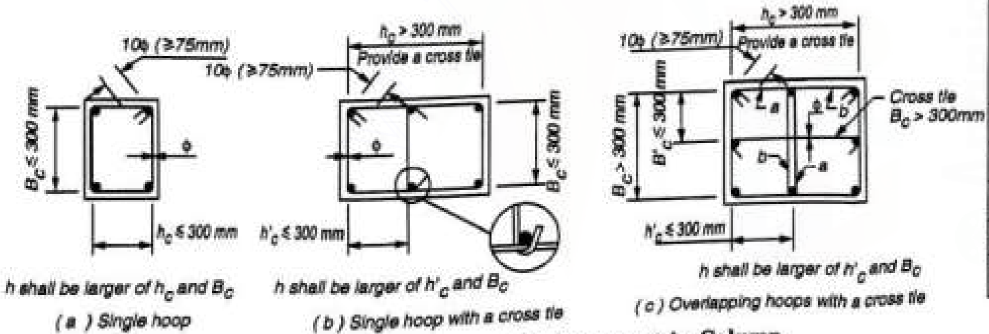
1) Diameter spacing of lateral ties shall be φ6 *mm* at 190 *mm* c/c
2) *N1 - D1 + N2 - D2* This nomenclature has the following meaning :
   N1- D1 Represent total no. of bars in first and last row
   N2 - D2 Represent total no. of bars between first and last row
   e.g. 6#16 + 4 #12 means three No of 16 *mm* diameter bars in first row and three No. of 16 *mm* in last row, and two No of 12 *mm* in second row and third row equispaced. Thus, toal No of rows = 4



If *N2* = 2 then total No of rows = 3

If *N2* = 0 then toal No of rows = 2

Notes : 1 Grade of concrete M20
Grade of Steel Fe415 denoted by #
Grade of Steel Fe250 denoted by φ
2. Environmental Exposure Condition : Mild
Nominal Cover for Column 40*mm*
3. Fire rating considered is for 1 hour maximum.
4. Do not scale the drawing.



Details of Transverse Reinforcement in Column

	STRUCTURES PUBLICATIONS		
	Four Storeyed Residential Building		
	Structural Details of Column		
PROJECT NO.	3	STRUCTURAL ENGR.	
PLOT NO.		DRG. NO.	
S. NO.		DRG. BY	
DATE :		SIGN	

CHAPTER-10

DESIGN OF PORTAL FRAME

10.1 INTRODUCTION

A portal frame is a frame having its elements rigidly connected at the joints. The rigid connections give geometrical stability to the frame. Such rigid jointed reinforced concrete frames are often used in the construction of assembly halls, workshop buildings, industrial structures, bridges and viaducts. According to the number of storeys in a building, frames are single storey *or* multi-storey and as regards to the number of spans they are single span and multi-span.

Some of the forms of portal frames in R.C. construction are shown in *Fig.10.1.1*

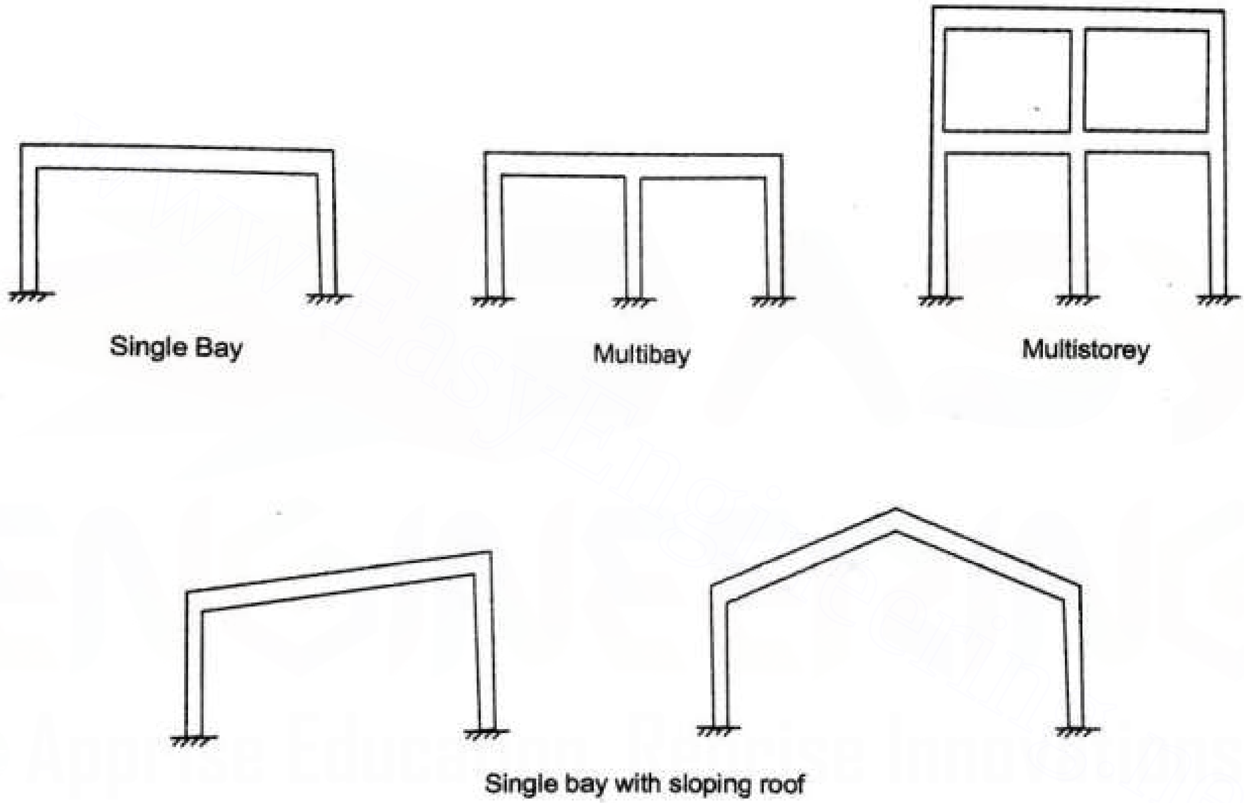


Fig. 10.1.1 Different types of R.C. Portal Frames

For warehouses and workshops the sloping roof comprising of purlins and asbestos sheet roofing between portal frames is provided. While for buildings the portal frame with R.C. slab roof cast monolithically is used. The R.C. slab may be provided above the beam *or* below the beam of the portal depending on the choice *or* requirement of the user. See *Fig. 10.2.1*

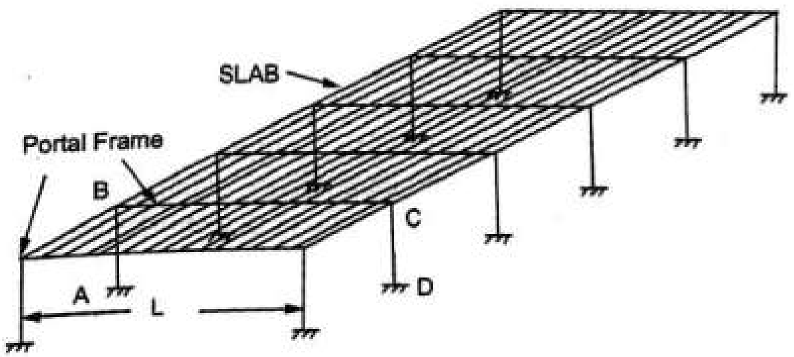


Fig. 10.1.2 Portal Frame with R.C. Slab Roof

10.2 ANALYSIS AND DESIGN OF PORTAL FRAMES

10.2.1 Introduction

Single-storey portal frames with flat roof have span of 12 to 15 meters and spaced at interval of 3*m* to 4 *m*. The columns of the frame are generally supported on isolated footings in which case the end condition may be taken as hinged *or* fixed. If the footing is resting on soil of very low bearing capacity, it can be assumed as hinged because the soil being compressible the foundation undergoes rotation relieving off the moment. However, if the isolated footing is to be provided on hard soil it can be assumed as fixed since the hard soil will not deform to allow the rotation of the foundation (see *Sect. 3.2.4*). In case of columns supported on pile foundation *or* raft the base of the column should be assumed as fixed.

10.2.2 Choice of preliminary Cross-Sectional Shape, and Dimension

In case of portal frames the relative dimensions of different structural components influence the analysis. Since the connections between the column and beam at the joint are monolithic the distribution of moment will depend on the relative stiffnesses of the beam and column (see *Sect.3.2.6.*) The stiffness of the member is a function of size of the section, length of the member and end conditions. If the stiffness of the column is much greater than the stiffness of the beam, the column will have high stability against lateral forces but the column will be subjected to large moments which may become costly. The section of the column should be assumed such that the desired moments are developed in the component members. Normally a solid rectangular section for column is chosen mainly from consideration of ease of construction and partly from the consideration of stiffness against lateral loads.

The depth of the beam may be assumed taking the ratio of span/depth between 12 to 16. The lower value of 12 to be adopted for heavy loads and larger value of 16 for lighter loads. Alternatively, the depth of the beam may be determined for a support moment equal to 2/3 times simply supported bending moment for superimposed load. The width of the beam may be taken between 0.3 to 0.5 of the depth of beam. If the slab is cast monolithically over the beam, the beam acts as a flanged section in the mid-span region and a rectangular section at the support where negative moments prevail. Thus, the beam has a varying moment of inertia along its length, which is very difficult to determine. Therefore, the moment of inertia may be calculated assuming a rectangular section *or* a flanged section *or* approximately twice that of rectangular section (for details see *Sect 3.3.3.*). The assumption of rectangular section of beam instead of a flanged section gives higher moments in the column and vice versa. The width of the column may be taken equal to *or* greater than that of beam but not less.

Normally the support section is required to be designed as a doubly reinforced section. The requirement of compression steel at the bottom of the section is automatically met with due to continuation of minimum positive reinforcement at bottom *(See Sect.5.3.3)*, So it is advantages to design the support section as doubly reinforced .

10.2.3 Methods of Analysis

The portal frames can be analyzed by various elastic methods *viz.* slope deflection method, moment distribution method, column analogy method, strain-energy method *or* matrix method. The stiffness method is more adaptable using a plane frame computer program while moment distribution method is generally used for hand computation. If the top beam of a portal is inclined with a slope not exceeding 1 in 8 it can be replaced by horizontal one to simplify the computations with practically no influence on the results.

The usual example of a rigid frame is a symmetrical portal frame shown in *Fig. 10.1.2*. The columns *AB* and *CD* have the same cross section and length. If the loading on the beam is symmetrical (say loaded by UDL.) there will be no movement of the frame *or* in other words it will be a non-sway frame. The bending moment and slope at ends of beam will be equal in magnitude but opposite in sign. Such frames can be analyzed by short cut method of moment distribution using modified stiffness factor for beam equal to $2EI/L$ instead of $4EI/L$ and distribution carried out for half the frame [10.1].The example given in *Sect 10.3.1* illustrates the method of analysis and design. In *Sect.10.3.2* the same example is worked out allowing redistribution of moments to understand the economics and advantages of redistribution of moments.

10.3 DESIGN OF FIXED BASE PORTAL FRAME *(Without Redistribution of Moments)*

The roof of a work shop 20*m* long and 12*m* wide between centres of columns is supported by a fixed base R.C.portal frame spaced at 4*m* apart. The height of the column from top of footing up to the centre of beam is 6 *m*. The columns are laterally braced at a height of 2*m* above plinth level. Design the roof slab and an intermediate portal frame for the following additional data :

Live load	*LL*	=	0.75 kN/m^2
Floor finish	*FF*	=	2.25 kN/m^2
Depth of foundation		=	1.4 *m* below G.L.
width of the beam		=	230*mm*
Soil bearing capacity SBC		=	400 kN/m^2
Materials		:	Concrete M20, Steel Fe415
Exposure condition		:	Mild Environment
Design assumptions		:	All members of the frame are rigid jointed.

10.3.1. Design of Portal without Redistribution of Moments

DATA : For mild environment *(See Table C1)* minimum grade of concrete is M20 and nominal cover is 20*mm*.

Effective Span of portal frame		= 12*m*
Spacing of portal frame		= 4*m*
Height of Columns above footing		= 6*m*
Live load on roof	*LL*	= 0.75 kN/m^2
Floor finish	*FF*	= 2.25 kN/m^2
Height of plinth		= 0.4 *m* above G.L.
Depth of foundation		= 1.4 *m* below G.L
Width of beam	*b*	= 230 *mm*
Soil bearing capacity	*SBC*	= 400 kN/m^2
Concrete grade	f_{ck}	= 20 N/mm^2
Main steel grade	f_y	= 415 N/mm^2

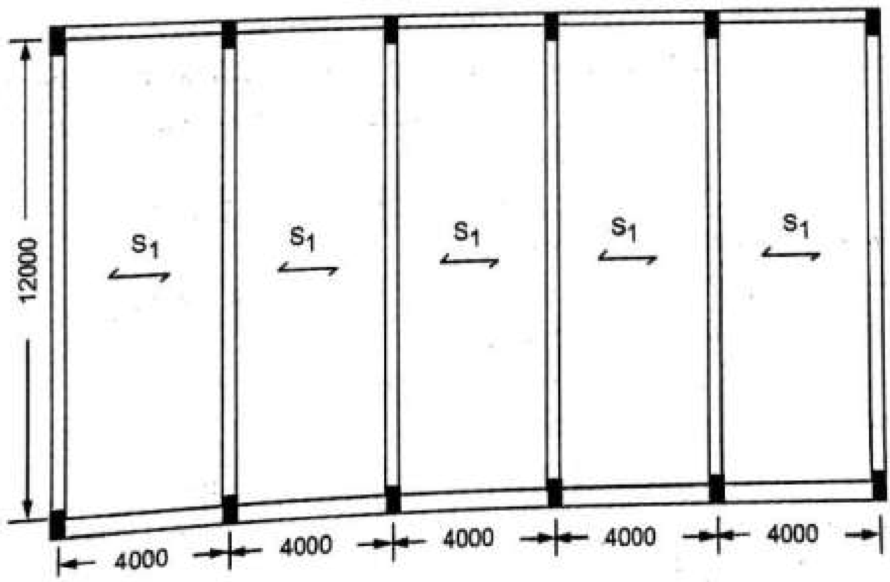


Fig. 10.3.1 Structural Plan of Assembly Hall

10.3.2 Design of Slab - S1

The slab is designed as a continuous slab supported by portal frames. Thus, the span of the slab is 4*m*. Since the number of spans are more than two, the slab is designed using I.S.code coefficients. For mild environment with bar size not exceeding 12*mm*, Nominal cover = 20 – 5 = 15*mm* (*Table C-1*)

Step No.	Design Calculations

1. *Type* : One-way continuous slab with 4 equal spans.
2. *Span* : $L = 4m$ = 4000*mm*
3. *Imposed loads* : Live load, *LL* = 0.75 kN/m^2
   Floor finish, *FF* = 2.25 kN/m^2
4. *Trial Depth* : This is decided by deflection criteria.
   Basic *L/d* ratio for continuous slab = $r_b = 26$
   Assuming $p_t = 0.35\%$
   Modification factor [10.2,10.3] α_1 = 1.4 (*Fig. 4.7.1*)
   Required depth d = 4000 /(1.4x 26) say 110 *mm*.
   Assuming effective cover = 20*mm*
   Required total depth = 110 + 20 = 130 *mm*.
   Using # 8*mm* bars effective depth provided $d = 130 - 15 - 8/2 = 111mm$
5. *Loads* : Consider one meter width of slab (*i.e.* $b = 1m$)
   Self weight+ floor finish = $w_d = 25 \times 0.13 + 2.25$ = 5.50 *kN/m*
   Live load w_L = 0.75 *kN/m*
   Total working load w = 6.25 *kN/m*
6. *Design moments*
   Bending moments are calculated at different sections using I.S. code coefficients.

Section		1	2	3	4
Dead load	α_d	1/12	–1/10	1/16	–1/12
Live load	α_L	1/10	–1/9	1/12	–1/9
Bending moment kN.m		12.8	–15.2	9.75	–13.0

These ultimate moments at different sections have been calculated using the equation :

$M_u = L.F\,(\alpha_d \times w_d L^2 + \alpha \times w_L L^2)$ where, $L.F$ = load factor = 1.5

At middle of outer span: (*i.e.* at *Sect.1*)

$M_{u1} = 1.5\,(5.5 \times 4^2/12 + 0.75 \times 4^2/10)$ = 12.8 *kN.m.*

At penultimate support : (*i.e.* at *Sect -2*)

$M_{u2} = 1.5\,(5.5 \times 4^2/10 + 0.75 \times 4^2/9)$ = 15.2 *kN.m.*

At middle of inner span : (i.e. at Section -3)

$M_{u3} = 1.5\,(5.5 \times 4^2/16 + 0.75 \times 4^2/12)$ = 9.75 *kN.m.*

At intermediate support : (i.e. at Section -4)

$M_{u4} = 1.5\,(5.5 \times 4^2/12 + 0.75 \times 4^2/9)$ = 13.0 *kN.m.*

Absolute maximum B.M. = $M_{u.max}$ = 15.2 *k.N.m.*

7. *Check for concrete Depth:*
   $M_{ur.max} = R_{u.max}\; bd^2$, For slab, b = 1000 *mm.*
   $M_{ur.max} = 2.76 \times 1000 \times 110^2 \times 10^{-6}$ = 33.4 *kN.m*.> 15.2 $kN.m^2$ ∴ safe

Design of Slab - S1 Continued.....

Step No.	Design Calculations

8. *Main steel :*

This is obtained at 4 different sections using equation :

$$A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6\, M_u}{f_{ck}\, bd^2}}\right] bd$$

or

$$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times M_u}{20 \times 1000 \times 111^2}}\right] \times 1000 \times 111$$

The areas of steel worked out as per above formula are given in *step 11*

9. *Check for Deflection :*

The deflection can be maximum at mid-span (and *not at support*)

Required maximum A_{st} at mid-span = A_{stl} = 342 mm^2

Required p_t = 342 x 100 /(1000 x 111) = 0.31% < 0.35 assumed ∴ safe

However, the detailed check as per code *(clause 23.2.1c)* is worked out.

$(p_t)_{provided}$ = 100 x 357/(1000 x 111) = 0.32%

f_s = 0.58 x 415 x 342/357 = 230 N/mm^2

For f_s = 230 N/mm^2 and p_t % = 0.32%, Modification factor = 1.48 *(Fig. 4.4.1)*

Required depth = 4000/(26x1.48) = 104 *mm* < 111 *mm* ∴ safe

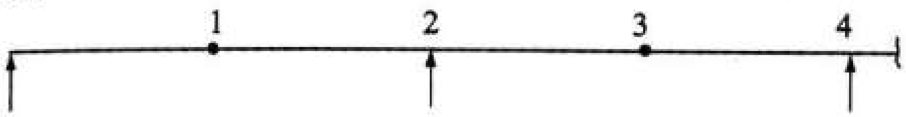
10. *Distribution Steel :*

For Fe415 grade A_{st} = 0.12% of bD = 0.12 x 1000 x 130/100 = 156 mm^2

Provide #8 *mm* at 320 *mm* c/c, Area provided = 157 mm^2

11. *Detailing of Reinforcement*:

Section



Section	1	2	3	4
Required A_{st} in mm^2	342	411	256	347
Diam and spacing	#8@140	#8@280+#8@380	#8@190	#8@380+#8@380
Provided A_{st} in mm^2	359	312	265	265
Required extra A_{st} in mm^2	-	99	-	82
Diam & spacing of extra bars	-	#8@500	-	#8@600
Provided extra steel in mm^2	-	100	-	83

Note : *(1) The requirement of maximum spacing of (3d or 300mm) whichever is less is not applicable for extra steel at support.*

(2) From practical considerations the extra steel at support may be provided at 380mm c/c instead of 500mm or 600mm so that they can be provided along the bent - up bars for ease of placing of bars and to save the labour.

10.3.3 Determination of Cross - Sectional Dimensions of Beam

Step No	Design Calculations
1.	*Span* : Beam span = 12*m* , Column height = 6 *m*.
2.	*Loads from slab* : For intermediate portal, Slab left = *S1* , w_{s1} = 0.5 $w\,L_x$ = 0.5 x 6.25 x 4 = 12.5 *kN/m* Slab Right = *S1* , w_{s2} = 0.5 $w\,L_x$ = 0.5 x 6.25 x 4 = 12.5 *kN/m* Total working load transferred from slab = 12.5 + 12.5= 25 *kN/m*
3.	*Beam Section* : Assume *b* = 230 *mm*. Depth of beam is worked out based on Span/ Depth ratio. The depth of the beam is also worked out corresponding to the support moment equal to 2/3 times simply supported moment for superimposed load transferred from slab. Then the suitable value for practical design is adopted. Assume *L/D* = 15, Depth of beam = 12000/15 = 800*mm* Approximate Support moment for superimposed load = $(1.5 \times 25 \times 12^2/8)$ = 675 *kN.m* Required depth = $\frac{2}{3} \times \sqrt{\frac{675 \times 10^6}{2.76 \times 300}}$ = 602 *mm* Assume beam section 300 *mm* x 650 *mm* ***Comments :*** *Since the moment of resistance varies with square of depth it is advantageous to adopt more depth than the width. But when the width of beam and column is kept equal to that of wall, the depth of the column required will be much more than the thickness of the wall due to which projection of column from the wall surface will be more. If it is objectionable from functional point of view then increase the width of beam, even though it is uneconomical.*
4.	*Total Load* : Beam self weight = 25 (0.65 – 0.13) x 0.3 = 3.9 *kN/m* Total Ultimate Load = w_u = 1.5 (25 + 3.9) = 43.4 *say 44 kN/m* Fixed end moment = $w_u L^2/12 = 44 \times 12^2/12$ = 528 *kN.m*
5.	*Column Section* : The column section is selected based on study of bending moment for different ratios of moment of inertia of beam (I_{BC}) to moment of inertia of column (I_{AB}) (for details see *Sect. 10.2.2*) For case studies three ratios of I_{BC}/I_{AB} have been taken *viz* $I_{BC}/I_{AB} = 1$, $I_{BC}/I_{AB} = 2$ and $I_{BC}/I_{AB} = 3$ The intention of doing this exercise is to understand how proportioning of its elements affect the load response, and to make the Structure economical and practical. The moment distribution process is given for one case. The advantage of odd span symmetry taken and the rotational stiffness factor for beam is taken to be equal to 2*EI/L* and distribution carried out for half frame. ***Case I :*** $I_{BC}/I_{AB} = 1$

Joint	Member	*R.S.F.	Sum	*D.F
B	BA	4EI/6 = 0.667	0.834	0.8
	BC	2EI/12 = 0.167		0.2

Case II : $I_{BC}/I_{AB} = 2, \therefore I_{BC} = 2\,I_{AB}$

Joint	Member	*R.S.F.	Sum	*D.F
B	BA	$4EI_{AB}/6 = 0.667$	1	0.667
	BC	$2E(2I_{AB})/12 = 0.333$		0.333

Case III : $I_{BC}/I_{AB} = 3 \quad \therefore I_{BC} = 3\,I_{AB}$

Joint	Member	*R.S.F.	Sum	*D.F
B	BA	$4EI_{AB}/6 = 0.667$	1.167	0.57
	BC	$2E(3\,I_{AB})/12 = 0.5$		0.43

*R.S.F. = Rotational Stiffness Factor and and *D.F. = Distribution Factor

$$\text{Fixed end moment} = w_u L^2/12 = 44 \times 12^2/12 = 528\ kN.m$$

Case - I : $I_{BC}/I_{AB} = 1$

Member	AB	BA	BC
Distribution Factor	---	0.8	0.2
Initial F.E.M.	---	---	−528
Distributed moments		422	106
Carry over moment	211		
Final moment	211	422	−422

$$\text{Mid-span moment} = w_u L^2/8 - 403 = 44 \times 12^2/8 - 422 = 370\ kN.m$$

The results of all three cases are given in the following table :

CASE No.	I_{BC}/I_{AB}	M_{AB}	M_{BA}	M_{BC}	M_E
I	1	211	422	−422	370
II	2	176	352	−352	440
III	3	150	301	−301	491

Comments : *From the above table it can be seen that if I_{BC}/I_{AB} is unity (i.e. moment of inertia of the beam is equal to moment of inertia of column), then the joint moment becomes much greater than the span moment. This will need large section of column and the beam will become doubly reinforced at support while the span moment will be less than the support moment with the result the advantage of the flanged section will not be obtained. On the other hand if I_{BC}/I_{AB} is large (say case 3) the joint moment is much lesser than the span moment. In this case the advantage of flanged section will be available but the beam may act as a singly reinforced section at support and the minimum bottom steel required to be continued to support will not be useful in resisting compression (See Sect. 3.3.3). Further the advantage on account of redistribution of moment cannot be taken. Hence I_{BC}/I_{AB} = 2 is selected. However, it may be noted that the the distribution of moments between column and beam is a function of stiffness of the connected members and **NOT** on the ratio of moment of inertia.*

Assume $I_{BC}/I_{AB} = 2$

Since the width of beam and column is to be kept the same, the depth of the column D is given by

$b \times 650^3/12 = 2\,(b \times D^3/12)$ or $D^3 = 650^3/2$

$\therefore D = 516\ mm$ say $550\ mm$

Assume column section of 300*mm* x 550*mm*

$$I_{BC}/I_{BA} = \frac{b \times 650^3/12}{b \times 550^3/12} = 1.65$$

Now, for the assumed values of sections of beam and column the design moments are worked our

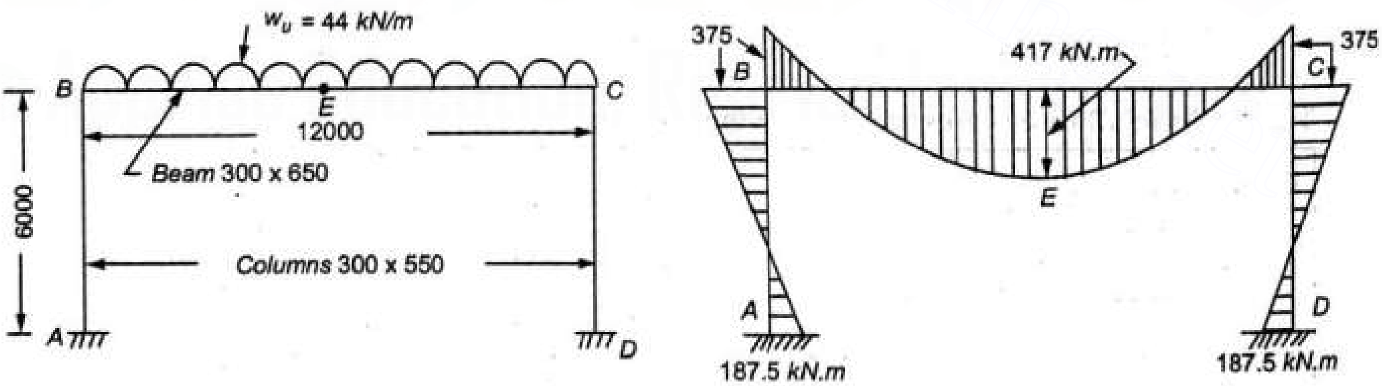
Joint	Member	R.S.F.	Sum	D.F.
B	BA	$4EI_{AB}/6 = 0.667$	0.942	0.71
	BC	$2E(1.65I_{AB})/12 = 0.275$		0.29

The final moments as per the section adopted are calculated as under :

Member	AB	BA	BC	M_E
Distribution factors		0.71	0.29	
Initial F.E.M.	—		−528	
Distributed moments		375	153	
Carry over moment	187.5			
Final moments	187.5	375	−375	417

Mid-span moment = $44 \times 12^2/8 - 375 = 417\ kN.m$

The section of the portal frame and bending moment diagram are shown in *Fig. 10.3.2*



(a) *Section of Portal* (b) *Bending Moment Diagram*

Fig. 10.3.2 Fixed Base Portal Frame

10.3.4 Design of Portal (Without Redistribution of Moments.)

(A) *Design of Beam*

Effective Span $L = 12\ m$

Ultimate load $w_u = 44\ kN/m$

Reaction at supports = $44 \times 12/2 = 264\ kN$

Section 300*mm* x 650 *mm*

Assuming #25*mm* diameter bar provided in two layers and #8*mm* diameter of stirrups ,

for mild environment, nominal cover = 25 *mm* (*Table C - 1*)

∴ Effective cover = 25 + 8 + 25 + 25/2 = 70.5 *mm*

∴ Effective depth provided = d = 650 – 70.5 = 579.5 *mm*

(*a*) *Mid-span section*

Design sagging moment = 417 *kN.m*

b_f = 0.7 x 12000/6 + 6 x 130) + 300 = 2480*mm* < *c/c* spacing (= 4000 *mm*) (*Eq.4.3.2*)

For $x_u = D_f$,

M_{ur} = 0.36 x 20 x 2480 x 130 (579.5 – 0.42 x 130) x 10^{-6} = 1218 *kN.m* > 417 *kN.m*

$\therefore x_u < D_f < x_{u.max}$ (= 278.16 *mm*)

$$\text{Required } A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 417 \times 10^6}{20 \times 2480 \times 579.5^2}}\right] \times 2480 \times 579.5$$

$$= 2055 \; mm^2$$

Provide 4 - # 25 *mm* + 2 – # 12*mm*, Area provided = 2190 mm^2

Check for cover :

Required cover = 25 + 8 + 25 + 25/2 = 70.5 *mm* (= assumed value)

Bar curtailment :

Minimum area of steel required to be extended into the support

$= A_{st}/4 = 2055/4 = 514 \; mm^2$

Curtail 2 bars of # 25 *mm*

No.of bars to be continued into the support = 2-#25 + 2#12*mm* giving area = 1208mm^2 > 514 mm^2

$$M_{ur} \text{ of } 2\#25mm+2\#12mm = 0.87 \times 415 \times 1208 \times \left(579.5 - \frac{415 \times 1208}{20 \times 300}\right) \times 10^{-6}$$

$$= 216 \; kN.m$$

Let x be the distance of the theoretical point of curtailment (*TPC*) from the support,

$264x - 44 \times x^2/2 - 375 = 216$

$x^2 - 12x + 26.8 = 0$

$(x - 6)^2 - (3.03)^2 = 0$

$\therefore x = 2.97m$

The bars to be curtailed are required to be extended through a minimum distance of effective depth from *TPC*

∴ Distance of actual point of curtailment *APC* = 2970 – 579.5 = 2390 *mm* ,

= say 2.39*m* from support.

(*b*) *Support section* :

Design moment at support= 375 *kN.m*

$M_{ur.max}$ = 2.76 x 300 x 579.5^2 x 10^{-6} = 278*kN.m* < 375 *kN.m*

∴ Section is doubly reinforced.

Balance moment = M_{u2} = 375 – 278 = 97 *kN.m*

Tension Steel :

$$A_{st1} = \frac{278 \times 10^6}{0.87 \times 415 \times (579.5 - 0.42 \times 0.48 \times 579.5)}$$

$$= 1664\ mm^2$$

$$A_{st2} = \frac{97 \times 10^6}{0.87 \times 415 \times (579.5 - 70.5)}$$

$$= 528\ mm^2$$

Total tension steel $= A_{st} = A_{st1} + A_{st2} = 1664 + 528 = 2192 mm^2$

Provide 4 - # 25 mm + 2#12mm , Area provided = 2190 mm^2 $\cong$ 2192 mm^2

Compression Steel :

For d_c/d = 70.5/579.5 = 0.12

A_{sc} = 1.0654 x 528 = 563 mm^2 (Table 4.2.3)

Reinforcement available is 2#25+2#12 = 1208 mm^2 > 563 mm^2 ∴ o.k.

Points of contraflexures :

End reaction = 44 x 12/2 = 264 kN

$x_{max} = R_A/w$ = 264/44 = 6m (Eq. 2.6.1)

Points of contraflexure from left support,

$$x_1 = x_{max} - \sqrt{x^2_{max} - 2M_A/w} \qquad \text{(Eq. 2.6.4)}$$

$$= 6 - \sqrt{36 - 2 \times 375/44}$$

$$= 6 - 4.36$$

$$= 1.64m$$

$$x_2 = 6 + 4.36 = 10.36\ m \qquad \text{(Eq. 2.6.5)}$$

Provide 4-#25mm bars at top of support beyond point of contraflexure for a distance:

> d (= 579.5 mm)

> 12 x dia of bar (= 300 mm)

> Clear span/16 (= 720 mm)

Provide top bars 4-#25mm for distance of (=1.64 + 0.72) say 2.4m from the face of support.

Check for Development Length :

$M_1/V_1 + L_o > L_d$ or $M_1 = V_1 (L_d - L_o)$ (Eq. 4.6.3a)

V_1 = shear force at point of contraflexure (TPC) = 264 –44 x 1.64 = 191.8 kN

L_d = 47φ = 47 x 25 = 1175 mm

L_o = Greater of [12φ (= 300mm) or d (=579.5 mm)] = 0.58m

M_1 = Moment of resistance of 2#25mm (982mm^2) curtailed bars

$$= 0.87 \times 415 \times 982 \times \left(579.5 - \frac{415 \times 982}{20 \times 300}\right) \times 10^{-6}$$

$$= 181.4\ kN.m$$

$$\frac{181.4}{191.8} + 0.58 = 1.52\ m > 1.175\ m\ \therefore \text{safe}$$

Design of Shear Reinforcement :

$V_{u.max}$ = 264 kN , V_{uD} = 264 – 44 (0.275 + 0.5795) = 226.4 kN

The area of tension steel available at top of support is 2190 mm^2. But the bottom tension steel available beyond TPC is only 1208 mm^2. Since the zone of shear reinforcement is not likely to be greater than the distance of TPC from support 4-#25mm + 2-#12mm bar area equal to 2190 mm^2 is taken for computation of shear resisted by concrete (*i.e.* V_{uc}).

$p_t\%$ = 100 x 2190/(300 x 579.5) = 1.26% $\therefore \tau_{uc} = 0.672\ N/mm^2$ *(Table.4.7.1)*

V_{uc} = 0.672 x 300 x 579.5/1000 = 116.8 kN

$V_{usv.min}$ = 0.4 x 300 x 579.5/1000 = 69.5 kN

$V_{ur.min}$ = 116.8 + 69.5 = 186.3 kN < V_{uD} (= 226.4 kN)

∴ Design shear reinforcement is required.

$V_{us} = V_{uD} - V_{uc}$ = 226.4 – 116.8 = 109.6 kN

Assuming #8mm 2-legged stirrups of Fe 415 (A_{sv} = 100.5 mm^2)

$$\text{Spacing, } s = \frac{0.87 \times 415 \times 100.5 \times 579.5}{109.6 \times 1000} = 191\ mm \text{ say} = 190\ mm$$

Length of shear zone, L_{s1} = (264 –186.3)/44 = 1.8m < TPC (=2.97m)

Provide #8mm 2-legged stirrups at 190mm c/c for distance of 1.8 m from support.

Area of tension steel at mid-span = 2190 mm^2

$p_t\%$ = 100 x 2190/ (300 x 579.5) = 1.26%

$\therefore \tau_{uc}$ = 0.672 N/mm^2

V_{uc} = 0.672 x 300 x 579.5 / 1000 = 116.8 kN

Length of nominal shear reinforcement zone : L_{s3} = 0.5 x 116.8/44 = 1.32 say 1.3m

Length of minimum shear reinforcement zone : $L_{s2} = L/2 - L_{s1} - L_{S3}$ = 6 – 1.8 – 1.3 = 2.9m

Spacing of #8mm 2-legged stirrups,

s = 0.87 x 415 x 100.5/(0.4 x 300) = 300mm

Provide #8mm 2-legged stirrups at 300mm c/c for length of 2.9m and for remaining distance of 1.3m provide #6mm 2-legged nominal stirrups at 300mm c/c.

Thus, Provide #8mm stirrups at 190mm c/c for 1.8m from the centre of support and #8mm Stirrups at 300mm c/c from 1.8m to 4.7 m from the centre of support and for the remaining middle span of 2.6m provide ϕ6 mm Stirrups at 300mm c/c

Check for Deflection : Basic L/d ratio = 26

Area of steel required at mid-span = 2055 mm^2

Area of steel provided at mid-span = 2189 mm^2

f_s = 0.58 x 415x2055/2189 = 226 N/mm^2 *(Eq. 4.7.1 b)*

% steel provided = 100 x 2189/(2480 x 579.5) = 0.15%

Modification factor α_1 = 1.9 , for p_t = 0.15% and f_s = 226 N/mm^2 *(Fig. 4.7.1)*

For flanged section, b_w / b_f = 300/2480 = 0.012 < 0.3

∴ Modification factor α_3 = 0.8 *(Fig. 4.7.3)*

Required effective depth = $L/(26 \times \alpha_1 \times \alpha_3)$ = 12000/(26 x 1.9 x 0.8)

= 303 mm < 579.5mm ∴ safe

Design of Column :

(a) *Top section*

Ultimate axial load P_u = 264 kN

Ultimate moment M_{ux} = 375 kN.m

Section of column = 300mm x 550mm

Height of column = 6m

Unsupported length of column = 6000 – 650/2 = 5675mm

Bending about x-axis bisecting depth of column:

e_{minx} = 5675/500 + 550 / 30 = 30 mm

M_{minx} = P_u x e_{minx} = 264 x 30/ 1000 = 7.9 kN.m < 375 kN.m ∴ M_{ux} = 375 kN.m

Since the column is subjected to vertical load only and its top end is held in position hence L_{eff} is assumed to be equal to 0.8 x unsupported length of column, on a safer side.

L_{eff} = 0.8 x 5675mm = 4540 mm

L_{eff}/D = 4540/500 = 9.1 < 12 ∴ Column is short

Assuming 8 mm lateral ties, and 25 mm diameter of bars ,

Effective cover = 40 + 8 + 25/2 = 60.5 mm

d_c/D = 60.5/550 = 0.11

$$\frac{P_u}{f_{ck}bD} = \frac{264 \times 10^3}{20 \times 300 \times 550} = 0.08 \quad \text{and} \quad \frac{M_u}{f_{ck}bD^2} = \frac{375 \times 10^6}{20 \times 300 \times 550^2} = 0.2$$

From interaction Chart 2G and Chart 3G by interpolation

d_c/D	$P_u/f_{ck}bD$	$M_u/f_{ck}bD^2$	p/f_{ck}
0.1	0.08	0.2	0.12
0.15	0.08	0.2	0.128
0.11	-	-	0.122

p/f_{ck} = 0.122, ∴ p = 0.122 x 20 = 2.44%

A_{sc} = (2.44 x 300 x 550)/100 = 4026 mm^2

Provide 8 - # 25mm bars *distributed equally on each face* (*i.e.* 4#25mm on each face) and 2-#12mm bars at mid-depth of section so that the spacing of longitudinal bars measured along the periphery of the column does not exceed 300 mm.

Total area provided = 4153mm² > 4026mm²

Bending about y-axis bisecting width of column :

The column is braced at lintel level by beam of size 300mm x 300mm at height of 2 m above plinth level.

Unsupported length of top section of column,

= 6 – (1.4 +0.4+2 +0.3) = 1.9 m

e_{miny} = 1900/500 + 300/30 = 13.4 mm < 20 mm ∴ e_{miny} = 20mm

M_{miny} = P_u x e_{miny} = 264 x 20/1000 = 5.28 kN.m

L_{effy}/b = 1900 / 300 = 6.3 < 12 ∴ Column is short

d_c/b = 60.5 / 300 = 0.2

$$\frac{P_u}{f_{ck}bD} = \frac{264 \times 1000}{20 \times 300 \times 550} = 0.08 \quad \text{and} \quad \frac{M_u}{f_{ck}bD^2} = \frac{5.24 \times 10^6}{20 \times 300 \times 550^2} = 0.03$$

From interaction Chart 5G , p/f_{ck} is very small ∴ safe.

(b) Bottom Section

Assuming depth of footing = 600 *mm*

Ultimate axial load at top of footing = 264 + [25 x 0.3 x 0.55 x (6 – 0.6–0.65/2)] x 1.5 = 295.4 *kN* say 296 *kN*

M_{ux} = 187.5 *kN.m*

d_c/D = 60.5/550 = 0.11

$P_u/(f_{ck}\,bD)$ = 296 x 1000 / (20 x 300 x 550) = 0.09

$M_{ux}/(f_{ck}\,bD^2)$ = 187.5 x 10^6 / (20 x 300 x 550^2) = 0.10

Using interaction diagram Chart - 2G and Chart 3G ,

d_c/D	$P_u/f_{ck}\,bD$	$M_u/f_{ck}\,bD^2$	p/f_{ck}
0.10	0.09	0.10	0.042
0.15	0.09	0.10	0.052
0.11	-	-	0.044

p = 0.044 x 20 = 0.88%   Area of steel = 0.88 x 300 x 550 / 100 = 1452 mm^2

Provide 4 - #25*mm* with 2 Nos. on each face and additional bars 2 - #12*mm* at mid-depth so that the spacing of longitudinal bars measured along the periphery of column does not exceed 300*mm*.

Area provided = 2189 mm^2 > 1452 mm^2

The top section of the column requires 8 Nos of 25 *mm* diameter bars while the bottom section of the column need much less than 4 nos. of 25 *mm* diameter of bars. Therefore, 4 Nos. of 25 *mm* diameter of bars can be theoretically curtailed at a distance of *h/3* (= 6/3 *m*) from top where the bending moment will be equal to *M*/2. Allowing for a bond length of 38 times diameter, the actual point of curtailment will be at a distance of , 6/3 + 0.65/2 + 38 x 0.025= 3.3*m* from top ∴ Curtail 4-#25*mm* at 3.3*m* from top.

A 90° opening re-entrant corner tends to open under loads (see *Fig. 5.5.1*). A haunch is provided to force the plastic hinge away from the face of joint.

Research[10.4,10.5,10.6] has shown that distribution of stresses at opening corner indicates that high tensile stresses exists at inner corner normal to the diagonal coupled with tensile stresses along the diagonal. This requires reinforcement along and across the diagonal.

It is recommended[10.6] that chamfer of size equal to the thickness of the framing member with reinforcement comprising of hoop steel and inclined bars of equal area should be provided. *Accordingly the detailing of reinforcement has been shown in Fig. 10.3.4.*

Lateral Ties : Minimum diameter = 25/4 = say 8 *mm*

Using #8 *mm* lateral ties. Spacing shall be least of (16 x 12 *or* 300 *mm or* 300 *mm*)

Provide #8 *mm* lateral ties at 190*mm* c/c.

(C) Design of Footing[10.7]

Ultimate load at column base = P_u = 296 *kN*

Ultimate moment at column base = M_{ux} = 187.5 *kN.m*

Size of column = 300 *mm* x 550 *mm*

Safe bearing capacity of soil = 400 kN/m^2

(a) Proportioning of base size :

Ultimate load transferred from column at base = P_u = 296 *kN*

Self weight of footing 10% = 30 *kN*

Total ultimate load = 326 *kN*

Safe Bearing capacity of soil = f_b = 400 kN/m^2

Area of footing required = A_f = 326/(1.5 x 400) = 0.54 m^2

Bending moment at column base = M_{ux} = 187.5 *kN.m*

Eccentricity of load at column base $e = M_{ux}/P_u$ = e = 187.5 / 296 = 0.63 *m* = 630 *mm*

Provide offset of 100 *mm* for seating of formwork for column

Size of plain concrete pedestal = 500 *mm* x 750 *mm* x 300 *mm* deep.

Area provided by pedestal = 0.5 x 0.75 $= 0.375\ m^2$

For concrete pedestal ,

$$\tan\alpha > 0.9\sqrt{\frac{100q}{f_{ck}} + 1} \qquad \textit{(clause 34.1.3)}$$

Where, α = Angle between bottom edge of the pedestal and the corresponding junction edge of the column with pedestal

q = calculated maximum bearing pressure at the base of pedestal in N/mm^2

$= 296 / (1.5 \times 0.375 \times 1000) \qquad = 0.53\ N/mm^2$

$\tan\alpha = 0.9\sqrt{(100 \times 0.53 / 20) + 1} \qquad = 1.72$

$\tan\alpha$ provided = 300 / 100 $= 3 > 1.72$ $\therefore$ Safe

Provide width of footing equal to width of pedestal $= B_f = 500\ mm$

Length of footing required $L = A_f / B_f = 0.54 / 0.5 \qquad = 1.08\ m$

Provide eccentric footing such that the C.G. of load from the column coincides with the C.G. of footing resulting in uniform pressure distribution.

Minimum length of footing to effect uniform pressure distribution

$= 2\ (e + D/2 + \text{offset})$

$= 2(630 + 550/2 + 100) = 2010\ mm$

Provide footing of size 500*mm* x 2200*mm*

Area of footing provided = 2.2 x 0.5 $= 1.1\ m^2 > 0.54\ m^2$

Comments : *Alternatively, the rectangular footing can be provided with linearly varying pressure distribution viz.*

(i) For no tension condition at the base : Length of footing required = 6e = 6 x 0.63 = 3.78 m.

So, the minimum size of footing required = 3.8 x 0.5

Area of footing $= 1.9\ m^2 > 1.05\ m^2 >> 0.54\ m^2$

Since the area of footing required is much more and pressure distribution being non - uniform this type of footing will be uneconomical.

(ii) If tension is permitted to be developed at the rear of the base, then the length of the footing required is given by :

$$L_f = \frac{2M_u}{P_u} + \frac{4\ P_u}{1.5 \times 3 \times f_b \times B_f} = \frac{2 \times 187.5}{296} + \frac{4 \times 296}{1.5 \times 3 \times 400 \times 0.5} = 2.58\ m$$

Length of footing in contact with soil $= 3(L_f / 2 - e) = 3(2.58 / 2 - 0.63) = 1.98m$

Area of footing $= 2.58 \times 0.5 = 1.29\ m^2 > 1.1\ m^2 \quad (= 2.2 \times 0.5)$

This footing in which tension develops at the rear of the base will be most uneconomical since only part length of footing will be in contact with the soil resisting the forces and in addition to this the pressure distribution will be nonuniform.

Minimum projection of the footing beyond column face $= L_f/2 - e - D/2 - 100$

$= 1100 - 630 - 275 - 100 = 95\ mm$

Maximum projection of the footing beyond column face $= C_x$

$= L_f/2 + (e - D/2 - 100)$

$= 1100 + (630 - 275 - 100) = 1355\ mm$

Intensity of ultimate soil pressure = $w_u = 296 / 1.1 \quad = 269\ kN/m^2$

(b) Depth of footing from bending moment considerations :

Bending moment at face of pedestal $M_{ux} = w_u \times B_f \times C_x \times C_x / 2$

$= 269 \times 0.5 \times 1.355^2/2$

$= 123.5\ kN.m$

$$\text{Required effective depth} = \sqrt{\frac{M_{ux}}{R_{u.max} \times b}} = \sqrt{\frac{123.5 \times 10^6}{2.76 \times 500}} = 300mm$$

Provide total depth of 600 *mm*

Effective depth provided = $d = 600 - 60 = 540\ mm$

Comments : *The depth of footing many times is governed by shear criteria and hence provided effective depth is taken much more than that required from bending moment considerations.*

The details of the footing are shown in *Fig. 10.3.3*

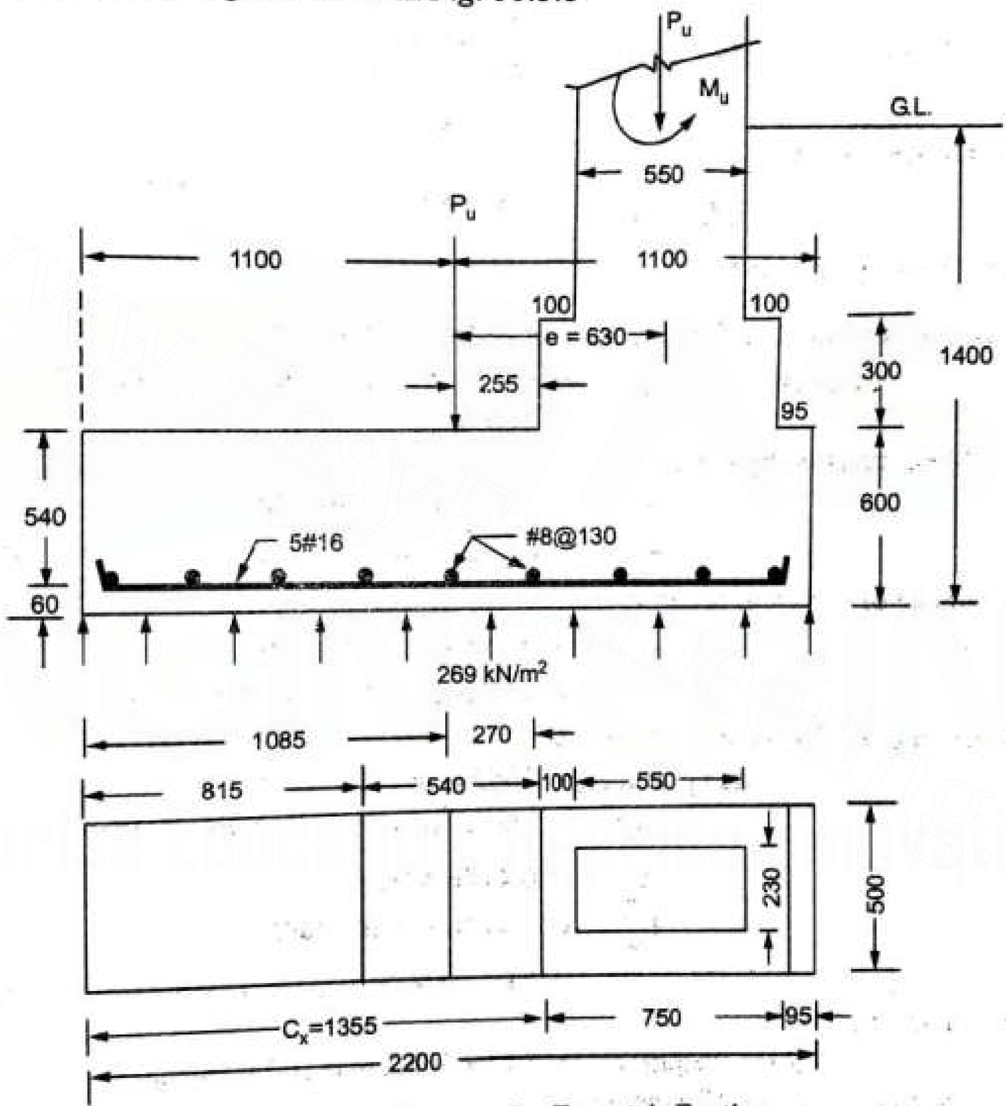


Fig. 10.3.3 Rectangular Eccentric Footing

(c) *Check for Two-way Shear :*

Critical section for two-way shear at distance $d/2$ (*i.e.* 270 *mm*) from the face of pedestal.

Perimeter at critical section = width of footing = 500 *mm*

Area resisting shear = width x d = 500 x 540 = 270000 mm^2

Shear strength of concrete :

$\tau'_{uc} = 0.25\sqrt{20} = 1.118\ N/mm^2$, $k_s = (0.5 + 500/750) > 1 \quad \therefore k_s = 1$

$\tau_{uc} = \tau'_{uc}\ k_s \quad \therefore \tau_{uc} = 1.118 \times 1 = 1.118\ N/mm^2$

Shear resistance of concrete = V_{uc2} = 1.118 x 270000 / 1000 = 301.9 *kN*

Design Shear = $V_{uD2} = w_u (C_x - d/2)\ B_f = 269\ (1.355 - 0.27) \times 0.5$

$= 146\ kN < V_{uc2}\ (= 301.9\ kN) \quad \therefore$ safe

(d) Area of Steel and Check for Development Length :

$$\text{Required } A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 123.5 \times 10^6}{20 \times 500 \times 540^2}}\right] \times 500 \times 540$$

$= 668\ mm^2$

Provide 4 - # 16*mm*, Area provided $= 804\ mm^2 > 668\ mm^2$

Clear distance between bars $= [(500 - 2 \times 50)/(4 - 1)] - 16$

$= 117\ mm$

Development required $(L_d)_{requd.} = \left[\dfrac{0.87 \times 415}{4 \times 1.2 \times 1.6}\right] \times 16$

$= 752\ mm$

Providing 90° bend

$(L_d)_{available} = 750 + 8 \times 16 = 878\ mm > 752\ mm$ ∴ safe

(e) Check for One-way Shear :

Critical section at distance d from the face of pedestal = 540*mm*

Design shear = $V_{uD} = w_u\ B_f (C_x - d) = 269 \times 0.5 \times (1.355 - 0.54)$

$= 109.6\ kN$

Shear resisted by concrete :

$p_t\% = 100 \times 804 / (500 \times 540) = 0.3\%$

$\tau_{uc} = 0.384\ N/mm^2$ *(Table 4.4.1)*

Shear resistance of concrete section :

$V_{uc} = 0.384 \times 500 \times 540/1000$

$= 103.6\ kN < V_{uD}\ (= 109.6\ kN)$ ∴ unsafe

Increase the steel *i.e.* Provide 5 - # 16 *mm*

Area provided = $1005\ mm^2$

$p_t\% = 100 \times 1005 / (500 \times 540) = 0.37\%$

$\tau_{uc} = 0.4176\ N/mm^2$

$V_{uc} = 0.4176 \times 500 \times 540/100$

$= 112.7\ kN > V_{uD}\ (= 109.6\ kN)$ ∴ safe

(f) Distribution steel :

Area required $= 0.12 \times 500 \times 600/100 = 360\ mm^2$

Provide # 8*mm*@ 130 *mm* c/c ,

Area Provided $= 387\ mm^2$

Details of footing :

Size of pedestal	: 750*mm* x 500*mm* x 300*mm* (deep)
Size of footing	: 2200*mm* x 500*mm* x 600*mm* (deep)
Main steel	: 5 - # 16*mm*
Distribution steel	: #8 @ 130*mm* c/c

The detailed drawing of fixed base portal frame without redistribution of moments is shown in *Fig. 10.3.4*

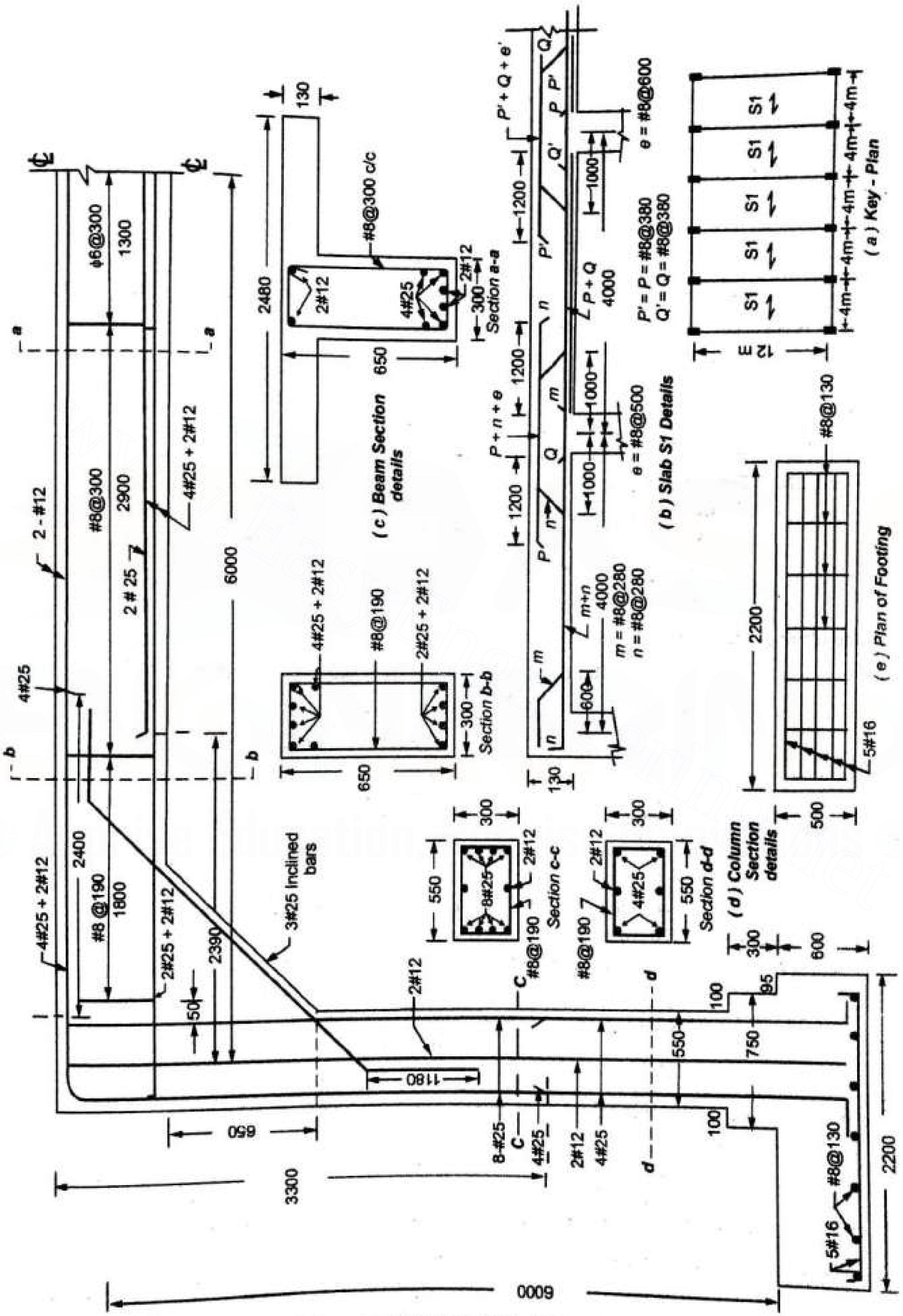


Fig. 10.3.4 Reinforcement Details of Fixed Base Portal Frame

10.3.5 Design of Portal Frame With 20% Redistribution of Moments

The design of slab and analysis of portal remains the same as given in sect. 10.3.2 and 10.3.3

Even though the bending moment at joint is less than the span moment in the beam, the decrease in support moment will effect in reducing the moment in the column thereby the steel requirement for column will reduce. Also the beam support steel will decrease. However, there will be increase in span moment and *marginal* increase in the positive steel because the central section acts as a flanged section.

Design of Beam :

(a) *Mid-span section :*

Section of beam : 300*mm* x 650*mm*,

Span L = 12*m*

Effective depth = 650 – 70.5 = 579.5 *mm* (Assuming effective cover 70.5*mm*)

Ultimate load = 44 *kN/m*

Flange width of beam b_f = 2480 *mm*

D_f = 130 *mm*

$w_u = 44\ kN/m$, $V_{u\text{-}max} = 264$ kN , $M_u = 375\ kN.m$

The support moments obtained from analysis in *Sect. 10.3.2* is 375 *kN.m*

When dM = 20% the design support moment = 375 x 0.8 = 300 *kN.m*

∴ Reduction in support moment = 375 - 300 = 75 kN.m

∴ Increase in span moment = 417 + 75 = 492 kN.m

$$k_{u.limit} = 0.6 - dM/100 = 0.6 - 0.2 = 0.4 < k_{u.max}$$

$$x_{u.limit} = 0.4 \times 579.5 = 231.8mm > D_f$$

The ultimate moment of resistance of T-beam for $x_u = D_f$ is given by :

$$(M_{ur1}) = 0.36 \times 20 \times 2480 \times 130\,(579.5 - 0.42 \times 130) \times 10^{-6}$$

$$= 1218\ kN.m > M_u\ (= 492\ kN.m)$$

$$\therefore x_u < D_f < x_{u.limit} \qquad \text{Section is under reinforced.}$$

$$\text{Required } A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 492 \times 10^6}{20 \times 2438 \times 579.5^2}}\right] \times 2438 \times 579.5$$

$$= 2438\ mm^2$$

Provide 4 -#25*m* + 2 -#16*mm* , Area provided = 2366 mm^2 ≅ 2438mm^2

Curtail 2#25*mm* ∴ No. of bars continued into support = 2-#25 + 2-#16

Moment of resistance M_{ur}, of bars of 2-#25*mm*+ 2-#16*mm* (area =1383mm^2) continued into support is :

$$M_{ur} = 0.87 \times 415 \times 1383\left(579.5 - \frac{415 \times 1383}{20 \times 300}\right) \times 10^{-6}$$

$$= 242\ kN.m$$

Let x be the distance of theoretical point of curtailment (*TPC*) from support

Then, $264x - 44\,x^2/2 - 300 = 242$

or $x^2 - 12x + 24.6 = 0$,

$\therefore x = 2.62\ m$

Distance of actual point of curtailment *APC* = 2.62 – 0.5795 = 2.04 *m*

Comments : *If more number of bars are proposed to be curtailed say 3#25mm then APC works out to be 1.57m only and the required shear reinforcement will also be more. Thus, there is not much advantage in curtailing more reinforcement.*

(b) *Support section :*

Design moment at support = 300 $kN.m$

$M_{ur.limit} = R_{u.limit}\, bd^2$, Where, $R_{u.limit} = 0.36 \times 20 \times 0.4\,(1 - 0.416 \times 0.4) = 2.4\ N/mm^2$

$M_{ur.limit} = 2.4 \times 300 \times 579.5^2 \times 10^{-6} = 241.8\ kN.m < 300\ kN.m$

∴ Section is doubly reinforced.

$M_{u2} = 300 - 241.8 = 58.2 kN.m$

$$A_{st1} = \frac{241.8 \times 10^6}{0.87 \times 415 \times (579.5 - 0.42 \times 0.4 \times 579.5)}$$

$= 1389\ mm^2$

$$A_{st2} = \frac{58.2 \times 10^6}{0.87 \times 415\,(579.5 - 70.5)} = 317\ mm^2$$

Total tension steel $= A_{st} = 1389 + 317 = 1706\ mm^2$

Provide 3-#25mm + 2 -#12mm , Area provided $= 1698\ mm^2 \cong 1706\ mm^2$

Compression steel :

For $d_c/d = 70.5/579.5 = 0.12$ and $k_{u.limit} = 0.4$, $f_{sc} = 339.6\ N/mm^2$ *(Table 4.2.2)*

$$A_{sc} = \frac{0.87 \times 415 \times 317}{(339 - 0.45 \times 20)} = 347\ mm^2$$

Reinforcement available at bottom is 2-#25mm + 2-#16mm, Area provided (1383 mm^2) > 446mm^2

End reaction = 44 x 12/2 = 264 kN. ,

$x_{max} = R_A / w_u = 264 / 44 = 6m$

Points of contraflexure :

$x_1 = 6 - \sqrt{36 - 2 \times 300/44} = 1.27m$

Top tension bars to be continued beyond points of inflection shall be greater of :
d (= 580) *or* 12 x dia. (= 300) *or* clear span/16 [= (12000 – 550)/16 = 715 mm
Provide top bars 2-#25mm for distance of 2 m (1.27 + 0.715 = say 2m) from support and 2-#16 will be continued throughout the span and will also act as anchor bars.

Design of Shear Reinforcement.

$V_{u.max} = 264\ kN$. ,

$V_{uD} = 264 - 44\,(0.55/2 + 0.5795) = 226.4\ kN$

Since length of shear zone is likely to be greater than top bar length of 2 m, minimum bottom steel is taken for shear design on safer side.

Area of tension steel at bottom is considered = 2-#25mm + 2-#16mm, $A_{st} = 1383\ mm^2$

$p_t\% = 100 \times 1383 / (300 \times 579.5) = 0.79\%$, $\tau_{uc} = 0.57\ N/mm^2$ *(Table 4.4.1)*

$V_{uc} = 0.57 \times 300 \times 579.5/1000 = 99.1\ kN$

$V_{usv.min} = 0.4 \times 300 \times 579.5/1000 = 69.5\ kN$

$V_{ur.min} = 99.1 + 69.5 = 168.6\ kN < V_{uD}\ (= 226.4\ kN)$

Design stirrups are required.

Shear to be resisted by stirrups

$= V_{us} = V_{uD} - V_{uc} = 226.4 - 99.1 = 127.3\ kN$

Using # 8*mm* 2-legged stirrups,

Spacing $= \dfrac{0.87 \times 415 \times 100.5 \times 579.5}{127.3 \times 1000} = 165\ mm$ say 160*mm*

Provide #8*mm* 2-legged stirrups at 160*mm* c/c

Length of shear zone,

$L_{s1} = (264 - 168.6)/44 = 2.17m > TPC\ (=2.62m)$

Area of tension steel available beyond shear zone = 4-#25*mm* + 2-#16*mm* = 2366mm^2

$p_t\% = 100 \times 2366/(300 \times 579.5) = 1.36\%$

Design shear strength of concrete (for p_t = 1.36% and M20) = τ_{uc} = 0.692 N/mm^2 *(Table 4.4.1)*

Shear resisted by concrete V_{uc} = 0.692 x 300 x 579.5/1000 = 120.3 *kN*

Zone of nominal shear reinforcement = L_{s3} = 0.5 x 120.3/44 = 1.37 *m*

Zone of minimum shear reinforcement = L_{s2} = 6 – 2.17 – 1.37 = 2.46 *m*

Spacing of ϕ6 *mm* 2-legged stirrups, *s* = 0.87 x 250 x 56/(0.4 x 300) say 100*mm c/c*

Provide #8*mm* 2-legged stirrups at 160*mm c/c* for length of 2.1 *m* and ϕ6*mm* 2-legged stirrups at 100*mm c/c* for length of 2.5*m* and for remaining central region of 2.8*m* provide ϕ 6*mm* 2-legged nominal stirrups at 300*mm c/c.*

Design of Column : 300*mm* x 550*mm*

(a) Top Section :

Ultimate axial load P_u = 264 *kN*

Ultimate moment M_{ux} = 300 *kN.m*

Section of column = 300*mm* x 550 *mm*

Height of column = 6*m*

Unsupported length of column = 6.0 – 0.65/2 = 5.7*m*

Bending about *x*-axis (bisecting depth of column)

$e_{minx} = 5700/500 + 550/30 = 30\ mm$

$M_{minx} = P_u \times e_{minx} = 264 \times 30/1000 = 7.9\ kN.m < 300\ kN.m$

$\therefore M_{ux} = 300\ kN.m$

$L_{eff} = 0.8 \times 5700mm = 4560\ mm$

$L_{eff}/D = 4560/550 = 8.3 < 12$ ∴ Column is short

Assuming #8*mm* ties, effective cover d_c = 40 + 8 + 25/2 = 60.5*mm* ∴ d_c/D = 60.5/550 = say 0.11

$P_u/f_{ck}\,bD = 264 \times 1000/(20 \times 300 \times 550) = 0.08$

$M_u/f_{ck}\,bD^2 = 300 \times 10^6/(20 \times 300 \times 550^2) = 0.16$

d_c/D	$P_u/f_{ck}\,bD$	$M_u/f_{ck}\,bD^2$	p/f_{ck}
0.10	0.08	0.16	0.09
0.15	0.08	0.16	0.10
0.11	-	-	0.0904

From interaction diagram *Chart - 2G* and *Chart 3G* by interpolation

$p/f_{ck} = 0.138$, ∴ $p = 0.0904 \times 20 = 1.8\%$

$A_{sc} = 1.8 \times 300 \times 550/100 = 2970\ mm^2$

Provide 6–#25*mm* + 2 – #12*mm* in 3 rows, Area provided = 3171 mm^2 > 2970 mm^2

The arrangement of bar shall be as under :

Row	No - Dia.
1	3 - #25
2	2 - #12
3	3 - #25

Area provided = $3171 mm^2$, p_t = 1.90% .

Remarks: *Whatever may be the distribution of bars, accurate and speedy results can be obtained from the software 10.8 prepared by author. The results obtained from the software ;*

k_u = 0.38

(P_u) = 264 kN , $\#(P_u)_{prov.}$ = 309 kN

M_u = 300kN.m, $\#(M_u)_{prov}$ = 300 kN.m , A_{st} = $3171 mm^2$,

(b) At bottom section :

Ultimate axial load =264 + 25 x 0.3 x 0.55(6–0.6 –0.65/2) x 1.5 = 295 kN

Ultimate moment = M_{ux} = 300/2 = 150 kN.m > M_{minx}

d_c/D = 60.5/550 = 0.11

$P_u/f_{ck}\, bD$ = 295= 1000/(20 x 300 x 550) = 0.09

$M_u/f_{ck}\, bD^2$ = 150 x 10²/(20 x 300 x 550²)= 0.08

d_c/D	$P_u/f_{ck}\, bD$	$M_u/f_{ck}\, bD^2$	p/f_{ck}
0.10	0.09	0.08	0.03
0.15	0.09	0.08	0.039
0.11	-	-	0.032

Using interaction *Chart - 2G and Chart - 3G*

p/f_{ck} = 0.032 , p = 0.032 x 20 = 1.64%

Area of steel = 1.64 x 300 x 550/100 = 1056 mm^2

Provide 4-#25*mm* + 2 # 12 *mm* with 2-#25*mm* on each face and 2#12mm at mid-depth of the column

Area provided = 2190 mm^2 >> 1056 mm^2, p_t = 1.3%

Area provided is much more than required to facilitate termination of 2 Nos of #25 *mm* bars at distance of 3.3 *m* (=6/3 + 0.65/2 + 38 x 0.025) from top

Provide #8 *mm* lateral ties at 190*mm c/c*.

Design of Footing :

Ultimate load at column base = 295 kN

Bending moment at column base = 150 kN.m

Size of column = 300 mm x 550 mm

Safe bearing capacity of soil = 400 kN/m^2

(A) Proportioning of Base size :

Ultimate load transferred from column at base = P_u = 295 kN

Add self weight of footing at 10% = 30 kN

Total ultimate load = 325 kN

Area of footing required A_f = 325 / (1.5 x 400) = 0.54 m^2

Bending moment at column base = M_{ux} = 150 kN.m

Eccentricity of load at base *e* = 150 x 1000 / 295 = 510 mm

Provide concrete pedestal of size 500 mm x 750 mm x 300 mm deep.

Width of footing B_f = 500 *mm*

Minimum length of footing = $2(e + D/2 + 100) = 2(500 + 550/2 + 100) = 1770$ *mm*

Length of footing required = $L_f = A_f / B_f = 0.54 / 0.50 = 1.08 < 1.77$ *m*

Area of footing provided $= 1.77 \times 0.5 = 0.885\ m^2 > 0.54\ m^2$

Projection of footing beyond face of pedestal

$= C_x = L_f/2 + (e - D/2 - 100) = 1770/2 + (508 - 550/2 - 100) = 1018$ *mm say 1020mm*

Ultimate upward intensity of soil pressure $= w_u = 295/0.885 = 333\ kN/m^2$

(b) *Depth of Footing from Bending Moment Considerations :*

Bending moment at face of pedestal $= w_u\ B_f\ C_x^{\ 2}/2$

$= 333 \times 0.5 \times 1.02^2/2$

$= 86.6\ kN.m$

Required effective depth $= \sqrt{86.6 \times 10^6 / (2.76 \times 500)} = 251\ mm$

Provide total depth of 500 *mm*, effective depth = 500 – 60 = 440 *mm*

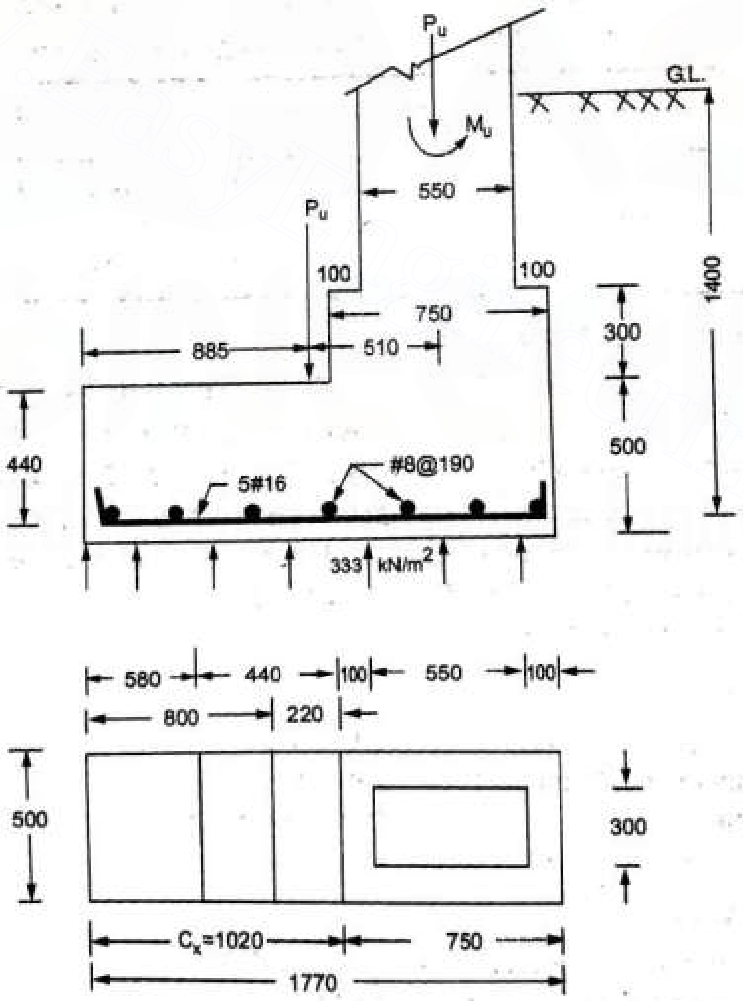


Fig. 10.3.5 Rectangular Eccentric Footing

(c) *Check for Two-way shear :*

Critical section is at distance *d*/2 from the face of pedestal

Perimeter at critical section = width of footing = 500*mm*

Area resisting shear $= 440 \times 500 = 220000\ mm^2$

Shear Strength of Concrete :

$\tau_{uc} = k_s \times \tau_{uc}\, k_s\, (0.25\sqrt{f_{ck}})$ where, $k_s = (0.5 + 500/750) > 1 \quad \therefore k_s = 1$

$\tau_{uc2} = (0.25\sqrt{20}) \times 1 = 1.118\ N/mm^2$

Shear resistance of concrete $= V_{uc2} = 1.118 \times 220000/1000 = 246\ kN$

Design shear force $= V_{uD2} = 333 \times 50 \times (1020 - 220) / 1000 = 133.2\ kN < V_{uc2} \quad \therefore safe$

(d) *Area of Steel :*

$$\text{Required } A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 86.6 \times 10^6}{20 \times 500 \times 440^2}}\right] \times 550 \times 440$$

$$= 635\ mm^2$$

Minimum area of tension reinforcement $= 0.85 \times 500 \times 440 / 415 = 388\ mm^2 < 635\ mm^2$

Provide 4-#16*mm*, Area provided $= 804\ mm^2$

Clear distance between bars $= (500 - 2 \times 40) / (4-1) - 16 = 140\ mm > 50\ mm$

(e) *Check for One - way Shear :*

Critical section at distance *d* from the face of pedestal $= 1020 - 440 = 580\ mm$

Design shear $= V_{uD} = 333 \times 0.50 \times 0.58 = 96.6\ kN$

Shear carried by concrete :

$p_t\% = 100 \times 804 / (500 \times 440) = 0.36\%$, $\tau_{uc} = 0.413\ N/mm^2$ *(Table 4.4.1)*

Shear resistance of concrete section

$= V_{uc} = 0.413 \times 500 \times 440/1000 = 90.8\ kN > V_{uD}\ (= 96.6\ kN) \quad \therefore$ unsafe

Provide 5-#16 *mm*, Area provided $= 1005\ mm^2$, $p_t = 100 \times 1005/(500 \times 440) = 0.461\%$

$\tau_{uc} = 0.461\ N/mm^2$, $V_{uc} = 0.461 \times 500 \times 440/1000 = 101.4\ kN > V_{uD}\ (=96.6\ kN)$

Distribution steel :

Area required $= 0.12 \times 500 \times 500/100 = 300\ mm^2$

Provide #8*mm* at 160*mm c/c*, Area provided $= 314\ mm^2$

Details of footing :

Size 1770 *mm* x500*mm* x 500 *mm* (deep)

Main steel 5 - #16*mm*

Distribution steel #8*mm* at 160 *mm c/c*

10.4 DESIGN OF HINGED BASE PORTAL FRAME[10.9]

The analysis of a hinged base portal frames is similar to that of fixed base portal frame except that the stiffness of the column will be 3*EI/L* instead of 4*EI/L*. The design of beam and column is to be worked out on the same lines as that of a fixed base portal frame. The only part that differs from that of fixed base portal frame is the design of hinges and foundation. The hinges are required to be designed to transmit shear force and permit rotation. The high value in compression is necessary to make the concrete plastic enough to permit required angular rotation. This high value in compression is obtained by constricting the section at the hinge and using compressive strength equal to half its ultimate stress. The length of the hinge shall not be more than twice its least lateral dimension. The slot at the hinge is filled up with bituminous material. The column main steel is terminated on each side of the hinged slot and hinge bars are crossed to resist the whole shear force at the base. The hinge bars are closely bound together by ties *or* spirals for dissipation of high stresses. The design of hinge and footing for a hinged base portal is given below :

Design of Hinge and Foundation for a Portal Frame *ABCD*

Design the hinge between portal frame and its foundation for the following data.

Ultimate load from column	= 274 *kN*.
Section of the column	= 230*mm* x 500*mm*
Section of the beam	= 230*mm* x 650*mm*
Distance of the hinge from the column base	= 1.4*m*
Ratio of M.I of beam to M.I of column I_{BC}/I_{AB}	= 2.2
Height of column above the hinge	= 4.6 *m*
Span of beam	= 12 *m*
Intensity of ultimate load on Portal	= 42 *kN/m*
Bearing Capacity of Soil	= 250 kN/m^2

Distribution factors :

Joint	Member	RSF	Sum	D.F.
B	BA	$3E\,I_{AB}/4.6 = 0.652$	1.019	0.64
	BC	$2E\,(2.2\,I_{AB})/12 = 0.367$		0.36

w_u = 42 *kN/m* , $M_{FBC} = 42 \times 12^2/12$ = 504 *kN.m* (see Sect.10.3.2 item 4)

The final moments are worked out as under :

Joint	A	B	
Member	AB	BA	BC
D.F	–	0.64	0.36
F.E.M	–	–	–504
Distribute		322	182
Final moment		322	–322

Ultimate shear at the hinge = 322/4.6 = 70 *kN*

Also design a suitable foundation if bearing capacity of the soil is 250 kN/m^2. Use concrete grade M20 and steel grade Fe415.

Design of Hinge :

For hinge action to persist the concrete will be made to reach nearly half its ultimate stress ($0.5f_{ck}$) so that the concrete will become plastic enough to permit small angular rotation. Also the reinforcement at the hinge will be crossed to produce the hinge action.

Ultimate axial load at column base = 274 *kN*. $\therefore P = 274/1.5 = 182.7$ *kN*

Area of hinge required $P/(0.5f_{ck})$ = 182.7 x 1000 x 20/2 = 18267 mm^2

Width of hinge = Area at hinge/width of column = 18267/230 = 79.4*mm*

Provide hinge of size 230*mm* x 100*mm*

Length of the hinge < 2 x smallest dimension at hinge cross - section

Provide Length of hinge = 120*mm* < 2 x 100*mm*

Ultimate shear at the hinge = 70 *kN*

This horizontal shear will be resisted by the reinforcing bars provided at the hinge.

Out of the two sets of bars marked *A* and *B* in *Fig. 10.3.6* the set *A* will be in tension while the other set *B* will be in compression.

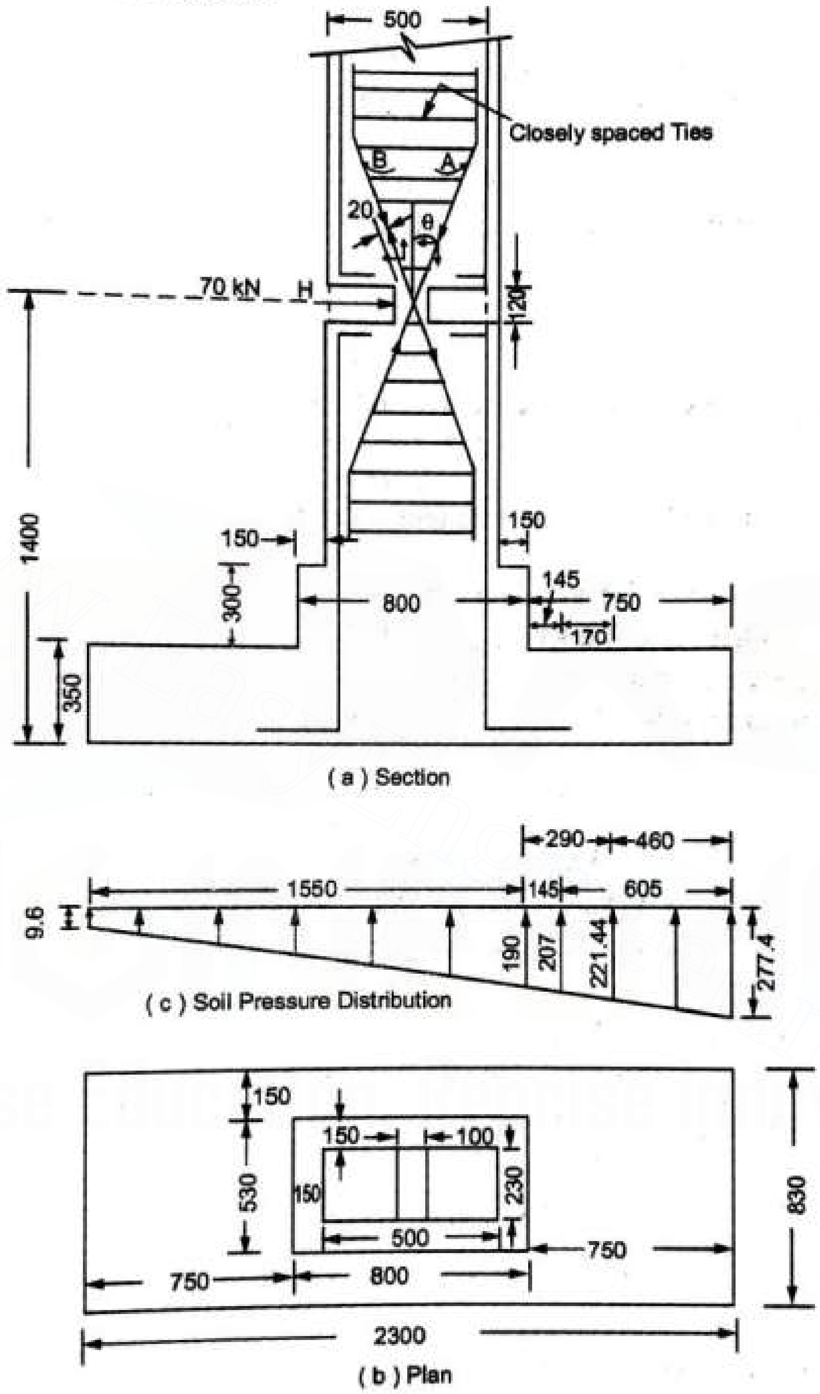


Fig. 10.3.6 Hinged Base for Portal Frame

For equilibrium : $(A_t \sin\theta) \times 0.87 f_y = H$

Where, A_t = total area of steel at the hinge

θ = inclination of reinforcing bars with vertical.

Providing minimum effective cover = 20*mm*

$\tan\theta = (50-20)/60 = 0.5, \theta = 26.56°$, $\sin\theta = 0.447$

$$A_t = \frac{70 \times 10^3}{0.87 \times 415 \times 0.447} = 434\ mm^2$$

Provide 2 bars of #12 *mm* dia. on each side of hinge (Area provided = 452 mm^2)and #10*mm* diameter closely spaced ties *or* stirrups to effect gradual dispersal of high stresses in constriction.

Design of Footing :

Ultimate load at column base P_u = 274 *kN*
Add self weight of footing at 10% = 28 *kN*
Total ultimate load = 302 *kN*
Area of footing required = 302/(1.5x 250) = 0.8 m^2
Ultimate moment at base = Shear at hinge x Distance of hinge from base

M_u = 70 x 1.4 = 98 *kN.m*

Eccentricity $e = M_u/P_u$ = 98 x 10^3/274
= 358 *mm*

Minimum Length of the footing for no tension condition
= 6*e* = 6 x 358
= 2148 *mm*

Select the size of the footing such that there is no tension at the base and bearing capacity of soil is not exceeded.

Provide the footing of size : 830*mm* x 2300*mm*
Area of footing provided = A_f = 0.83 x 2.3 = 1.9m^2 > 0.8 m^2
Provide concrete pedestal of size 530*mm* x 800*mm* x 300*mm* deep
Maximum and minimum intensities of pressure at base,

$$P'_{u.max} = \frac{274}{0.83 \times 2.3} + \frac{98}{0.83 \times 2.3^2/6}$$

$$= 158.20 + 133.9 = 292\ kN/m^2$$

$$P'_{u.min} = 158.2 - 133.9 = 24.3\ kN/m^2 > 0$$

Maximum intensity of pressure at working load

$P_{u.max}$ = 292/1.5 = 195 kN/m^2 < Soil Bearing Capacity (= 250 kN/m^2)

Ultimate upward intensity of soil pressure is given by

$$P_{u.max} = \frac{274}{0.83 \times 2.3} + \frac{98}{0.83 \times 2.3^2/6}$$

$$= 143.5 + 133.9 = 277.4\ kN/m^2$$

$$P_{u.min} = 143.5 - 133.9 = 9.6\ kN/m^2$$

Maximum pressure intensity at the face of pedestal

= 9.6 + (277.4 – 9.6) x 1550/2300 = 190 kN/m^2

Depth of Footing from B.M. Considerations :

Ultimate moment at face of pedestal ,

= M_{ux}
= (190 x 0.75 x 0.83) x 0.75/2 + (277.4 – 190) x 0.75 x 0.83 x 0.75x2/3
= 71.5 *kN.m*

Required effective depth = $\sqrt{71.5 \times 10^6/(2.76 \times 530)}$ = 221 *mm*
Provide total depth of 350*mm*
Effective depth provided = *d* = 350 – 62 = 290*mm*

Check for Two-way Shear :

Critical section is at distance $d/2$ (*i.e.* 145*mm*) from the face of pedestal

Stress at critical section $= 9.6 + (277.4 - 9.6) \times 1695/2300 = 207\ N/mm^2$

Perimeter at critical section $\cong 830mm$

Area resisting shear $= 830 \times 290 = 240700 mm^2$

Shear Strength of Concrete :

$\tau'_{uc} = 0.25\sqrt{20} = 1.118\ N/mm^2$,

$k_s = 0.5 + 530/800 > 1 \therefore k_s = 1$

$\tau_{uc} = 1 \times 1.118 = 1.118\ N/mm^2$

Shear resistance of concrete $= V_{uc2} = 1.118 \times 240700/1000$

$= 269\ kN$

Design shear $= V_{uD2} = (207 + 277.4)/2 \times 0.605 \times 0.83$

$= 121\ kN < V_{uc2}\ (=269\ kN)$ $\therefore$ safe

Area of Steel :

$$\text{Required } A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 71.5 \times 10^6}{20 \times 830 \times 290^2}}\right] \times 830 \times 290$$

$= 729\ mm^2$

Provide 4-#16*mm*, Area provided $= 804 mm^2$

Check for one-way shear :

Critical section is at distance d ($= 290mm$) from the face of pedestal

Distance of critical section from the edge of footing,

$= 750 - 290 = 460mm$

Stress at critical section $= 9.6 + (277.4 - 9.6) \times 1840/2300$

$= 223.8\ kN/m^2$

Design shear force $= V_{uD} = (277.4 + 223.8) \times 0.46 \times 0.83/2$

$= 95.7\ kN$

$p_t\% = 100 \times 804/(830 \times 290) = 0.33\ \%$, $\tau_{uc} = 0.3984\ N/mm^2$ *(Table 4.4.1)*

Shear resistance of concrete

$= V_{uc} = 0.3984 \times 830 \times 290/1000 = 95.9\ kN > 95.7\ kN$ $\therefore$ safe

Distribution steel :

Required area $= 0.12 \times 830 \times 350/100 = 349 mm^2$

Provide #8*mm* @ 140*mm* c/c

, Area provided $= 359\ mm^2$ *(Table H2)*

Details of Footing :

Size of pedestal	: 800*mm* x 530*mm* x 300*mm* deep
Size of footing	: 2300*mm* x 830*mm* x 350*mm* deep
Main Steel	: 4 - #16*mm*
Distribution steel	: #8*mm* @140*mm* c/c

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10.2 Shah V.L., "Select observations on IS : 456 - 2000" National workshop on IS : 456 - 2000, Maharashtra - India Chaper of ACI, Mumbai, Oct. 2000, *pp* 136 - 138.

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CHAPTER-11

DESIGN OF PORCH

11.1 INTRODUCTION

Porch is normally provided over an entrance of a building with slab projecting out for parking of car *or* from aesthetic considerations *or* for using it for a seat out. The porch slab is normally provided at lintel level that is at about 2.2*m* from bottom storey floor level. Sometimes it is also provided at the floor level. The projection of the slab is about 2.5*m or* less from the outer face of the wall.

11.2 DIFFERENT TYPES OF LAYOUTS

The different types of layouts used for the construction of the porch are given as under :

(a) Slab supported on cantilever beams which are embedded in wall. *(Fig. 11.2.1a).*
(b) Cantilever slab supported over beam, which is rigidly connected with columns *(Fig. 11.2.1b)*
(c) Slab simply supported on the beams with supporting end-beam resting on cantilever end of floor beams *(Fig. 11.2.1c)*
(d) Slab simply supported over the cantilever portion of floor beam (*Fig. 11.2.1d*).
(e) Slab supported along its edges by beams with columns at its ends .*(Fig. 11.2.1e)*

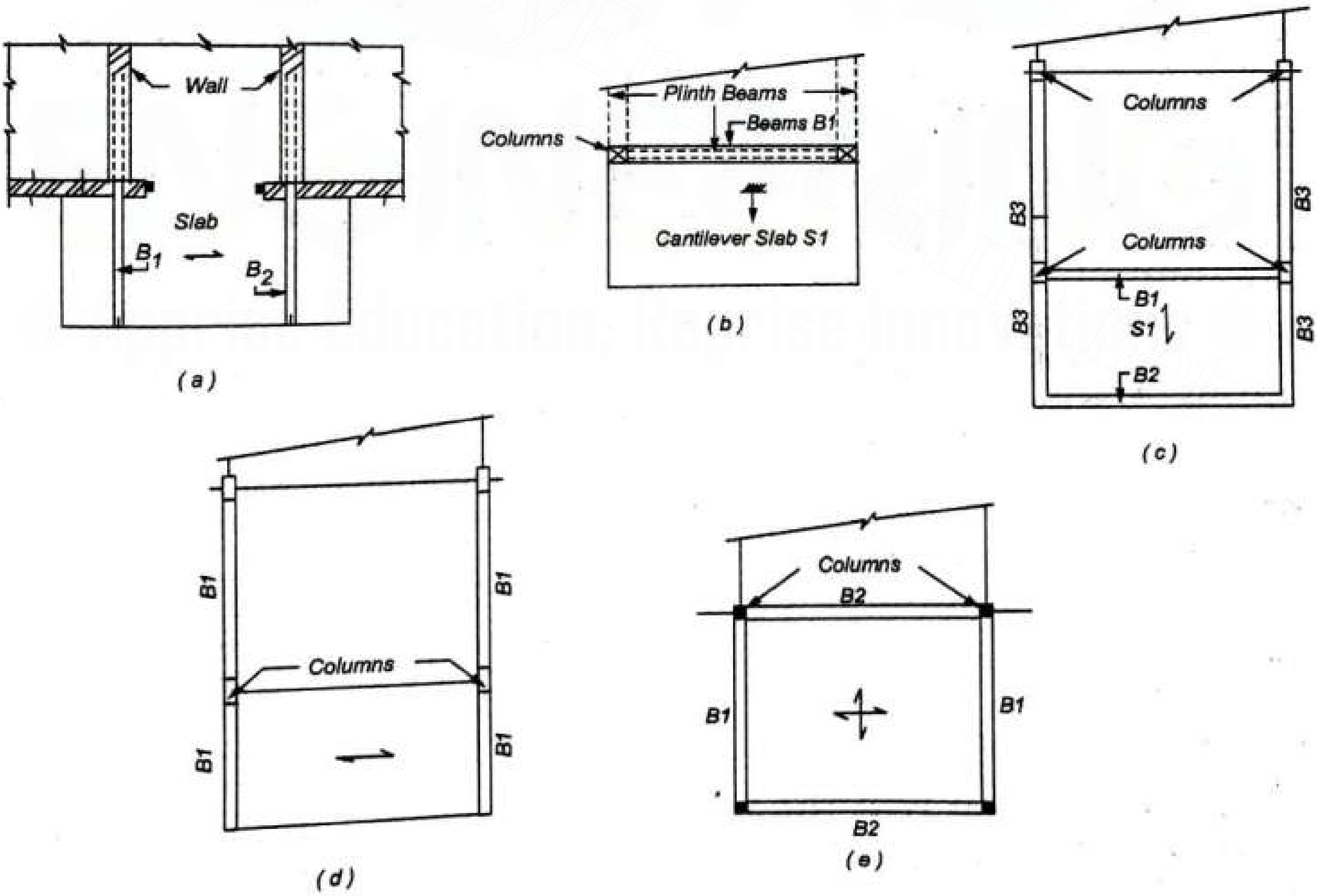


Fig. 11.2.1 Different layouts of Porch

11.2.1 Slab Supported on Cantilever Beams which are Embedded in Walls

In this case the slab is either simply supported over the beams *or* the slab may overhang on each side of the beam *(Fig. 11.2.1a)*. The overhanging portion of the slab helps to reduce the mid-span moment resulting in requirement of lesser thickness of the slab and the reinforcement. The cantilever beam supporting the slab is extended inside the wall to get the counterweight to maintain static equilibrium, even when overturning moment is doubled. The required counter weight is obtained from the floor slab and/or the wall over the embedded portion of the beam. Therefore, this type of construction is normally made in load bearing structures.

11.2.2 Cantilever Slab Supported over Beams which are Rigidly Connected with Columns

In this case, as the cantilever slab projects out from the beam, the beam is subjected to uniformly distributed twisting moment over its entire length in addition to bending moment and shear force *(See Fig. 11.2.1b)*. A rigid connection between the column and the beam is necessary to prevent rotation of the beam for resisting equilibrium torsion. The distribution of the moments between the column and the beam at the joint depends on the stiffness of the column in two orthogonal directions and the corresponding bending stiffness and torsional stiffness of the beam. If exact analysis is not carried out, the beam may be designed to resist full torsion and partial fixity may be assumed for calculating beam moments.

11.2.3 Slab Simply Supported on Beams with Supporting End-beam Resting on Cantilever ends of Floor Beams

Fig. 11.2.1c shows the porch slab is simply supported over the beams *B1* and *B2*. The beam *B2* is supported at the cantilever ends of floor beams *B3*. If the distance between the top of opening and the floor is not large, floor beams having depth equal to the distance between the top of opening and the floor are provided and cantilevered out to support the beam *B2*. However, if this gap is large then the floor beams of required depth are provided and the floor slab rests on the masonry provided above the beams *B3*, with beam *B2* supported at their cantilever ends of *B3*. The cantilever part of the beams *B3* may be tapered with depth at the cantilever end equal to the depth of beam *B2*. This type of arrangement is preferable when facial wall at the end of porch is required for advertisement *or* for name board etc. The *B2* is designed to resist the load of facial wall and the part load transferred from the porch slab. In this case, as the porch slab spans in the shorter direction the required thickness is much less. The superimposed load on the beam *B3*, will consist of load transferred from the floor slab and the wall load due to upper storey if any and a concentrated load from beam *B2* at its cantilever end.

11.2.4 Slab Simply Supported over Cantilever Portion of Floor Beam

When facial slab is not required the porch slab can be made to span across the cantilever portion of floor beams *B1* as shown in *Fig. 11.2.1d*. Since the length of the porch is more than its width the thickness of the slab works out to be more than the one required in the earlier case. The floor beams are provided in the same way as given in *Sect. 11.2.3* and will carry the load as detailed therein with the change that the cantilever portion of the beams *B1* will carry a uniformly distributed load transferred from the porch slab.

11.2.5 Slab Supported along all its Four Edges by Beams

In this case the slab is supported by beams along all its edges. The beams are supported by four columns provided at the corners of the porch as shown in *Fig. 11.2.1e*. This type of layout is very simple but generally not preferred from aesthetic considerations. This is suitable when very large size of porch is required to be provided.

11.3 ILLUSTRATIVE EXAMPLES

11.3.1 Over Hanging Porch Slab Supported on Beams

Ex. 11.3.1 A cantilever porch of size 2500*mm* wide x 5000*mm* long provided at a height of 2.2 m above floor level overhangs 2500*mm* beyond the face of the wall. The slab is supported by two cantilever beams each of length 2.5*m* spaced at 3*m* centre to centre so that the slab overhangs for a length of 1*m* on each side from the centre line of beam. The slab and beams are cast monolithic to give a flat soffit. Design the slab and the beam. *(Fig. 11.3.1)*

If the stabilizing load acting on each beam in the bearing portion due to the slab of adjoining rooms and wall above the beam is 35 kN/m, calculate the length of embedment of the beam inside the supporting brick wall of 250mm thick. Assume the following data :

Live load = 0.75 kN/m^2 , Load due to finish = 0.8 kN/m^2 , Mild environment , Steel grade Fe415

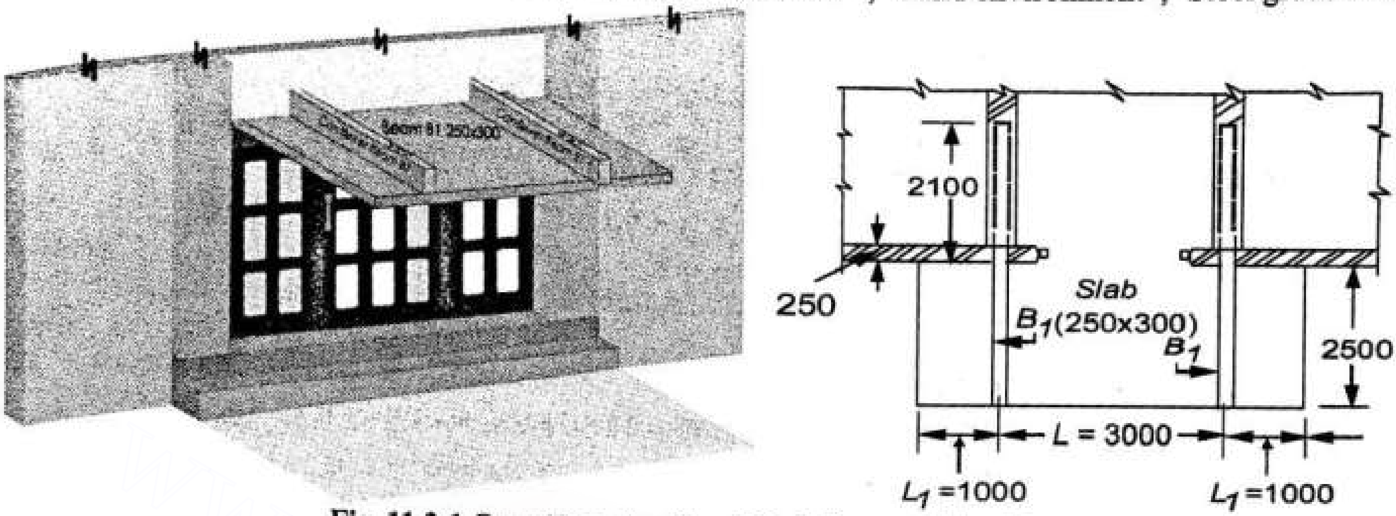


Fig. 11.3.1 Over Hanging Porch Slab Supported on Beams

Solution :

For mild environment f_{ck} = 20 N/mm^2 and nominal cover = 20 mm *(Table C1)*

f_{ck} = 20 N/mm^2 , f_y = 415 N/mm^2 , LL = 0.75 kN/mm^2

FF = 0.8 kN/m^2 , $k_{u.max}$ = 0.48 , $R_{u.max}$ = 2.76 N/mm^2

(a) Design of Slab

1. *Span :* Simply supported span L = 3.0m and cantilever span = L_1 = 1m.
2. Trial Depth : Even though the slab is simply supported, the overhanging portion of the slab effects in reducing the deflection and span moment.

   Hence, L/d will be taken to be equal to 26.

   Assuming p_t = 0.2%, Modification factor α_1 = 1.62

   Basic L/d = 26,

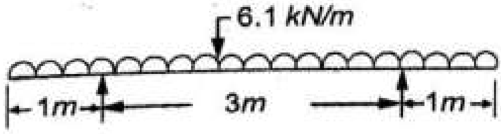
   Required effective depth = 3000/(1.62x26) = 72mm say 80mm

   Provide total depth D = 100mm ∴ d = 100 – 20 = 80mm
3. *Loads :* Consider 1m width of the slab *i.e.* b = 1000mm

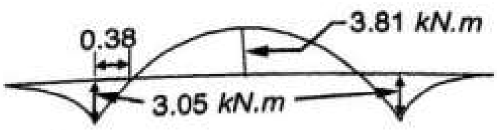
   Working load/m = $D.L.$ of slab + FF + LL = 25 x 0.10 + 0.8 + 0.75 = 4.05 kN/m

   Ultimate load w_u = 1.5 x 4.05 = say 6.1 kN/m

   The loading is shown in *Fig. 11.3.2a*



(a) Loading on beam B_1



(b) Bending Moment Diagram

Fig. 11.3.2

4. *Design Moments :*

Maximum - ve moment at mid-span :

$= w_u L_1^2/2 = 6.1 \times 1^2/2 = 3.05\ kN.m$

Support reaction $= R_A = 6.1 \times 5/2 = 15.25\ kN$

Maximum +ve moment at mid-span :

$= 15.25 \times 3/2 - 6.1 \times 2.5^2/2 = 3.81\ kN/m.$

Points of contraflexures :

Let the distance of point of contraflexure be x from A

$M_x = R_A.x - w_u(1+x)^2/2 = 0 \quad \therefore 15.25 \times x - 6.1 \times (1+x)^2/2 = 0$

$\therefore x = 0.38\ m$ or $x = 2.62\ m$

The bending moment diagram is shown in *Fig. 11.3.2b*

5. *Check depth from B.M. Criteria :*

$M_{ur.max} = R_{u.max} bd^2 = 2.76 \times 1000 \times 80^2 \times 10^{-6} = 17.66\ kN.m >> 3.81\ kN.m$

$\therefore$ Section is under-reinforced.

6. *Main Steel :*

$$\text{Area of Steel at mid-span} = A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 3.81 \times 10^6}{20 \times 1000 \times 80^2}}\right] \times 1000 \times 80 = 136 mm^2$$

Minimum reinforcement $= A_{st.min} = 0.12 \times 1000 \times 100/100 = 120 mm^2 < 136\ mm^2$

Using #8 *mm* bars (Area= 50 mm^2) ,

Spacing = 1000 x 50/136 = 367 *mm* <= (300*mm* or 3*d* =240 *mm*) whichever is less

$\therefore$ Provide #8 *mm* bars at 240 *mm* c/c, Area provided = 209 mm^2

$$\text{Area of steel at support} = A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 3.05 \times 10^6}{20 \times 1000 \times 80^2}}\right] \times 1000 \times 80$$

$= 108\ mm^2 < A_{st.min}\ (= 120\ mm^2)$

$\therefore A_{st} = 120 mm^2$

$$(p_t)_{reqd.} = \frac{100 \times 120}{1000 \times 80} = 0.15\%$$

Development length $= 47\phi = 47 \times 8 = 376 mm.$

Provide #8*mm* bars at 240 *mm c/c* at bottom. The alternate bars from bottom are bent up at 400 *mm* from centre of supports and remaining 50% bars are taken above the bottom reinforcement of beam to derive support benefit and terminate them at the outer face of the beam.

Also provide #8 *mm* bars at 480 *c/c* from the end of cantilever and take them inside the span for a distance of 400*mm* (> L_d) from the centre of support.

Area provided = #8 at 240 c/c = 209 mm^2

7. *Distribution Steel :*

$A_{st} = 1.2D = 1.2 \times 100 = 120\ mm^2$

Using #8*mm* bars, spacing = 1000 x 50/120 = 416 *mm* = 300mm (300*mm* or 5*d* =400 *mm*) whichever is less

Provide #8*mm* bars at 300*mm* c/c.

8. *Check for deflection :*

(a) At mid-span :

$(p_t)_{reqd.}$ = 100 x 136/(1000 x 80) = 0.17 % < 0.2% ∴ detailed check not carried out

(b) At support :

Basic $L/d = 7$, $(A_{st})_{reqd.} = 120 mm^2$, $(A_{st})_{prov.} = 209\ mm^2$

$f_s = 0.58 \times 415 \times 120/209 = 138\ N/mm^2$

For $p_t = 0.15\%$ and $f_s = 138\ N/mm^2$, Modification factor $\alpha_1 = 1.98$

$$\therefore (d)_{requd} = \frac{1000}{7 \times 1.98} = 72\ mm < 80\ mm.$$

Note : *In case of cantilevers, the depth required from deflection criteria is more because L/d ratio is only 7. In such case it is preferable to use mild steel reinforcement which gives more value of modification factor thereby the requirement of depth gets reduced to some extent.*

9. *Check for shear :*

As mentioned earlier, the slab being a shallower member fail at higher nominal shear stress and hence is safe against shear.

(b.) Design of Beam

1. Span :

Cantilever span = $L = 2.5m$ = 2500*mm*.

2. Trial depth :

The beam and slab are cast monolithic with rib of the beam provided above the slab. Since it is a cantilever beam the compression will be in bottom fibers and the slab will lie in the compression zone with respect to bending of the beam, therefore, the beam will act as a *T-beam*.

Assume the depth of the beam = D = 300*mm*,

For mild environment nominal cover = 20 *mm* ,

Assuming diameter of stirrups equal to 6*mm* and main steel #16 *mm*.

Effective cover = 20 + 6 + 16/2 = say 35 *mm*

Effective depth = d = 300 − 35 = 265*mm*.

Width of beam = Thickness of wall = 250 *mm*.

Assumed section of the beam is 250 *mm* x 300 *mm*

3. Load :

Slab load/*m* = 1.5(25 *D* + *LL* + *FF*) x (cantilever span + 1/2 x simply supported span)

= 1.5(25 x 0.10 + 0.75 + 0.8) x (1 + 3/2) = 15.19 *kN/m*

Self weight of beam = 1.5(25 x 0.25 x (0.30 − 0.10) = 1.875 *kN/m*

Total load/*m* = w_u = 15.19 + 1.875 = 17.10 *kN/m*.

4. Design Moment :

$M_u = w_u L^2/2 = 17.1 \times 2.5^2/2 = 53.44\ kN.m$

The width of the flange will be minimum of the following :

(i) $b_f = L_o/6 + 6D_f + b_w$ In this case $L_o = L = 2500\ mm$

$b_f = 2500/6 + 6 \times 100 + 250 = 1266\ mm$

(ii) Available width of flange

= Cantilever projection of slab + 1/2 x spacing of beams.

= 1000 + 1/2 x 3000

= 2500 *mm* > 1266 *mm*

$\therefore b_f = 1266\ mm$

for $x_u = D_f$. $M_{ur1} = 0.36 \times 20 \times 1266 \times 100\,(265 - 0.42 \times 100) \times 10^{-6}$

$= 203\ kN.m >> M_u\ (= 53.44\ kN.m)$

$\therefore x_u < D_f$

5. *Main Steel :*

$$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 53.44 \times 10^6}{20 \times 1266 \times 265^2}}\right] \times 1266 \times 265$$

$= 580\ mm^2$

$A_{st.min} = 0.85 \times bd/f_y = 0.85 \times 250 \times 265/415 = 136\ mm^2 < A_{st}\ (= 580\ mm^2)$

Provide 3 bars of # 16 *mm*, Area provided $= 603\ mm^2 > 580\ mm^2$

Check for the width :

Required width $= (3 \times 16 + 4 \times 25) = 148\ mm < 250\ mm$.

$\therefore$ All bars can be accommodated in one row.

Curtailment of bar :

Out of 3 bars , curtail 1 bars of 16 *mm* at a distance x from free end.

Moment of resistance of 2 bars = $53.44 \times 2/3 = 35.62\ kN.m$

$M_{ux} = w_u\, x^2/2 = 17.10 \times x^2/2 = 35.62\ kN.m$

$\therefore x = 2.04\ m$.

Hence, curtail 1 bars at a distance $= [(2500 - 2040) + 12 \times 16] = 652\ mm + L_d\ (=752mm)$

Curtail 1 bars at a distance of 760 *mm* from the fixed end.

Length of embedment of beam inside the support :

Let x be the length of embedment of the beam inside the wall.

This length of embedment shall be such that static equilibrium should remain when overturning moment is doubled *i.e.* factor of safely to be provided against overturning shall be 2, at working load.*(clause 20.1.1)*

Now, the stabilizing load on the beam from slab of adjoining rooms and walls is given to be equal to 35 *kN/m*.

$\therefore$ Stabilizing moment = $(35 \times x) \times x/2 = 17.5\, x^2$

Overturning moment at working load $= M_u/1.5 = 53.44/1.5 = 35.63\ kN.m$

Now factor of safety against overturning = 2

$\therefore$ Stabilizing moment = 2 x overturning moment

$17.5\, x^2 = 2 \times 35.63$ $\therefore x = 2.02\ m$. say 2.1 m

Note : *In this case it is assumed that it is a two storeyed load bearing structure with slab resting on wall and transferring dead load of 18 kN/m. and the 250 mm thick wall of height 3.4 m giving load of 17 kN/m. Thus total balancing dead load = 35 kN/m.*

6. *Design for shear :*

Maximum shear $= V_{u.max} = w_u.L = 17.1 \times 2.5 = 42.75\ kN$

Area of steel at support $= 3 - \#\,16\ mm = 603\ mm^2$

$p_t\% = 100 \times 603/(250 \times 265) = 0.9\%$, $\tau_{uc} = 0.6\ N/mm^2$ for $M20$ *(Table.4.4.1)*

$V_{uc} = \tau_{uc}\, bd = 0.6 \times 250 \times 265/1000 = 39.75\ kN$

Shear resisted by minimum stirrups ,

$V_{usv.min} = 0.4 \times 250 \times 265/1000 = 26.5\ kN$.

$V_{ur.min} = V_{uc} + V_{usv.min} = 39.75 + 26.5 = 66.25\ kN > V_{u.max}\ (= 42.75\ \text{kN})$

$\therefore$ Minimum shear reinforcement is sufficient.

Using $\phi 6\ mm$ 2 - legged stirrups of grade *Fe* 250

Spacing = 0.87 x 250 x (2 x 28) / (0.4 x 250) = 120 *mm* < = (300 *mm or* 0.75 x 265)

Provide $\phi 6 mm$ 2-legged stirrups at 120 *mm* c/c

7. *Check for deflection :*

The deflection is maximum at the cantilever end where the area of tension steel consists of 2 bars of #16*mm* i.e. A_{st} = 2 x 201 = 402 mm^2

p_t = 100 x 402/(1266 x 265) = 0.12%, Modification factor = α_1 = 1.85 *(Fig 4.4.1)*

Now, b_w / b_f = 230/1266 = 0.18, Reduction factor = α_2 = 0.8 *(Fig.4.4.3)*

Basic *L/d* = 7 , Allowable *L/d* = 7 x α_1 x α_2 = 7 x 1.85 x 0.8

Required *d* = 2500/ (7 x 1.85 x 0.8) = 241 *mm* < 265 *mm* ∴ safe

The details of reinforcement are shown in *Fig. 11.3.3*

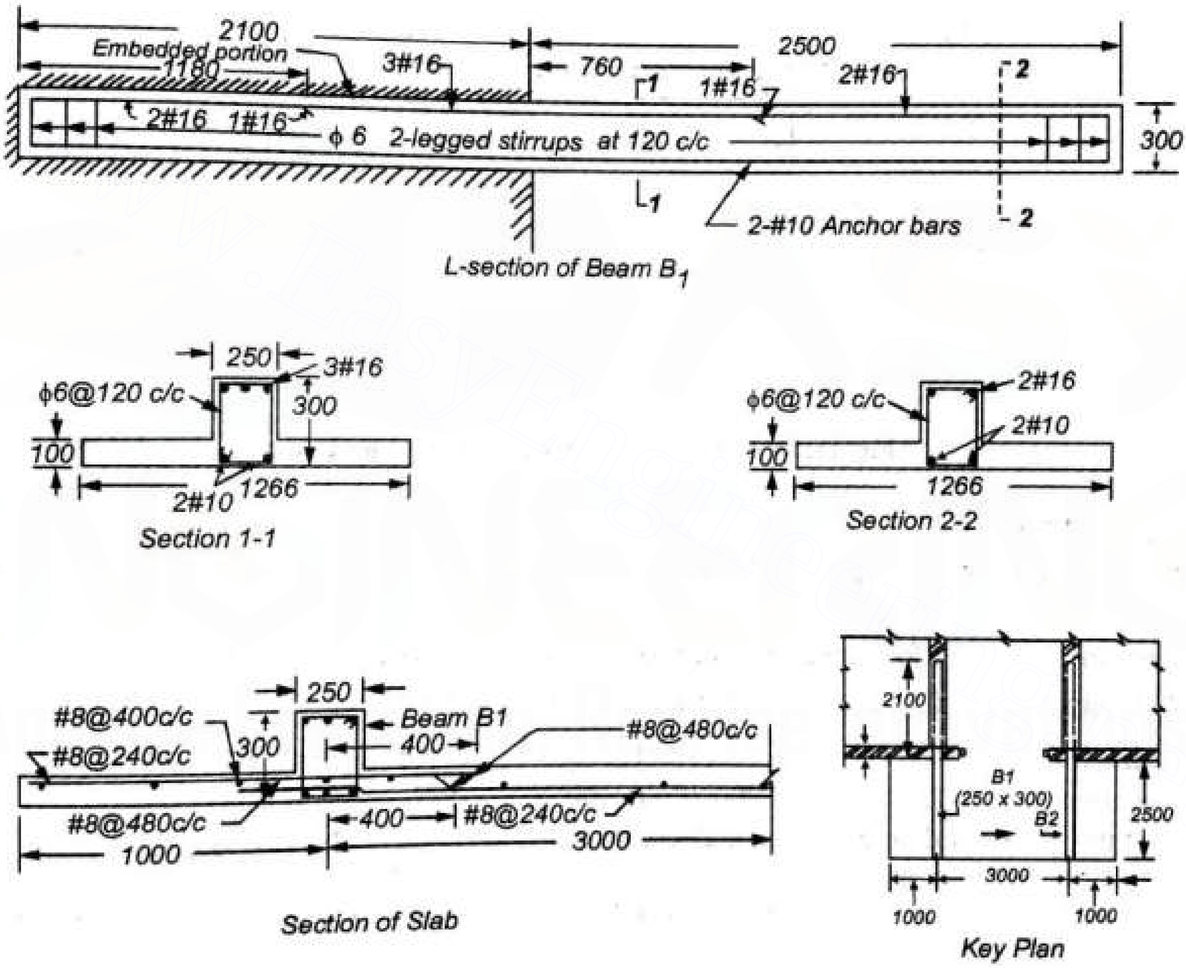


Fig. 11.3.3 Reinforcement Details of Overhanging Porch

11.3.2 Cantilever Porch Supported on Beam

Ex. 11.3.2 In a multistoreyed commercial building a cantilever porch of size 2500 *mm* wide and 5000 *mm* long, provided at the height of 2.2 *m* above the floor level. It overhangs 2500 *mm* beyond the face of the beam. The beam is supported by columns and the construction is monolithic *(See Fig. 11.3.4)*

Assume following data :

Live load = 0.75 kN/m^2 , Load due to finish = 0.8 kN/m^2

Size of column = 350 *mm* x 350 *mm* , Mild environment

Floor to floor height is 3.4*m*. Plinth beams are provided in both orthogonal directions *(See Fig. 11.3.4)*

Concrete grade *M*20 and Mild steel reinforcement is used.

Provide suitable arrangement of beam and slab to satisfy the functional requirements.

Solution :

Since the construction is monolithic, the beam will act as *L*-beam if the slab is provided above the rib of the beam while it will act as a rectangular section if the slab is provided below the rib of the beam. This type of porch is normally provided at the entrance of a building where the height of opening is about 2.2 *m* from the floor level. In such case, if the rib of the beam is provided below the slab the clear head room decreases which mars the functional requirements. Hence, the beam will be provided above the slab and will be designed as a rectangular section.

f_{ck} = 20 N/mm^2 , f_y = 250 N/mm^2 , $LL = 0.75\ kN/m^2$, $FF = 0.8\ kN/m^2$

$k_{u.max}$ = 0.53 , $R_{u.max}$ = 2.97 N/mm^2 (Table 4.1.1)

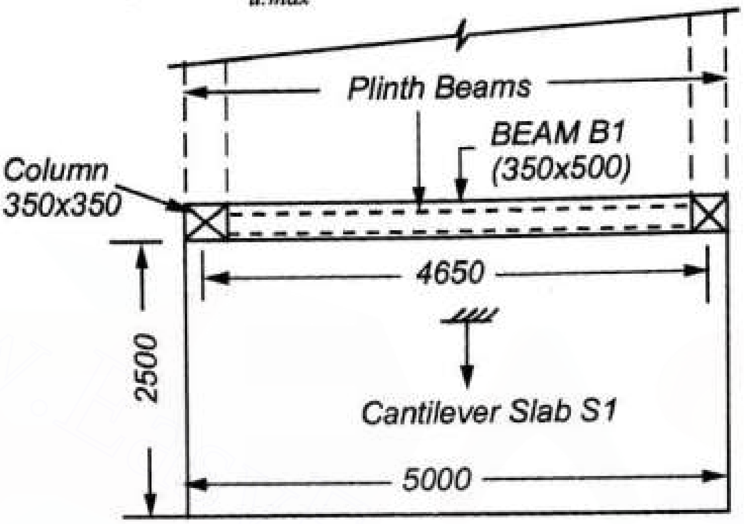


Fig. 11.3.4 Cantilever Porch Supported on Beam

Design of Slab

1. *Span* : Cantilever span $L_1 = 2.5\ m$.

2. *Trial Depth* :

Since mild steel reinforcement is being used it will help in reducing the depth of the slab to some extent.

Assuming $p_t\%$ = 0.4% , Modification factor = $\alpha_1 = 2$

Basic *L/d* ratio = 7 , Allowable *L/d* ratio = α_1 x 7 = 2 x 7 = 14

Required d = $L/14 = 2500/14 = 178.6\ mm$.

Assuming 10 *mm* diameter of bar, effective cover = 15 + 10/2 = 20 *mm*.

Provide total depth of 200 *mm* $\therefore d = 200 - 20$ = 180 *mm*.

Let the overall depth of the slab be reduced to 100 *mm* at the cantilever end where bending moment is zero.

3. *Loads* :

Consider one meter width of the slab.

Self wt. of slab = (0.20 + 0.10)/2 x 25 = 3.75 *kN/m*

Weight due to finish = 0.80 *kN/m*

Live load = 0.75 *kN/m*

Total = 5.30 *kN/m*

Ultimate load/*m* = w_u = 5.3 x 1.5 = 7.95 *kN/m*

Maximum -ve moment at support = $M_u = w_u L_1^2/2 = 7.95 \times 2.5^2/2 = 24.84\ kN.m$

4. *Check depth from B.M. considerations* :

$M_{ur.max}$ = $2.97 \times 1000 \times 180^2 \times 10^{-6}$ = 96.2 *kN.m* >> 24.84 *kN.m* (Table 4.1.1)

$\therefore$ Section is under - reinforced .

6. Area of Steel

$$A_{st} = \frac{0.5 \times 20}{250}\left[1-\sqrt{1-\frac{4.6 \times 24.84 \times 10^6}{20 \times 1000 \times 180^2}}\right] \times 1000 \times 180$$

$= 666\ mm^2$

Using $\phi 10\ mm$ bars, area $= 78.5\ mm^2$

Spacing $= 1000 \times 78.5/666 = 117 mm$ say $110\ mm < (300\ mm$ and or $3d = 540\ mm)$

Provide ϕ 10 mm bars at 110 mm c/c

Area provided $= 1000 \times 78.5/110 = 713\ mm^2$

Curtailment of steel :

It is proposed to curtail half of the steel required at the support. Since the depth of the slab is tapering and B.M. variation parabolic the area of reinforcement will get reduced to half at a distance greater than 1/2 the span from the free end.

Let us check the requirement of steel at distance 1.6 m from the free end.

Effective depth at 1.6 m from free end = 144 mm (=164 – 20)

$(M_u)_{x=1.6} = 7.95 \times 1.6^2/2 = 10.176\ kN.m.$

$$\text{Required } A_{st} = \frac{0.5 \times 20}{250}\left[1-\sqrt{1-\frac{4.6 \times 10.176 \times 10^6}{20 \times 1000 \times 144^2}}\right] \times 1000 \times 144$$

$= 335\ mm^2 < (1/2 \times 713)$ provided at support.

Distance of TPC from support = 2500 – 1600 = 900 mm.

Curtail 50% of the bars at a distance greater of the following :

(a) $900 + 12\phi = 900 + 120 = 1020\ mm$.

(b) $900 + d = 900 + 144 = 1044\ mm$.

Curtail 50% steel at a distance of 1050 mm ($> L_d$) from the face of support.

7. Distribution Steel :

Area required $= 1.5\ D = 1.5 \times (200 + 100)/2 = 225\ mm^2$

Provide $\phi 6\ mm$ @ 120 mm c/c.

8. Check for deflection :

$p_t = 100 \times 666/(1000 \times 180) = 0.37\%, < 0.4\%$ ∴ safe

9. Check for development length :

Required $L_d = [0.87 \times 250 / (4 \times 1.2)]\ \phi = 45.3\phi$ say 46 ϕ

$= 46 \times 10 = 460\ mm$.

(a) The available length of curtail bars from support up to $TPC = 900\ mm > L_d$

(b) The available length of uncurtail bars from TPC to end of cantilever $= 1600\ mm > L_d$

Comments :

(1) Since depth of the slab is governed by deflection criteria it is preferable to use Fe250 steel which has got much higher value of modification factor than HYSD bars. If HYSD bars are to be used distribution steel may be provided at the bottom of the slab which will act as compression steel and reduce the deflection. This would also be helpful in counteracting possible reversal of stress.

(2) The top steel of cantilever slab must be supported by chairs to prevent their bending downward during concreting. In practice, every third bar from main steel is bent back to support the top steel, and chairs provided at the end of cantilever.

(B) Design of Beam

As the slab is provided at the bottom of the beam it will lie in the tension zone with respect to bending of the beam and hence the beam will act as a rectangular section in the mid-span region. Since the beam is subjected to equilibrium torsion it is necessary that the column must provide almost full fixity to the beam to prevent the rotation of the beam due to torsion. This means that the column must be rigid. Hence larger section of column and provision of plinth beams to reduce the effective length of column are provided. The beam will be designed to resist full torsion and bending moment of $w_u L^2/16$ at support and mid-span assuming partial fixity.

1. Span : Assume size of column $= 350\ mm \times 350\ mm$

$L_2 = 5000 - 350 = 4650\ mm = 4.65\ m$

Assume the size of the beam $= 350 mm \times 500 mm$, $d = 500 - 35 = 465 mm.$

2. *Loads :*

Load from slab $= 7.95 \times 2.5 = 19.90\ kN/m$

Self weight of beam $= 1.5 \times 25 \times 0.35 \times 0.5 = 6.56\ kN/m$

Total ultimate load $= w_u = 26.46$ say $26.5\ kN/m$

3. Design moment :

Assuming partial fixity at ends,

$M_u = w_u L^2/16 = 26.5 \times 4.65^2/16 = 35.81\ kN.m$

Torsional moment on beam/m $= w_{u1} \times L_1 \times (L_1/2 + b_w/2)$
$= 7.95 \times 2.5 \times (2.5/2 + 0.35/2)$
$= 28.32\ kN/m$

Torsional moment at support $= T_u = 28.32 \times L_2/2 = 28.32 \times 4.65/2$
$= 65.84\ kN.m$

Explanatory Note : The theoretical proof for assuming design moment of $w_u L^2/16$ is given as under : At the rigid joint between the beam and column the moments will get distributed between them in the ratio of the distribution factors. Thus, if the stiffness of the column and beam are taken into account, the support moments will be distributed between column and beam as under :

Column Stiffness :

Moment of inertia of column $= 1/12 \times 350 \times 350^3 = 1250 \times 10^6\ mm^4$

Length of ground floor column= Height of column above floor level = 2.2m

Rotational bending stiffness of the ground floor column about both orthogonal axes

$K_g = 4E \times 1250 \times 10^6/2200 = 2.27 \times E \times 10^6\ N.mm$

Length of column between opening and floor level $= 3.4 - 2.2 = 1.2\ m$

Stiffness of this upper column $= K_u = 4E \times 1250 \times 10^6/1200 = 4.17 \times E \times 10^6\ N.mm.$

Stiffness of Beam :

Size of beam $= 350mm \times 500mm$, $L = 4650mm$

(a) Torsional stiffness of beam $= k_t = \dfrac{T}{\phi} = \dfrac{\beta D b^3 G}{L}$

$G = E/[2(1+\nu)]$

Taking $\nu = 0.15$, $G = 0.43\ E$, $D/b = 500/350 = 1.428$

β = coefficient for rectangular section11.1 corresponding to $D/b = 1.428$ is ,

$= 0.166 + (0.196 - 0.166) \times 0.228/0.3 = 0.1888$

Torsional stiffness $= K_t = 0.1888 \times 500 \times 350^3 \times (0.43\ E)/4650$

$= 0.374\ E \times 10^6\ N.mm$

(b) Bending stiffness $= K_b = 4E \times (1/12 \times 350 \times 500^3)/4650 = 3.136\ E \times 10^6\ N.mm$

Distribution factor for Beam for torsion $= K_t/(K_t + K_g + K_u) = 0.374/(0.374 + 2.27 + 4.17) = 0.055$

Distributed moment in beam $= 65.84 \times 0.055 = 3.62\ kN.m$

Torsion in beam = 65.84 – 3.62 = 62.22 $kN.m.$ (*i.e.* about 95% of fixed end moment)

Distribution factor for beam for bending = $K_b/(K_b + K_g + K_u)$ = 3.136/(3.136 + 2.27 + 4.17) = 0.327

Fixed end moment = 26.5 x $4.65^2/12$ = 47.75 $kN.m$

Distributed moment in beam = 47.75 x 0.327 = 15.61 $kN.m$

Bending moment in beam = 47.75 – 15.61 = 32.14 $kN.m.$ (= $w_u\,L^2/16$ = 35.81 $kN.m$)

The end section of the beam should be designed for bending moment of 32.14 $kN.m.$ and twisting moment of 62.22 $kN.m$ and mid-span section for bending moment of 39.48 $kN.m$ (= 26.5 x $4.65^2/8$ – 32.14). Thus, the beam can be designed to resist full torsion of 65.84 $kN.m$ and bending moment of $w_u L^2/16$ both at mid-span and at support.

3. Check for Shear :

Since the beam is subjected to heavy torsional moment the section of the beam is required to be checked so that the maximum permissible shear stress does not exceed 2.8 N/mm^2 for $M20$ mix.

Vertical shear = V_u = 26.5 x 4.65/2 = 61.61 kN.

Equivalent shear = V_{ue} = $V_u + 1.6\,T_u/b_w$ *(Eq. 4.5.3)*

= 61.61 + 1.6 x 65.84/0.35 = 362.6 kN.

Shear stress = τ_{ue} = $V_{ue}/(b_w d)$ = 362.6 x 1000/(350 x 465)

= 2.23 N/mm^2 < 2.8 N/mm^2 ∴ safe

4. Check for Depth from B.M. Consideration :

$$M_{ur.max} = R_{u.max}\,bd^2 = 2.97 \times 350 \times 465^2 \times 10^{-6} = 224.7\ kN.m > M_u$$

∴ Section is singly reinforced.

5. Main Steel :

(a) Support Section :

Equivalent bending moment = $M_{ue1} = M_u + M_t$

where, $M_t = T_u(1+D/b_w)/1.7$ = 65.84 x (1 + 500/350)/1.7 = 94.06 $kN.m$

$$M_{ue1} = M_u + M_t \quad \textit{(Eq. 4.5.1)}$$

= 35.81 + 94.06 = 129.87 kN.m

$$\text{Required } A_{st} = \frac{0.5 \times 20}{250}\left[1 - \sqrt{1 - \frac{4.6 \times 129.84 \times 10^6}{20 \times 350 \times 465^2}}\right] \times 350 \times 465$$

= 1445 mm^2

Provide 2-φ12mm + 4-φ20mm, Area provided = 226 + 1256 = 1482mm^2 > 1445 mm^2

As $M_t > M_u$, the longitudinal reinforcement is required on compression face to resist equivalent moment M_{e2} given by : $M_{e2} = M_t - M_u$ = 94.06 – 35.81 = 58.25 $kN.m$ *(Eq. 4.5.2)*

$$A_{sc} = \frac{M_{e2}}{0.87 f_y (d - d_c)} = \frac{58.25 \times 10^6}{0.87 \times 250 \times (465 - 35)}$$

= 623 mm^2

(b) Mid - Span Section :

The twisting moment at mid-span section is zero. Since partial fixity is assumed at support the mid-span section is designed for bending moment of $w_u\,L^2/16$

$$M_u = w_u\,L^2/16 = 26.5 \times 4.65^2/16 = 35.81\ kN.m.$$

$$\text{Required } A_{st} = \frac{0.5 \times 20}{250}\left[1-\sqrt{1-\frac{4.6 \times 35.81 \times 10^6}{20 \times 350 \times 465^2}}\right] \times 350 \times 465$$

$$= 365\ mm^2$$

Provide 2 - ϕ16 mm + 1 - ϕ12mm , Area provided = 628 mm^2

$p_t\%$ = 100 x 628/(350 x 465) = 0.38%,

All the bars will be continued at bottom face so that the required area of 623 mm^2 at support on compression face will be met with.

Side face reinforcement :

As the depth of the member exceeds 450 mm side face reinforcement is required to be provided along the two faces.

Required total area of steel = 0.1 x bD/100 = 0.1 x 350 x 500/100 = 175 mm^2

Provide 1 bar of ϕ12 mm on each face at mid depth giving total area = 226 mm^2 at spacing

(500 – 2 x 25 – 2 x 12)/2 = 213mm which is less than 300mm

Curtailment of Top steel :

At 1.2m. from support :

Twisting moment = 65.84 – 28.32 x 1.2
= 31.86 $kN.m$

Bending moment = 61.61 x 1.2 – 35.81 – 26.5 x 1.2²/2
= 19.04 $kN.m$

Shear force = 61.61 – 26.5 x 1.2
= 29.81 kN

$$M_{ue} = 31.86 \times (1 + 500/350)/1.7 + 19.04 \qquad \textit{(Eq. 4.5.1)}$$

$$= 64.55\ kN.m$$

$$\text{Required } A_{st} = \frac{0.5 \times 20}{250}\left[1-\sqrt{1-\frac{4.6 \times 64.55 \times 10^6}{20 \times 350 \times 465^2}}\right] \times 350 \times 465$$

$$= 673\ mm^2$$

Curtail 2 No.20mm diameter bars. Balance area consisting of (2-ϕ12 + 2-ϕ20) = 851 mm^2

Extend the bars for a distance of effective depth.

Actual point of curtailment = 1.2 + 0.465 = say 1.7m from support.

(c) *Design for Shear*

Maximum equivalent shear, V_{ue} = 362.6 kN.

Area of steel at support = 1482 mm^2

$$p_t\% = \frac{100 \times 1482}{350 \times 465} = 0.91\%\ ,$$

For M20 and $p_t\%$ = 0.91 % , τ_{uc} = 0.6 N/mm^2

V_{uc} = 0.6 x 350 x 465/1000 = 97.65 kN

$V_{usv\,min}$ = 0.4 x 350 x 465/1000 = 65.1 kN

$V_{ur.min}$ = 97.65 + 65.1 = 162.75 kN < V_{ue} (= 362.6 kN)

∴ Design shear reinforcement is required.

Shear resisted by stirrups = V_{us} = $V_{ue} - V_{uc}$ = 362.6 – 97.65 = 265 kN.

Now spacing of stirrups = $s = \dfrac{0.87 f_y A_{sv} d}{V_{use}}$

where, $V_{use} = \left(\dfrac{T_u}{b_1} + \dfrac{V_u}{2.5}\right) \times \dfrac{d}{d_1}$ or $(V_{ue} - V_{uc})$ whichever is greater (Eq. 4.5.5a)

Now $(V_{ue} - V_{uc}) = 265\ kN.$

$b_1 = 350 - 2(25 + 20/2) = 280 mm$, $d_1 = 500 - 2(25 + 20/2) = 430\ mm$, $d = 465 mm$

$\therefore V_{us} = \left(\dfrac{65.84}{0.28} + \dfrac{61.61}{2.5}\right) \times \dfrac{465}{430}$

$= 280.93\ kN > 265\ kN$

$\therefore V_{use} = 280.93\ kN.$

Using 10*mm* 4 - legged stirrups , Area = 4 x 78.5 = 314 mm^2

$s = \dfrac{0.87 \times 250 \times 314 \times 465}{280.93 \times 1000}$

= 113 *mm* say 110 *mm*

Now $s <$ = (0.75*d* or x_1, or $(x_1 + y_1)/4$ or 300*mm*) whichever is less. (Eq.4.5.6)

$x_1 = 350 - 2(25 - 10/2) = 310\ mm$

$y_1 = 500 - 2 \times 25 + 10 = 460\ mm$

$\dfrac{x_1 + y_1}{4} = \dfrac{310 + 460}{4} = 192\ mm$

$0.75d = 0.75 \times 465 = 348\ mm$

least value = 192*mm*

Provide $s = 110mm < 192mm$

Let *x* be the distance from the support where minimum reinforcement is sufficient. Assuming the minimum value of tension steel i.e. $A_{st} = 628\ mm^2$ ($p_t = 0.38\%$) which is provided at mid-span.

for $p_t = 0.38\%$, $\tau_{uc} = 0.43\ N/mm^2$, for M20 mix (Table 4.4.1)

$V_{uc} = 0.43 \times 350 \times 465/1000 = 69.98\ kN$

$V_{usv.min} = 0.4 \times 350 \times 465/1000 = 65.1\ kN$

$V_{ur.min} = 69.98 + 65.1 = 135.08\ kN$

Equating the equivalent shear force $= V_u + 1.6\, T_u/b_w$ at distance *x* from the support to $V_{ur.min}$ we get,

$(61.61 - 26.5x) + 1.6\,(65.84 - 28.32x)/0.35 = 135.08$

$\therefore 156\,x = 227.5$

$\therefore x = 1.46m$

Using φ6 *mm* 2-legged stirrups. (Eq. 4.4.7)

Spacing = 0.87 x 250 x 56/(0.4 x 350) = 90*mm*

Provide φ6*mm* 2-legged minimum stirrups at 90*mm* c/c from 1.46*m* from each support towards mid-span. The reinforcement details are shown in *Fig. 11.3.5*

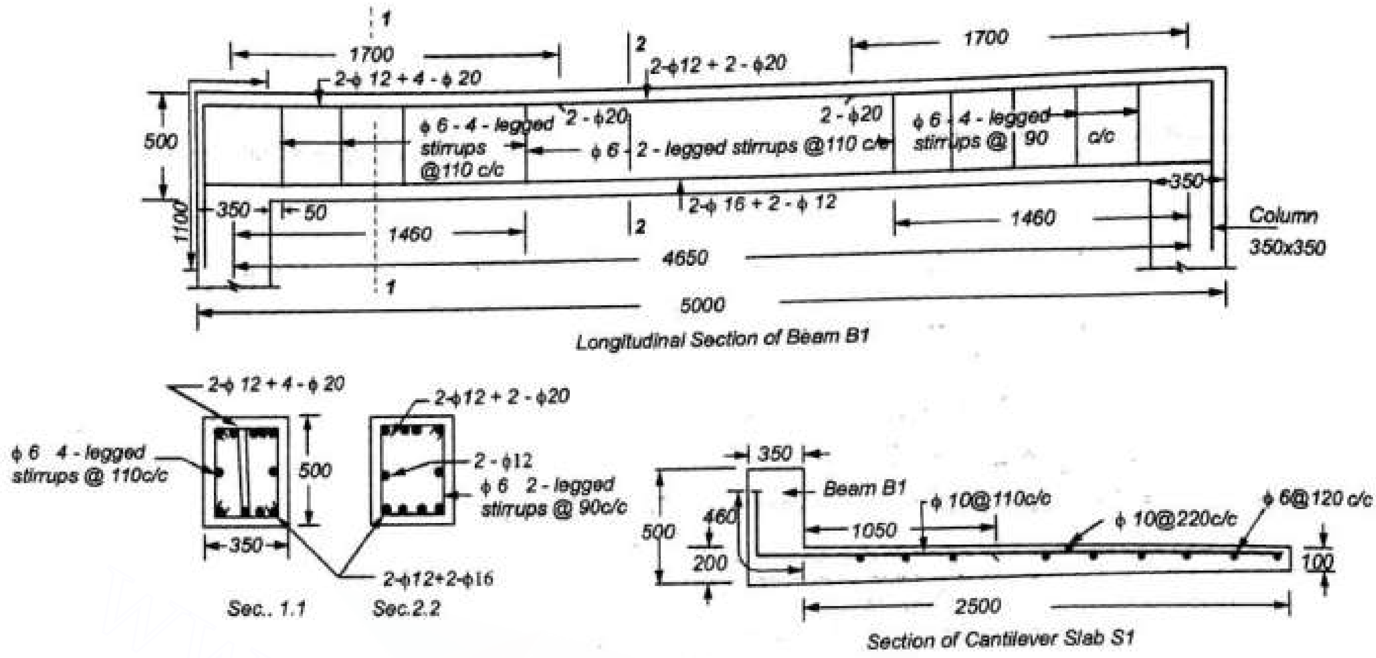


Fig. 11.3.5 Reinforcement Details of Cantilever Porch and Supporting Beam

11.3.6 Porch Slab Simply Supported on Beams

Ex.11.3.3 A porch of size 2500*mm* wide x 5000*mm* long and height 2.2*m*. above floor level is to be provided at the entrance of a three - storeyed commercial building with a facial wall of thickness 80*mm* and height 1*m*. It is be provided at the outer end of porch for exhibiting advertisement. The floor to floor height of building is 3.6*m*. The porch slab is simply supported over the beams *B1* and *B2*. The end beam *B2* is to be supported at the cantilever end of floor beam *B3* which are provided at the level of porch slab as shown in *Fig. 11.3.6*. The thickness of the wall is 200*mm* and size of column is 200*mm* x 400*mm*. Use steel *Fe*415. Assume load on beam *B3* due to wall and floor slab equal to 40 *kN/m* over a length of 4.2*m*. and moderate environment. *LL* on porch slab = 0.75 kN/m^2, Load due to finish = 0.8 kN/m^2

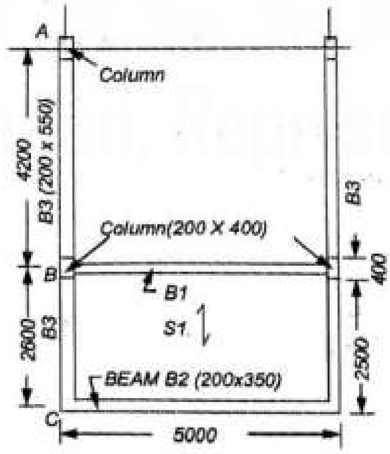


Fig. 11.3.6 Porch Slab Simply Supported on Beams

Given : For moderate environment minimum grade of concrete is M25 and Nominal cover = 30*mm*.

∴ $f_{ck} = 25\ N/mm^2$, $f_y = 415\ N/mm^2$

Floor to floor height = 3.6*m*, Height of porch slab above floor level = 2.2*m*.

Size of porch slab = 2500*mm* x 5000*mm*.

R.C.C. facial wall = 80*mm* x 1000*mm*. high.

LL = 0.75 kN/m^2 , *FF* = 0.8 kN/m^2

Floor + wall load on beam *B3* = 40 *kN/m*

Solution :

(A) Design of slab :

Since the slab is at a lower level than the floor slab it will be designed as simply supported over the beam *B1* and *B2*. The slab is provided at the bottom of the beam to get a plane surface.

1. *Span :* 2500 + 400/2 – 200/2 = 2600 *mm*.
2. *Trial Design :* Assuming p_t = 0.2% , Modification factor = 1.7

$$\text{Required effective depth } d = \frac{2600}{20 \times 1.7} = 77 \text{ mm}$$

Provide total depth of 120 *mm* , $\therefore d = 120 - 30 - 8/2 = 86$ *mm*

3. *Load :* Consider 1 *m* width of slab

Self weight of slab = 25 x 0.12	=	3.00 *kN/m*
Weight due to finish	=	0.80
Live Load	=	0.75
Total working load	=	4.55 *kN/m*
Ultimate load = 1.5 x 4.55	=	6.83 *kN/m*

4. *Design Moment :*

$$M_u = w_u L^2/8 = 6.83 \times 2.6^2/8 = 5.77 \text{ kN.m}$$

5. *Check for depth from B.M. Considerations :*

$$M_{ur.max} = 3.45 \times 1000 \times 86^2 \times 10^{-6} = 25.5 \text{ kN.m} > 5.77 \text{ kN.m} \qquad \text{(Table 4.1.1)}$$

6. *Main Steel :*

$$\text{Required } A_{st} = \frac{0.5 \times 25}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 5.77 \times 10^6}{25 \times 1000 \times 86^2}}\right] \times 1000 \times 86$$

$$= 193 \text{ mm}^2$$

Using # 8*mm* bars

Spacing = 1000 x 50/193 = 259*mm* say 250 *mm* < (3 x 86 *or* 300*mm*)

$\therefore$ Provide #8*mm* bar @ 250*mm* c/c, $(A_{st})_{provided} = 201$ mm^2

7. *Distribution Steel :*

Using 6 *mm* diameter bar of grade Fe250

Required area = 0.15 x 1000 x 120/100 = 180 mm^2

Spacing = 1000 x 28/180 = 155*mm* say 150*mm*

Provide ϕ6 *mm* at 150 *mm c/c*

8. *Check for deflection :*

$(p_t)_{requd}$ = 100 x 193/(1000 x 86) = 0.22% > 0.2% assumed

$\therefore$ Detailed check is carried out

f_s = 0.58 x 415 x 193/201 = 231 N/mm^2

$(p_t)_{prov}$ = 100 x 201/(1000 x 86) = 0.233%

For p_t% = 0.233% and f_s = 231 N/mm^2, α_1 = 1.7 (Fig. 4.7.1)

$(d)_{requd}$ = 2600/(1.7 x 20) = 77*mm* < 0.86*mm* $\therefore$ safe

(B) *Design of Beam B2*

Assume b = 200*mm*. D = 350*mm*

Assuming diameter of stirrup 8*mm* and bar diameter 12*mm*,

d = 350 – 30 – 8 – 12/2 = 350 – 44 = 306 *mm*

1. *Span :* L = 5000 – 200 = 4800 *mm*
2. *Loads :* Slab load/*m* = 1.5 (25*D* + *LL* + *FF*) x 2.6/2

Slab load/m $= 1.5\,(25 \times 0.12 + 0.75 + 0.8) \times 2.6/2 = 8.9\ kN/m$
Weight of facial wall $= (25 \times 0.08 \times 1) \times 1.5 = 3.0\ kN/m$
Self wt. $= 1.5 \times 25 \times 0.20 \times (0.35 - 0.12) = 1.8\ kN/m$
Total ultimate load $= w_u = 13.7\ kN/m$

3. *Design moment* :

$$M_u = w_u L^2/8 = 13.7 \times 4.8^2/8 = 39.5\ kN.m$$

Since the slab is provided at the bottom of the beam it will act as a rectangular section

$$M_{ur.max} = 3.45 \times 200 \times 306^2 \times 10^{-6} = 65\ kN.m > M_u\ (=39.5\ kN/m)$$

$$\text{Required } A_{st} = \frac{0.5 \times 25}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 39.5 \times 10^6}{25 \times 200 \times 306^2}}\right] \times 200 \times 306$$

$$= 402\ mm^2$$

Provide 4 - # 12*mm* bars , Area provided = 452 mm^2

4. *Design for shear* :

$$V_{u.max} = w_u L/2 = 13.7 \times 4.8/2 = 32.9\ kN$$

$p_t = 100 \times 452/(200 \times 306) = 0.73\%$, $\tau_{uc} = 0.563\ N/mm^2$ for $M25$ mix (*Table 4.4.1*)

$$V_{uc} = \tau_{uc}\,bd = 0.563 \times 200 \times 306/1000 = 34.4\ kN$$

$$V_{usv.min} = 0.4 \times 200 \times 306/1000 = 24.5\ kN.$$

$$V_{ur.min} = 34.4 + 24.5 = 58.9\ kN > V_{u.max}$$

$\therefore$ Minimum stirrups are sufficient

Using $\phi 6$ *mm* 2-legged Mild steel stirrups of grade Fe250

$$\text{Spacing } s = \frac{0.87 \times 250 \times (2 \times 28)}{0.4 \times 200}$$

$$= 150mm < (0.75 \times 306 \text{ or } 300mm)$$

Provide $\phi 6mm$ 2-legged stirrups at 150*mm c/c*

(C) *Design of Beam B3*

1. *Type* : Overhanging beam *ABC* 6.8*m* long is simply supported at *A* and *B* over a span 4.2*m* and cantilever span *BC* equal to 2.6 *m*

2. *Span* : $AB = 4.2m$ and $BC = 2.5 - 0.2/2 + 0.4/2 = 2.6m$

3. *Section* : Assumed section of beam 200*mm* x 550*mm* $\therefore d = 550 - 30 - 8 - 16/2 = 504mm$

Since the difference between the level of porch slab and floor slab is 1.4*m* (=3.6 – 2.2) the beam is provided at the level of porch slab. The floor slab will be cast on the masonry constructed over the beam providing bed block of 150*mm*. The beam will act as a rectangular beam.

Comments : There are other alternatives for providing beam *B3*. They are :

(1) The beam B3 may be provided having total depth of 1.4m supporting the floor slab and Porch beams.
(2) The beam B2 of depth 350mm may be supported at the bottom of beam B3, which can have depth of 1050mm (=3.6 – 2.2 – 0.35). In such a case proper supporting arrangements are required to be made. In these cases the beams can be designed as a flanged section.

4. *Loads* : *Span AB* : w_{AB} = Self weight + (weight of floor slab + wall)
$= 25 \times 0.2 \times 0.55 + 40 = 42.75\ kN/m$
Ultimate load $= w_{uAB} = 1.5 \times 42.75 = 64.13\ kN/m$

Span BC : Self weight + reaction from beam *B2* at end *C*
Ultimate load $= 1.5\,[(25 \times 0.2 \times (0.55 - 0.12)] = 3.3\ kN/m$ + Concentrated load of 32.9 *kN* at *C*
$= 3.3\ kN/m$ over length of 2.6 *m* and 32.9 *kN* at *C*

5. *Design Moment* :

M_{uB} = $3.3 \times 2.6^2/2 + 32.9 \times 2.6$ = 96.7 *kN/m*

R_{uA} = $64.13 \times 4.2/2 - 96.7/4.2$ = 111.6 *kN*

$x_{u.max}$ = 111.6/64.13 = 1.74*m* from *A* (Eq. 2.6.1)

The point where +ve BM is maximum is at distance 1.74*m* from *A*

$(M_u)_{max}$ = $111.6 \times 1.74/2 - 0$ = 97.1 *kN.m* at *D*

Point of contraflexure, x_1 = $2\,x_{u.max}$ = 3.48*m* from *A* or 0.72*m* from *B* (See Fig.11.3.7)

The bending moment diagram is shown in *Fig. 11.3.7*

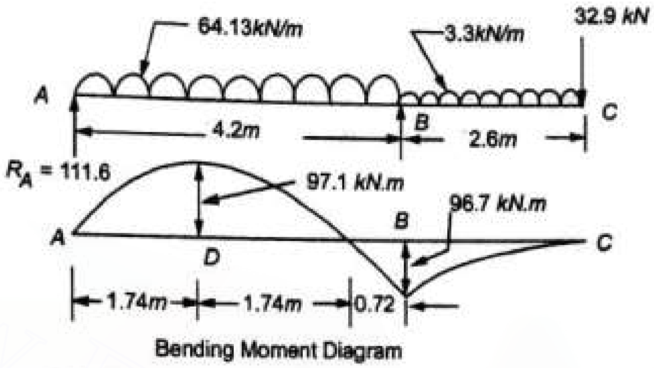


Fig. 11.3.7 Bending Moment Diagram of Beam B3

6. *Check Depth from B.M. Considerations :*

$M_{ur.max}$ = $3.45 \times 200 \times 504^2 \times 10^{-6}$ = 175 *kN.m* > 97.1 *kN.m*

∴ The Section is singly reinforced.

7. *Main Steel :*

(a) Mid-span of beam *AB* :

$$\text{Required } A_{st} \text{ at } D = \frac{0.5 \times 25}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 97.1 \times 10^6}{25 \times 200 \times 504^2}}\right] \times 200 \times 504$$

$$= 591\ mm^2$$

Provide 4-#16 *mm* bars, Area provided = 804 mm^2

$$A_{st} \text{ at } B = \frac{0.5 \times 25}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 96.7 \times 10^6}{25 \times 200 \times 504^2}}\right] \times 200 \times 504$$

$$= 589\ mm^2$$

Provide 2-#12*mm* + 2-#16 *mm* at top of support *B*, Area provided = 628 mm^2

Curtailment of Bars :

Minimum positive moment reinforcement required to be extended at bottom into the support at

- discontinuous end = $A_{st}/3 = 591/3 = 197 mm^2$
- continuous end = $A_{st}/4 = 591/4 = 148\ mm^2$

Now, the requirement of bars from consideration of development length as per *Cl.26.2.3.3* is examined.

At point of contraflexure, the compression reaction does not exist

$$L_d < M_1/V + L_o$$

If two bars of 16*mm* are to be curtailed the M_{ur} of remaining 2-#16*mm* is given by :

$$M_1 = 0.87 \times 415 \times 402 \times 504 \times \left[1 - \frac{415 \times 402}{25 \times 200 \times 504}\right] \times 10^{-6} = 68.3\ kN.m$$

V = SF at point of contraflexure = 111.6 – 64.13 x 3.48 = – 111.6 *kN*
= 111.6 *kN* in magnitude

L_o = 12ϕ *or d* whichever is greater
= 12 x 16 *or* 504*mm* whichever is greater

∴ L_o = 504*mm*

For M25 and Fe415,

$$L_d = \frac{0.87 f_y}{4(\tau_{bd} \times 1.6)} \times \phi = \frac{0.87 \times 415}{4(1.4 \times 1.6)} \times \phi = 40\ \phi = 40 \times 16$$

= 640 *mm*

$$640 < \frac{68.3 \times 1000}{111.6} + 50.4$$

< 111.6 *mm* ∴ safe

2 bars of 16 *mm* can be curtailed. *i.e.* 50% reinforcement will be curtailed.

Moment of resistance at point of cutoff = M_1 = 68.3 *kN.m*

Theoretical point of cutoff *(TPC)* from left support.

Let x_1 be the distance of *TPC* from *A*

$R_A x_1^2 - w_{ux} x^2/2 = 68.3$

∴ $111.6\, x_1 - 64.13\, x_1^2/2 = 68.3$

∴ $x_1 = 0.79m$ *or* $x_2 = 2.68m$

Actual point of cut - off = $x_1 - d$ = 790 – 504 = 286*mm*

Since this is very small there is no advantage is carrying out curtailment on left side.

The other point of curtailment be *Q* on right side

Distance of *Q* from *A* = $x_2 + d$ = 2.68 + 0.504 say 3.2*m* from *A*

Curtail 2-#16*mm* bottom bars at distance of 3.2*m* from support *A or* from 1*m* from support *B* and required 2#16*mm* will be continued into support *B*.

Top bars 2-#16*mm* shall be extended beyond the point of contraflexure for a distance greater of the following (i) *d* (=504*mm*) (ii) 12ϕ (=192*mm*) (iii) clear span/16 (= 4000/16)

Top bars 2-#16 will be provided for a length of (720 + 504) say 1.22*m* from support *B* in span *AB*

Design for Shear

Span AB

(a) Support A

V_{AB}	=	111.6 *kN.* , A_{st} = 4-#16	= 804 mm^2		
p_t	=	100 x 804 /(200 x 504)	= 0.8%,	∴ τ_{uc} = 0.584 N/mm^2	*(Table 4.4.1)*
V_{uc}	=	0.584 x 200 x 504/1000	= 58.87 *kN*		
$V_{usv.min}$	=	0.4 x 200 x 0.504	= 40.32 *kN*		
$V_{ur.min}$	=	58.87 + 40.32	= 99.2 *kN*		
V_{uD}	=	111.6 – 64.13 (200/2 + 504)/1000	= 72.86 *kN* < $V_{ur.min}$ (= 99.2 *kN*)		

∴ Minimum shear reinforcement is sufficient.

Using $\phi 6mm$ 2-legged stirrups

Spacing = 250 x 56/(0.4 x 200) = 175*mm* (< = .75 x 504 or 300*mm*)

(b) Support B (Span BA)

$V_{u.max} = V_{BA} = 64.13 \times 4.2/2 + 96.7 / 4.2 = 157.7\ kN,$

Area of steel at top

$= A_{st} = 2\text{-}\#12 + 2\#16 = 628\ mm^2$

$p_t = 100 \times 628/(200 \times 504) = 0.62\%,$ $\tau_{uc} = 0.528\ N/mm^2$ for *M25*

$V_{uc} = 0.528 \times 200 \times 0.504 = 53.22\ kN,$

$V_{usv.min} = 40.32\ kN.$ as obtained in *(a)*

$V_{ur.min} = 53.22 + 40.32 = 93.5\ kN$

$V_{uD} = 157.7 - 64.13\ (200/2 + 504) / 1000 = 118.9\ kN$

Since $V_{uD} > V_{ur.min}$ design shear reinforced is required.

$V_{us} = V_{uD} - V_{uc} = 118.9 - 53.22 = 65.68\ kN$

Using $\phi 6mm$ 2-legged stirrups ,

Spacing = 0.87 x 250 x (2 x 28) x 504/(65.68 x 1000)

= 93 *mm* say 90*mm*

$L_{s1} = (157.7 - 93.54)/64.13 = 1m$

Cantilever Span BC

$V_{BC} = 32.9 + 3.3 \times 2.6 = 41.48\ kN. < V_{ur.min}\ (=93.44\ kN)$

∴ Minimum Stirrups are sufficient.

Provide $\phi 6mm$ 2-legged stirrups at 90*mm c/c* from support *B* towards *A* for distance of 1*m* and rest provide $\phi 6mm$ 2 legged stirrups at 175*mm c/c*

Curtailment of top bars 2-#16mm

Moment of resistance of 2 - #12*mm* is given by :

$$M_{ur} = 0.87 \times 415 \times 226 \times \left(504 - \frac{415 \times 226}{25 \times 200} \right) \times 10^{-6}$$

$= 39.6\ kN.m$

Let x be the distance from cantilever end where $M_u = 39.6\ kN.m$

$\therefore 32.9x - 3.3\ x^2/2 = 39.6$

$\therefore\ x = 1.29m$

Actual point of curtailment of 2-#16 *mm* bars = 1.29 - 0.504 = 0.78*m* from cantilever end *or* 1.82*m* from *B*.

Check for deflection :

$L/d = 4200/504 = 8.3 < 20$ *or* $L/d = 2600/504 = 5.1 < 7$ ∴ safe

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The reinforcement details are shown in *Fig. 11.3.8*

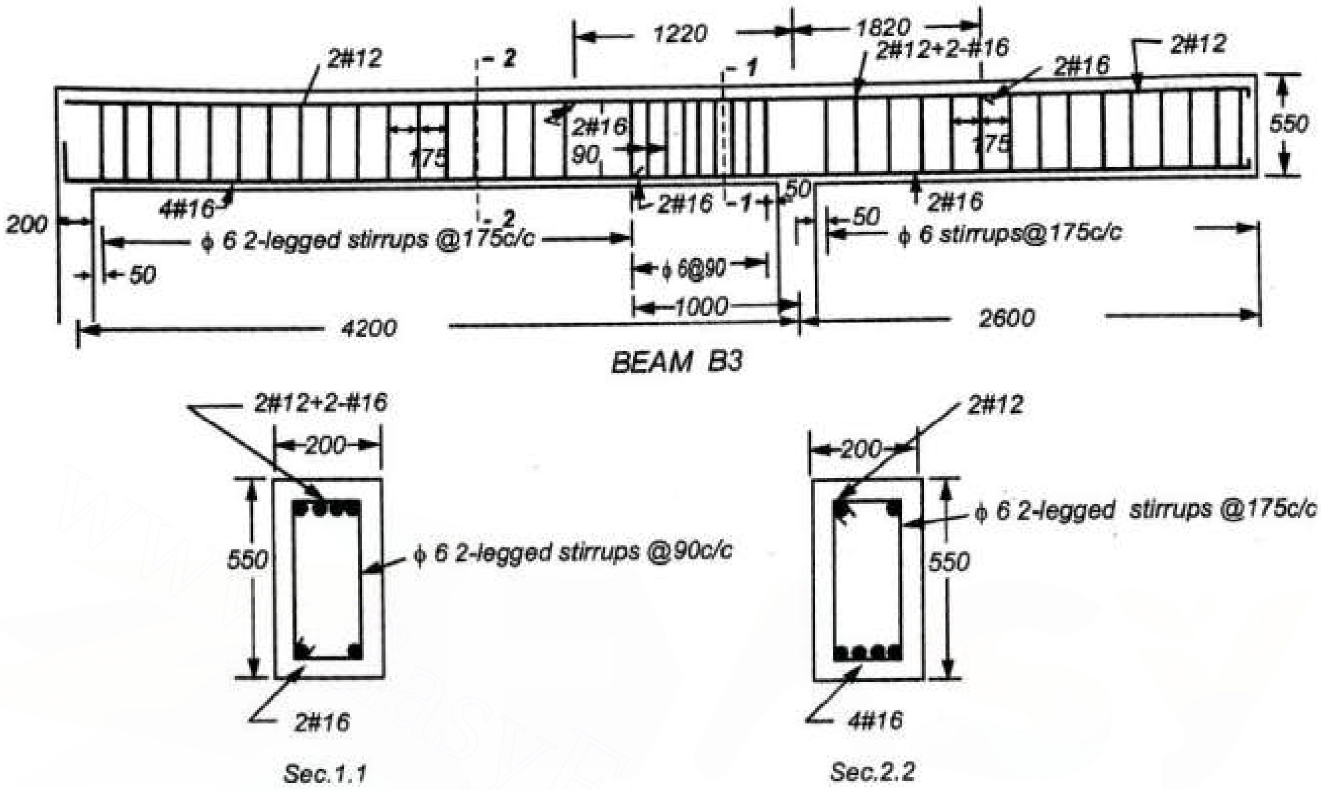


Fig. 11.3.8 Details of Reinforcement of Beam B3

References

11.1 Timoshenko, S.P. and Goodier, J. N., "Theory of Elasticity", Mc Graw - Hill Publication

CHAPTER - 12

COMBINED FOOTING[12.1]

12.1 INTRODUCTION

The footing supporting more than a single column is called a combined footing.

Combined footing becomes necessary when :

(i) an isolated footing overlap due to closeness of columns,
(ii) the exterior column of a building is on or near the boundary line,
(iii) both the columns are on the property line.

There are three alternative methods of design of combined rectangular footings consisting of :

(1) Slab-type footing,
(2) Beam-slab type footing , and
(3) Strap footing[12.2].

The details are restricted[3.1] to some practical problems which normally occur in rural (Gaothan) areas. In one case the property line restricts the extension of the footing on one side while in the other case the building to be constructed lies between the existing structures for which both the footings must lie within the plot limits.

Normally in these cases the rectangular footings are provided wherein the load on the outer column is less than the inner column. However, when external column is more heavily loaded than the internal column trapezoidal footing are preferred. But such cases are rare in practice, hence only rectangular footings are considered.

12.2 ILLUSTRATIVE EXAMPLES

Ex.12.1 Design a Slab-type combined footing connecting the two columns *A and B*,5m*c/c* Column *A* located on the boundary line is of size 400*mm* x 400*mm* and carries a load of 800 *kN*. Column *B* , 500 *mm* x 500 *mm* in size carries a load of 1200 *kN*.

The bearing capacity of soil is 200 kN/m^2. Use concrete M20 and steel Fe415

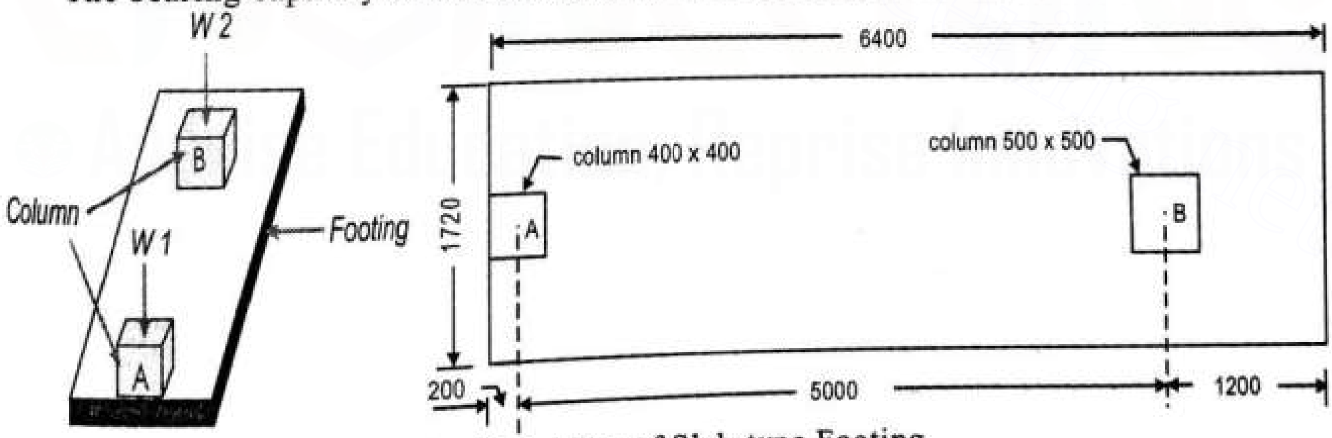


Fig. 12.1 Plan of Slab-type Footing

Solution :

Working load carried by column A = P_A = 800 *kN*

Working load carried by column B = P_B = 1200 *kN*

Proportioning of base size :

Ultimate axial load on column A P_{uA} = 1.5 x 800 = 1200 *kN*

Ultimate axial load on column B P_{uB} = 1.5 x 1200 = 1800 *kN*

Total ultimate load from columns = 1200 + 1800 = 3000 *kN*

Self weight of footing 10 % = 300 *kN*

Total ultimate load = 3300 *kN*

Centre of gravity of the loads from the property line,

$$y = \frac{(1200 \times 0.2 + 1800 \times 5.2)}{(1200 + 1800)} = 3.2\ m$$

Required length of the footing $L_f = 2 \times 3.2 = 6.4m$

Area of footing = 3300 / (1.5 x 200)

= 11 m^2

Width of footing = 11 / 6.4 = 1.718 *m* say 1.72 *m*

Provide footing size of 6.4 *m* x 1.72 *m*

Area of footing provided = 6.4 x 1.72 = 11.01m^2

Ultimate upward soil intensity of pressure = (3000 / 11.01) = 272.5 kN/m^2

Net upward soil pressure w_u = 272.5 x 1.72= 468.7 *kN/m* say 469 *kN/m*

Shear force and Bending moment (Fig. 12.2) :

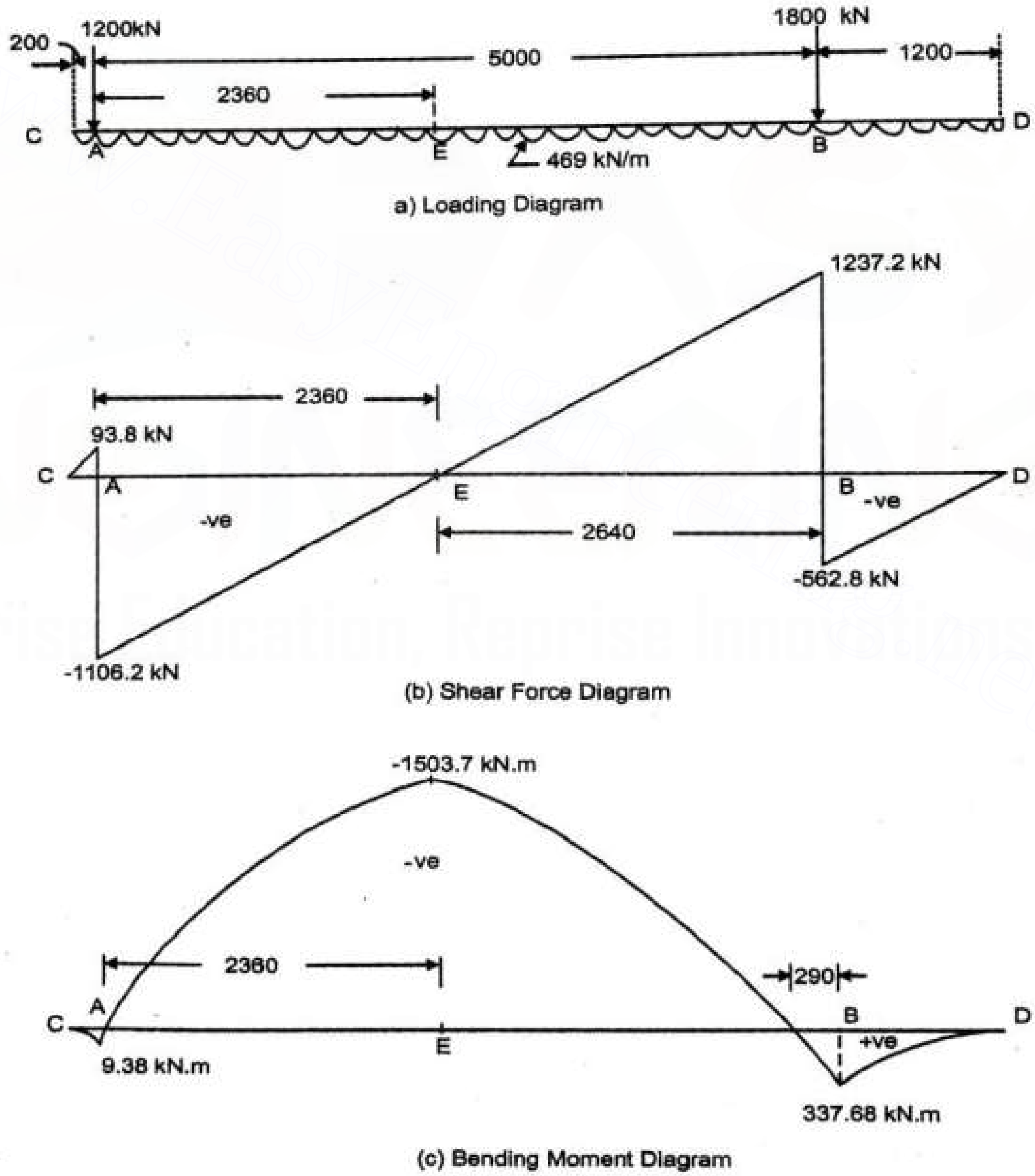


Fig. 12.2 Bending Moment and Shear force Diagram

Shear force, $V_{CA} = 469 \times 0.2 = 93.8\ kN$,
$V_A = 93.8 - 1200 = -1106.2\ kN$
$V_{DB} = 1.2 \times 469 = -562.8\ kN$,
$V_B = 1800 - 562.8 = 1237.2\ kN$. Variation between AB is linear.

Let x be the point E of zero shear from 1200 kN load at which $B.M.$ is maximum,
$x = (1106.2 \times 5) / (1237.2 + 1106.2) = 2.36\ m$
Bending moment under column A, $M_{uA} = 469 \times 0.2 \times 0.2/2 = 9.38\ kN.m$
Maximum bending moment occurs at $E = x = 2.36\ m$ where SF is zero,
$M_{uE} = 469 \times 2.38^2/2 - 1200 \times 2.36 = 1503.7\ kN.m$
$M_{uB} = 469 \times 1.2^2/2 = 337.68\ kN.m$
Required effective depth = $\sqrt{(1503.7 \times 10^6) / (2.76 \times 1720)}$
$= 562.8\ mm$
Provide total depth = 630 mm
Effective depth provided $d = 630 - 60 = 570 mm$

(ii) Check the depth for Two - way Shear :
The two– way shear is critical under column B which is carrying larger axial load of 1800 kN.
Critical section is at a distance $d/2$ from the face of the column.

(i) Column – B

Shear stress resisted by concrete = $\tau_{uc2} = \tau'_{uc} \times k_s$
where, $\tau'_{uc} = 0.25\sqrt{f_{ck}} = 0.25\sqrt{20} = 1.118\ N/mm^2$
$k_s = 0.5 + 500/500 = 1.5 > 1 \quad \therefore k_s = 1$
$\tau_{uc2} = 1.118 \times 1 = 1.118\ N/mm^2$
Width at the critical section $= b + 2 \times d/2 = b + d = 500 + 570 = 1070\ mm$
Perimeter at the critical section = $2 \times (1070 + 1070) = 4280\ mm$
Area resisting shear = $4280 \times 570 = 2439600\ mm^2$
Design shear V_{uD2} = Column load – upward intensity of soil pressure × area at critical section
$= 1800 - 469 \times 1.07 \times 1.07$
$= 1263.04\ kN$
Actual shear stress = $1263.04 \times 10^3 / 2439600 = 0.52\ N/mm^2 < \tau_{uc2}\ (= 1.118\ N/mm^2)$ $\therefore$ safe

Area of reinforcement between AB at top,
(i) Area of reinforcement for maximum hogging moment M_{uE} (= 1503.7 $kN.m.$)

$$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 1503.7 \times 10^6}{20 \times 1720 \times 570^2}}\right] \times 1720 \times 570$$

$= 9040\ mm^2$

Using 32 mm bars, No. of bar =9040/804.2= 11.2 say 12 Nos.
Area provided = $9651\ mm^2$
Provide 12 bars of # 32mm diameter distributed over a width of 1.72m.
Curtail 8 #12 bars at the end of column B and remaining 4 bars continued up to the end.

Bending moment at the face of the column = $469 \times 0.95^2/2 = 211.6$ $kN.m$

Area of steel for BM under column B at bottom,

$$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 211.6 \times 10^6}{20 \times 1720 \times 570^2}}\right] \times 1720 \times 570$$

$$= 1052\ mm^2$$

Provide 4 - #20 at bottom , Area provided = 1256 mm^2

Since the moment under column A is only 9.38 kN.m ,the required area of steel is negligible.

Shear reinforcement :

Shear reinforcement between AB

Point of contraflexure at a distance x from B,

$$M_x = 1800x - 469\,(x + 1.2)^2/2 = 0$$

$$x = 0.288 \text{ say } 0.29\ m$$

SF at point of contraflexure $V_{uD} = 1237.2 - 469 \times 0.29 = 1101.2\ kN$

$p_t\% = 100 \times 9651 / (1720 \times 570) = 0.98\%$, $\tau_{uc} = 0.61\ N/mm^2$

$V_{uc} = 0.61 \times 1720 \times 570 / 1000 = 598\ kN$

$V_{usv.min} = 0.4 \times 1720 \times 570 / 1000 = 392\ kN$

$V_{ur\ min} = 598 + 392 = 990\ kN < 1101.2\ kN$

∴ Design stirrups are required.

$V_{us} = 1101.2 - 598 = 503.2\ kN$

Using #12mm 2-legged stirrups at :

Spacing $s = (0.87 \times 415 \times 226) \times 570 / (503.2 \times 1000) = 92.4\ mm$ say 90 mm

Zone of design shear reinforcement= (1237.2 - 990) /469 = 0.53 m

Provide #12 mm 2-legged stirrups at 90 mm c/c for 530 mm length from support B towards A

Shear at a distance d from the face of support A

$= 1106.2 - 469\,(0.02 + 0.570)$

$= 830\ kN < 990\ kN$

only minimum shear reinforcement is required.

Minimum shear reinforcement using #12 mm bars , $s = 0.87 \times 415 \times 226 / (0.4 \times 1720) = 110\ mm$

Transverse Reinforcement :

The footing bends in the transverse direction in the vicinity of the column. The transverse reinforcement under each column should be provided with effective width equal to the width of the column plus twice the effective depth.

∴ Effective width = $b + 2d$

For column B, band width = $500 + 2 \times 570 = 1640\ mm$

Factored upward pressure for column $B = P_{uB} / B_f = 1800 / 1.72 = 1046.5\ kN/m$.

Cantilever projection = (1720 - 500) / 2 = 610 mm.

B.M. at the face of column = $1046.5 \times 0.61^2 / 2 = 194.7\ kN.m$

Effective depth = 570 -20/2-12/2 = 554 mm

$$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 194.7 \times 10^6}{20 \times 1640 \times 554^2}}\right] \times 1640 \times 554$$

$$= 997\ mm^2$$

Provide 10 Nos. of # 12 mm in a width of 1.64 m , Area provided = 1131 mm^2

For column A, width = 400 + 2 x 570 = 1540 *mm*

Factored upward pressure for column *A* $= P_{uA} / B_f = w_u = 1200 / 1.72 = 698$ *kN/m.*

Cantilever projection = (1720 - 400) / 2 = 660 *mm.*

B.M. at the face of column = 698 x 0.66² / 2 = 152 *kN.m*

Effective depth = 554 *mm*

$$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 152 \times 10^6}{20 \times 1540 \times 554^2}}\right] \times 1540 \times 554$$

$$= 760\ mm^2$$

Provide 8 Nos. of # 12 *mm* in a width of 1.54 *m* , Area provided = 904 mm^2

Minimum transverse reinforcement in a length of 3.22 m

$$= 0.12 \times 1000 \times 533 = 639.6\ mm^2$$

Provide #12 *mm* at spacing $s = 1000 \times 113 / 639.6 =$ say 180 *mm*

The details of reinforcement are shown in *Fig. 12.3*

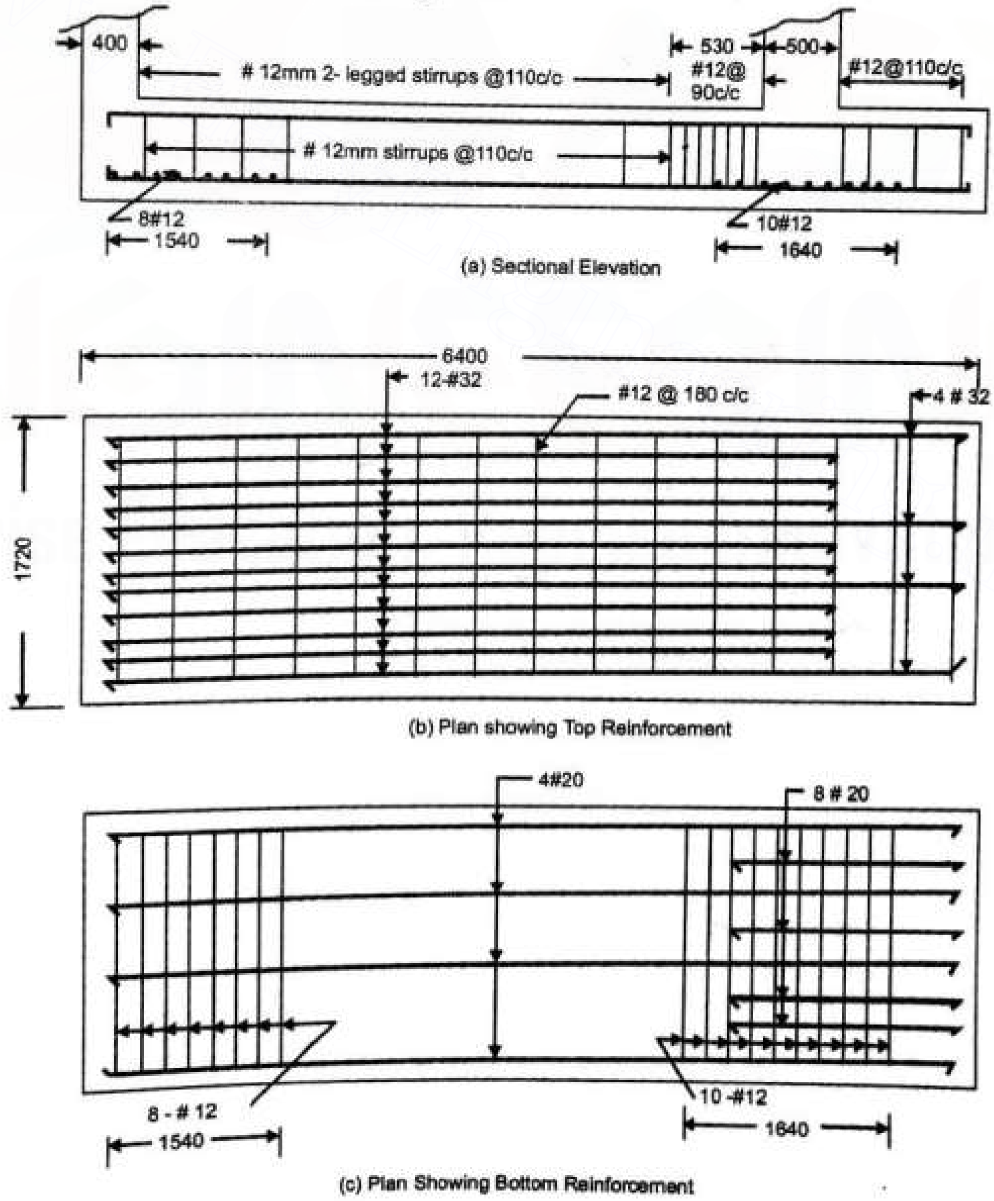


Fig. 12.3 Details of Reinforcement for Slab-type Footing

Ex. 12.2 Design a Beam-slab type combined footing connecting two columns *A and B*, 5m c/c. Column *A* located on the boundary line is of size 400*mm* x 400*mm* and carries a load of 800 *kN*. Column *B*, 500 *mm* x 500 *mm* in size carries a load of 1200 *kN*. (*Fig. 12.4*)

The bearing capacity of soil is 200 kN/m^2. Use concrete M20 and steel Fe415

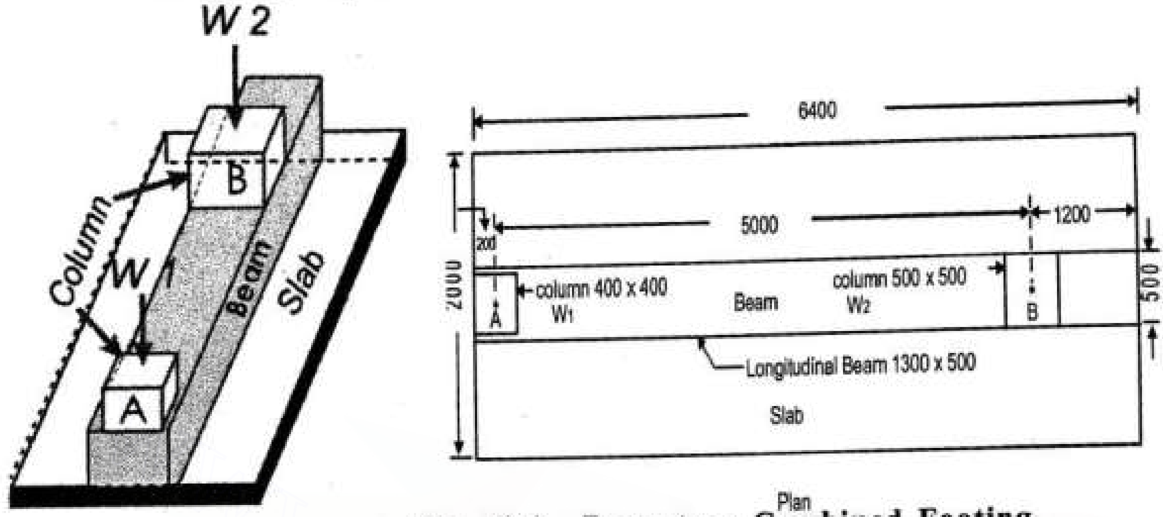


Fig. 12.4 Beam-type Combined Footing

Solution :

Working load carried by column $A = P_A = 800\ kN$

Working load carried by column $B = P_B = 1200\ kN$

(a) Proportioning of base size :

Ultimate axial load on column A $P_{uA} = 1.5 \times 800 = 1200\ kN$

Ultimate axial load on column B $P_{uB} = 1.5 \times 1200 = 1800\ kN$

Centre of gravity of the loads from the property line,

$$y = \frac{(1200 \times 0.2 + 1800 \times 5.2)}{(1200 + 1800)} = 3.2\ m$$

Required length of the footing $L_f = 2 \times 3.2 = 6.4 m$

Area of footing $= 3300 / (1.5 \times 200) = 11\ m^2$

Required width of footing $= 11 / 6.4 = 1.72\ m$

Provide footing size of 6.4 *m* x 2 *m*

Area of footing provided $= 6.4 \times 2 = 12.8\ m^2 > 11\ m^2$

Ultimate upward soil intesity of pressure,

$$w_u = (3000 / 12.8) = 234.4\ kN/m^2$$

Design of slab :

Intensity of upward pressure $= w_u = 234.4\ kN/m^2$

Consider one meter width of the slab ($b = 1\ m$)

Load per *m* run of slab $= 234.4 \times 1 = 234.4\ kN/m$

Cantilever projection of the slab $= 1000 - 250 = 750\ mm$

Maximum ultimate moment $= 234.4 \times 0.75^2/2 = 66\ kN.m.$

For M20 and Fe 415, $R_{u.max} = 2.76\ N/mm^2$ and $k_{u.max} = 0.48$ ... ... (*Table 4.5.1*)

Required effective depth $= \sqrt{66 \times 10^6 / (2.76 \times 1000)} = 155\ mm$

Since the slab is in contact with the soil cover of 50 *mm* is assumed.

Provide total depth of 220 *mm*

Using 20 *mm* diameter bars

Effective depth = 220 - 50 - 20/2 = 160 *mm*

Required, $$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 66 \times 10^6}{20 \times 1000 \times 160^2}}\right] \times 1000 \times 160$$

$= 1395\ mm^2$

Use #20 *mm* diameter bar at 110 *mm*

Area provided = 1000×314.16 / 110 = 2856 mm^2

Larger depth and more area of steel are provided to make it safe against diagonal tension.

Check the depth from one-way shear considerations :

p_t = 100 × 2856 / (1000 × 160) = 1.75 % , τ_{uc} = 0.76 N/mm^2 ... ... (*Table 5.71*)

Value of k for 200 *mm* thick slab = 1.16 ... ... (*Table 9.4.2*)

Permissible shear stress = 1.16 × 0.76 = 0.877 N/mm^2

Shear resisted by concrete V_{uc} = 0.877 x 160 = 140.3 *kN*

V_{uD} = 234.4 × (0.75 – 0.16) = 138.3 *kN* , $V_{uc} = V_{uD}$ (= 138.3 *kN*) ∴ safe

Check for development length

L_d = [0.87 × 415 / (4×1.2 x1.6)]ϕ = 47 ϕ = 47 × 20 = 940 *mm* ... (*Table 6.2.2*)

Modified development length = $L_{dm} = L_d \times (A_s)_{prov} / (A_s)_{reqd}$

= 940 × 2856 / 1393 = 458 *mm*

Available length of bar = 1000 – 250 - 50 + 90° hook allowance (= 8ϕ)

= 1000 – 250 - 50 + 8 × 20

= 860 *mm* > 458 *mm* ∴ safe

Transverse reinforcement

Using 8 *mm* bars , required area = 0.12 x 1000 x 220 / 100 = 264 mm^2

Provide #8 *mm* at 180 c/c , Area provided = 279 mm^2

Design of Longitudinal Beam

Two columns are joined by means of a beam monolithic with the footing slab.

As the width of the footing is 2 *m* , the net upward soil pressure per meter length of the beam

= w_u = 234.4 × 2 = 468.8 *kN/m*

Shear Force and Bending Moment (Fig. 12.5)

V_{CA} = 468.8 × 0.2 = 93.6 *kN* , V_A = 93.7 -1200 = -1106.3 *kN*

V_B = 1800 - 562.6 = 1237.4 *kN* , V_{DB} = 468.8 x 1.2 = -562.6 *kN* ,

Point of zero shear from A,

x = 1106.3 x 5 / (1106.3 + 1237.4) = 2.36 *m* from column A or at 2.38 *m* from end C

Maximum B.M. occurs at E at a distance of 2.38 *m* from *end C*

M_{uE} = 468.8 x 2.38²/2 - 1200 x 2.36 = 1504 *kN.m*

Bending moment under column A = M_{uA} = 468.8 x 0.2² / 2 = 9.38 *kN.m*

Bending moment under column B = M_{uB} = 468.8 x 1.2²/2 = 337.54 *kN.m.*

Let the point of contraflexure be at a distance x from support B

Then , $M_x = 1800x - 468.8\,(x + 1.2)^2 / 2 = 0$

$\therefore \quad x = 0.29\ m$ from B. or $4.71\ m$ from column A

The Bending Moment and Shear force Diagrams are shown in *Fig. 12.5*

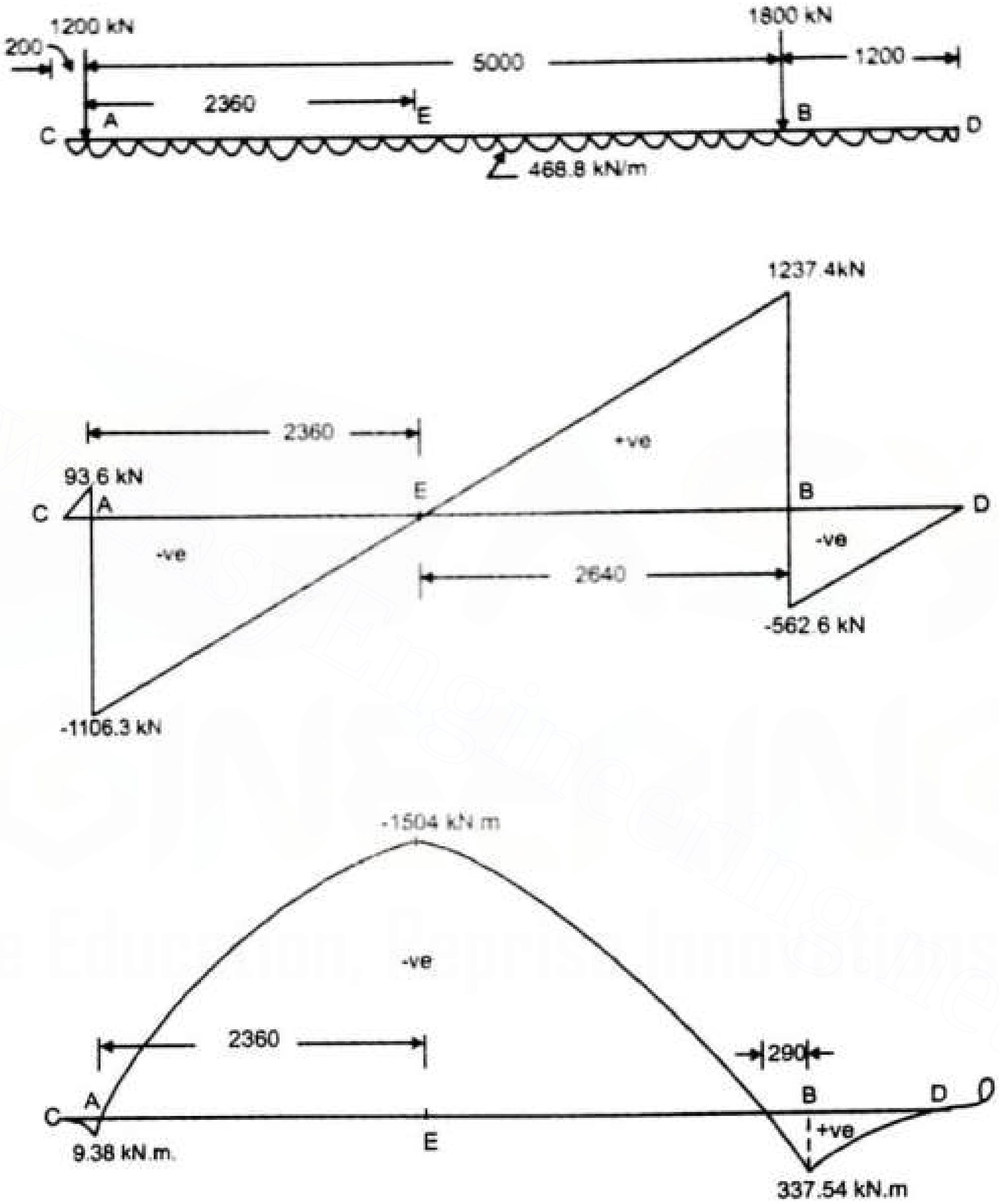


Fig. 12.5 Bending Moment and Shear Force Diagram

Since the negative moment (producing tension at top) is greater than the sagging moment and further the region of negative moment is larger than the positive (*i.e.* sagging) moment, the beam will be provided above the slab. In such a case the footing slab will lie in the compression zone with respect to the bending of the beam and will act as an inverted T – beam between the points of contraflexures. In the cantilever portion tension will be at the bottom and the beam will be designed as rectangular section.

(i) Depth of beam from B.M. Considerations :

The width of beam is kept equal to the maximum width of the column i.e. 500 *mm*. Even though the beam acts as a T-section the depth required for shear in foundation beam is usually more than that required for flexure. Therefore, the trial depth is assumed treating the section as rectangular.

$$d = \sqrt{1504 \times 10^6 / (2.76 \times 500)} = 1044 \ mm$$

Provide total depth of 1200 *mm*. Assuming two rows of bars effective depth is given by :

Effective depth provided $d = 1200 - 50 - 25 - 25/2 = 1112.5 \ mm$

(ii) Check the depth for Two - way Shear :

The column *B* can punch through the footing only if it shears against the depth of the beam along its two opposite edges, and along the depth of the slab on the remaining two edges. The critical section for two - way shear is taken at distance $d/2$ (*i.e.* 1112.5 / 2 *mm*) from the face of the column on the side of beam and at a distance half the effective depth of the slab ($d_s/2$) on the other side as shown in *Fig. 12.6*

In this case $b = D = 500 \ mm$, $d_b = 1112.5 \ mm$, $d_s = 160 \ mm$

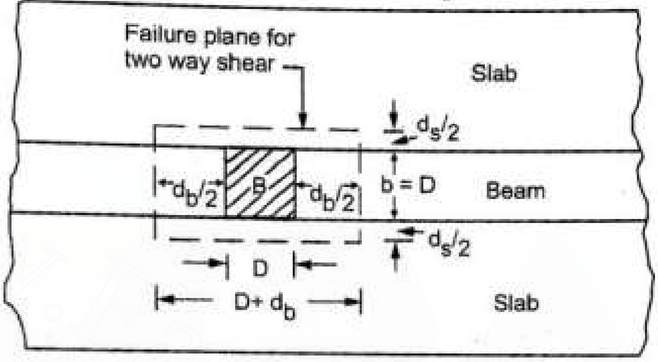


Fig. 12.6 Critical Section for Two-way shear

Area resisting two – way shear

$$= 2 (D \times d_b + d_s \times d_s) + 2 (D + d_b) d_s$$
$$= 2 (500 \times 1112.5 + 160 \times 160) + 2 (500 + 1112.5) 160$$
$$= 1679700 \ mm^2$$

Shear stress resisted by concrete $= \tau_{uc2} = \tau'_{uc} \times k_s$

where, $\tau'_{uc} = 0.25\sqrt{f_{ck}} = 0.25 \sqrt{20} = 1.118 \ N/mm^2$

$k_s = 0.5 + d/D = 0.5 + 500/500 = 1.5 > 1 \quad \therefore k_s = 1$

$\therefore \quad \tau_{uc2} = 1 \times 1.118 = 1.118 \ N/mm^2$

Shear resisted by concrete

$V_{uc2} = 1.118 \times 1679700 / 1000 = 1877.9 \ kN.$

Design shear $= V_{uD2}$ = column load – $w_u \times$ area at critical section

$= [1800 - w_u (b + d_s) \times (D + d_b)$

$= 1800 - [468.8 \times (500 + 160) \times (500 + 1112.5) \times 10^{-6}]$

$= 1301 \ kN < V_{uc2}$ (= 1877.95 *kN*) $\therefore$ Safe

Distance of cantilever from the face of column *B*

= 1200 - 500/2

= 950 *mm*

Bending moment at the face of column

$= 468.8 \times 0.95^2 / 2$

$= 211.5 \ kN.m$

The beam acts as *T*-beam between the points of contraflexures.

$$b_f = \frac{0.7\,L_o}{(0.7L_o/b+4)} + b_w \quad \frac{0.7 \times 5000}{(0.7 \times 5000/2000 + 4)} + 500$$

$$= 1110\ mm$$

$$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 1504 \times 10^6}{20 \times 1110 \times 1112.5^2}}\right] \times 1110 \times 1112.5$$

$$= 4018\ mm^2$$

Provide 9 - #25 *mm* on top face, Area provided = 4418 mm^2

Area of reinforcement in cantilever portion *BD*

$$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 211.5 \times 10^6}{20 \times 500 \times 1112.5^2}}\right] \times 500 \times 1112.5$$

$$= 537.6\ mm^2$$

Provide 3-#12 *mm* + 1- #16 *mm* , Area provided = 540 mm^2

Bending moment in cantilever portion *AC* is very small. The same steel is continued at bottom face.

Design of shear reinforcement :

Portion between column *i.e. AB*

In this case the crack due to diagonal tension will occur at the point of contraflexure because the distance of the point of contraflexure from the column is less than the effective depth $d\,(= 1112.5\ mm)$

(i) Maximum shear force at B = $V_{u.max}$ = 1237.4 *kN*

SF at point of contraflexure = 1237.5 - 468.8 x 0.29 = 1101.5 *kN*

Area of steel available 9- #25 *mm* , A_{st} = 4418 mm^2

p_t = 100 × 4418 / (500 × 1112.5) = 0.794%

τ_{uc} = 0.56 + 0.06 × 0.04 / 0.25 = 0.57 N/mm^2 ... ... (*Table 5.7.1*)

Shear resisted by concrete = $V_{uc} = \tau_{uc} b_d$ = 0.57 × 500 × 1112.5 / 1000 = 317 *kN*.

Shear resisted by minimum stirrups $V_{usv.min}$ = 0.4 *bd* = 0.4 × 500 × 1112.5/1000 = 222.5 *kN*

$V_{ur.min}$ = 317 + 222.5 = 539.5 *kN* < V_{uD} (= 1101.5 *kN*) Shear reinforcement is required

Shear to be resisted by stirrups = $V_{usv} = V_{uD} - V_{uc}$ = 1101.5 - 317 = 784.5 *kN*

Using 12 *mm* diameter 2 – legged stirrups,

Spacing *s* = 0.87 × 415 × (2 × 113) × 1112.5 / (784.5 × 1000) = 115 *mm say* 110 *mm*

Zone of shear reinforcement

$L_{s1} = (V_{u.max} - V_{ur.min}) / w_u$ = (1237.4 - 539.5) / 468.8 = 1.5 *m*

= 1.5 *m* from support *B* towards *A*

Provide #12 mm 2-legged stirrups at 110 *mm* c/c for a distance of 1.5 *m* from face of support towards *A*

Maximum shear force at a distance of effective depth from the face of support *A is:*

= 1106.3 - 468.8(0.2 + 1.1125)

= 491 *kN* < 539.5 *kN* Only minimum shear reinforcement is required.

For the remaining portions provide minimum shear reinforcement using 8 *mm* diameter bars 2 – legged stirrups at

Spacing , $s = 0.87 \times 415 \times 100 / (0.4 \times 500) = 180\ mm$ c/c. < 300 *mm*

Provide #8 *mm* 2 - legged stirrups at 180 *mm* in the remaining portions

Side face reinforcement = 0.1 x 500 x 1200 / 100 = 600 mm

Provide 6 - #12 *mm* , Area provided = 678 mm^2

The details of reinforcement are shown in Fig.12.7

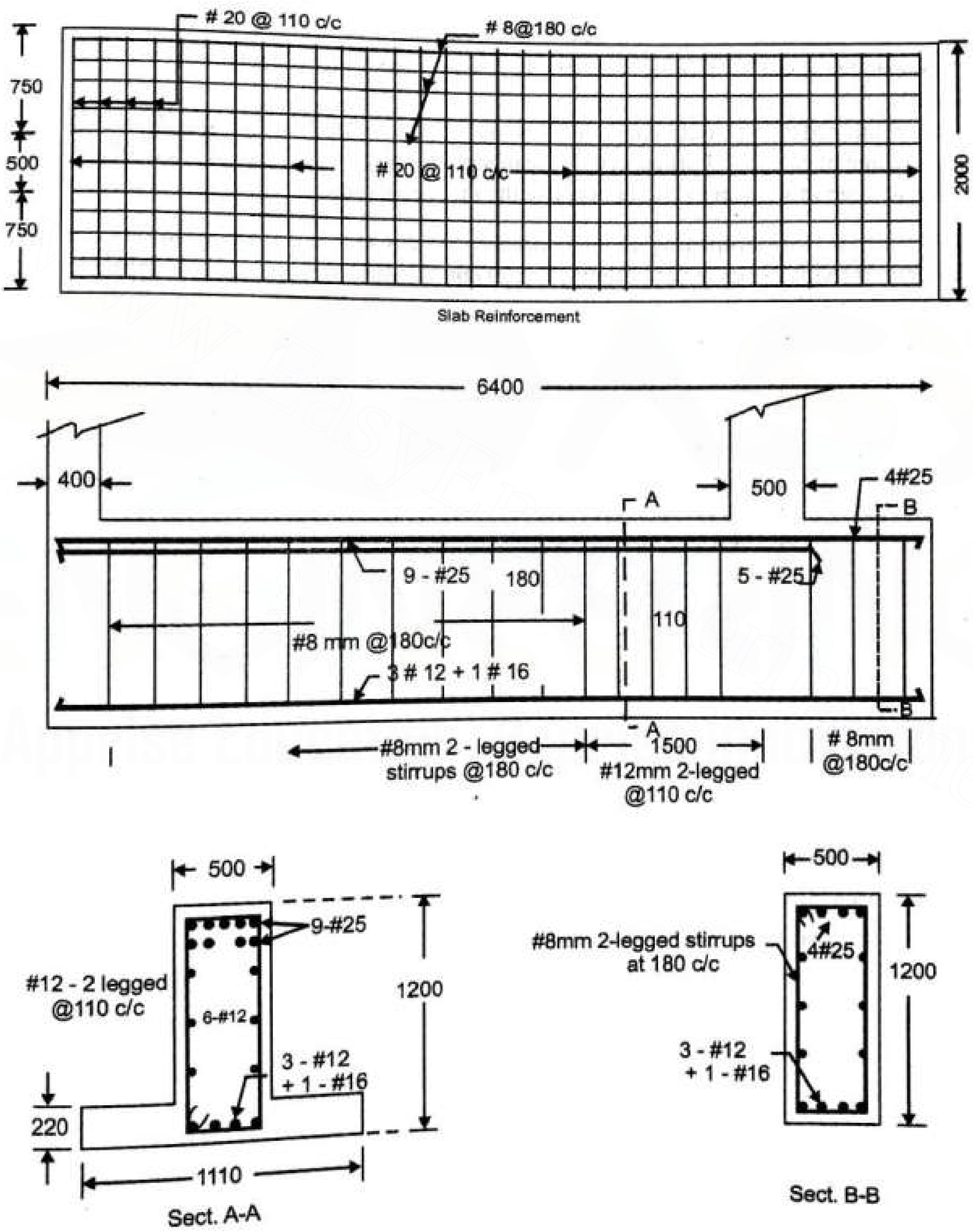


Fig.12.7 Details of Reinforcement for Beam-slab type Footing

The Data for *Ex.13.1, 13.2 (and Ex. 12.9.3* contained in book of " Limit State Theory and Design of R.C. , Structures Publications , Pune 411009 " have been kept the same for comparison as given below:

Case. No	Footing	Concrete in m^3	Steel in kg
1	Slab-type	6.93	665
2	Beam-slab type	5.94	609
3	Strap	5.32	512

It will be seen that slab-type footing is costly than beam-type footing . But strap type footing is very economical.
It may be mentioned that the outcome of this results is not deterministic because it depends on the centre to centre spacing of columns. Further in the case of strap footing the length and width of slab can be varied and different alternatives can be tried to choose the most economical one.

Ex.13.3 In a two storey building two columns *A and B* are boundary columns 5*m* out to out carrying equal loads. The size of columns is 250 *mm* x 250 *mm*, and each is carrying load of 500 *kN*. Design the beam-slab type combined footing. (*Fig. 12.8*)
The bearing capacity of soil is 150 kN/m^2. Use concrete M20 and steel Fe415

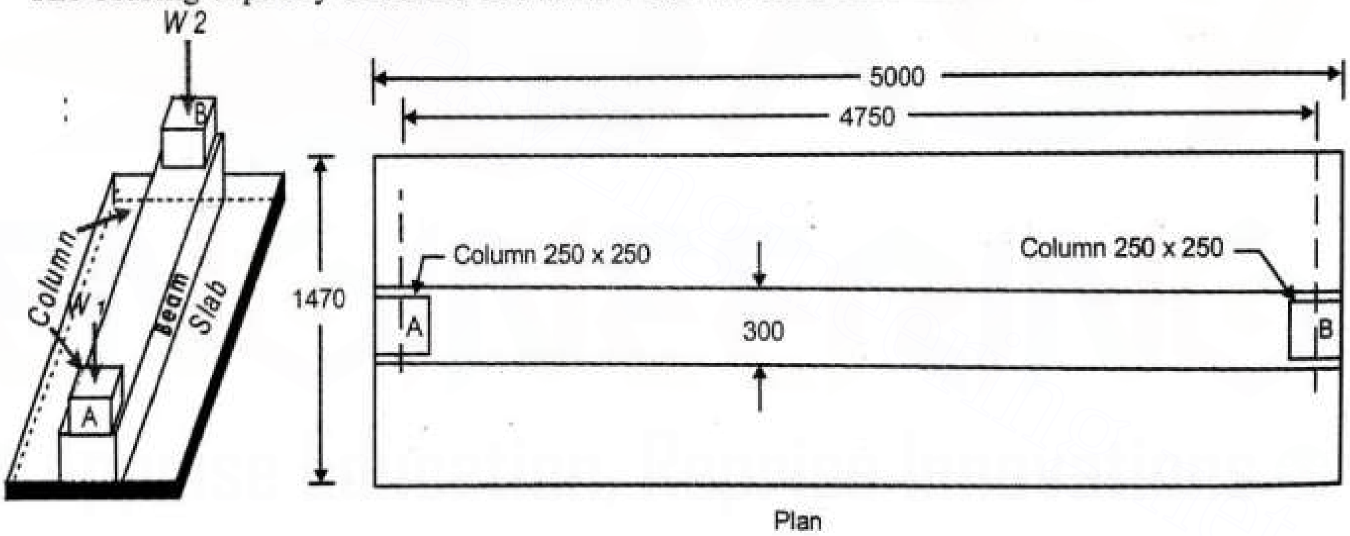


Fig. 12.8 Plan of Both Columns on Boundaries

Working load carried by column A = P_A = 500 kN

Working load carried by column B = P_B = 500 kN

Total load from columns = $P_A + P_B$ =500 + 500 = 1000 kN

Self weight of footing at 10 % = 100 kN

Total working load = 1100 kN

Ultimate load = 1.5 x 1100 = 1650 kN

Area of footing = 1650/(1.5 x 150) = 7.33 m^2

Length of the footing = 5 m

Required width of footing = 7.33 / 5 = 1.47 m

Provide footing of size 1.47 m x 5 m

Area provided A_f = 1.47 x 5 = 7.35 m^2

Ultimate load P_u = 1.5 x 1000 = 1500 kN

Upward intensity of soil pressure $P_u / A_f = w_u$ =1500 / 7.35 = 204 kN/m^2

Design of slab :

Intensity of upward pressure = w_u = 204 kN/m^2

Consider one meter width of the slab ($b = 1\ m$)

Load per m run of slab = 204 × 1 = 204 kN/m

Projection of the slab on both sides of column = $\frac{(1470 - 250)}{2}$ = 610 mm

Maximum ultimate moment M_u = $204 \times 0.61^2 / 2 = 38\ kN.m.$

For M20 and Fe415, $R_{u.max}$ = 2.76 N/mm^2 and $k_{u.max}$ = 0.48 ... ... (*Table 4.1.1*)

Required effective depth = $\sqrt{38 \times 10^6 / (2.76 \times 1000)}$ = 118 mm

Since the slab is in contact with the soil clear cover of 50 mm is assumed.

Using 12 mm diameter bars

Required total depth = 118 + 12 / 2 + 50 = 174 mm

Provide total depth = 200 mm

Effective depth provided d = 200 – 50 – 12 /2 = 144 mm

$$\text{Required,}\quad A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 38 \times 10^6}{20 \times 1000 \times 144^2}}\right] \times 1000 \times 144$$

= 830 mm^2

Provide #12 mm diameter bar at 110 mm c/c

Area provided = 1000 × 113 / 110 = 1027 mm^2 > 830 mm^2

Larger area has been provided for resisting shear.

Check the depth for one – way shear considerations :

$$p_t = \frac{100 \times 1027}{(1000 \times 144)} = 0.71\ \%\quad ,\ \tau_{uc} = 0.55\ N/mm^2$$ ... ... (*Table 4.4.1*)

k for 200 mm thick slab = 1.2 ... ... (*Table 4.4.2*)

Permissible shear stress = 1.2 × 0.55 = 0.66 N/mm^2

Shear resisted by concrete V_{uc} = 0.66 × 1000 × 144 / 1000

= 95 kN

Design shear at distance d = 144 mm from the face of column

V_{uD} = 204 x (0.61 - 0.144) = 95 kN = V_{uc} ∴ safe

Check for Development Length

Required development length = 47 ϕ = 47 x 12 = 564 mm

Available development length = 610 - 50 + 8ϕ = 610 - 50 + 8 x 12

= 656 mm > 564 mm ∴ safe

Distribution steel :

Required A_{st} = 0.12 $b\,D$ / 100 = 0.12 x 1000 x 200 / 100 = 240 mm^2

Using #8 mm bars , spacing = 1000 x 50 / 240 = 208 mm

Provide distribution steel of #8 mm at 200 mm c/c

Design of Longitudinal Beam

As the width of the footing is 1.47 *m* , the net upward soil pressure per metre length of the beam

$$= w_u = 204 \times 1.47 = 300\ kN/m$$

Shear force and Bending moment : (Fig. 13.9)

$$V_{CA} = 300 \times 0.125 = 37.5\ kN = V_{DB}$$
$$V_A = 750 - 37.5 = 712.5\ kN = V_B$$

The shear force is zero at mid-span (2.5 *m*) where the BM is maximum

Maximum bending moment $M_{uE} = 750 \times 4.75/2 - 300 \times 5^2/8$
$= 843.8\ kN.m$

The Bending Moment and Shear force Diagram is shown in Fig. 12.9

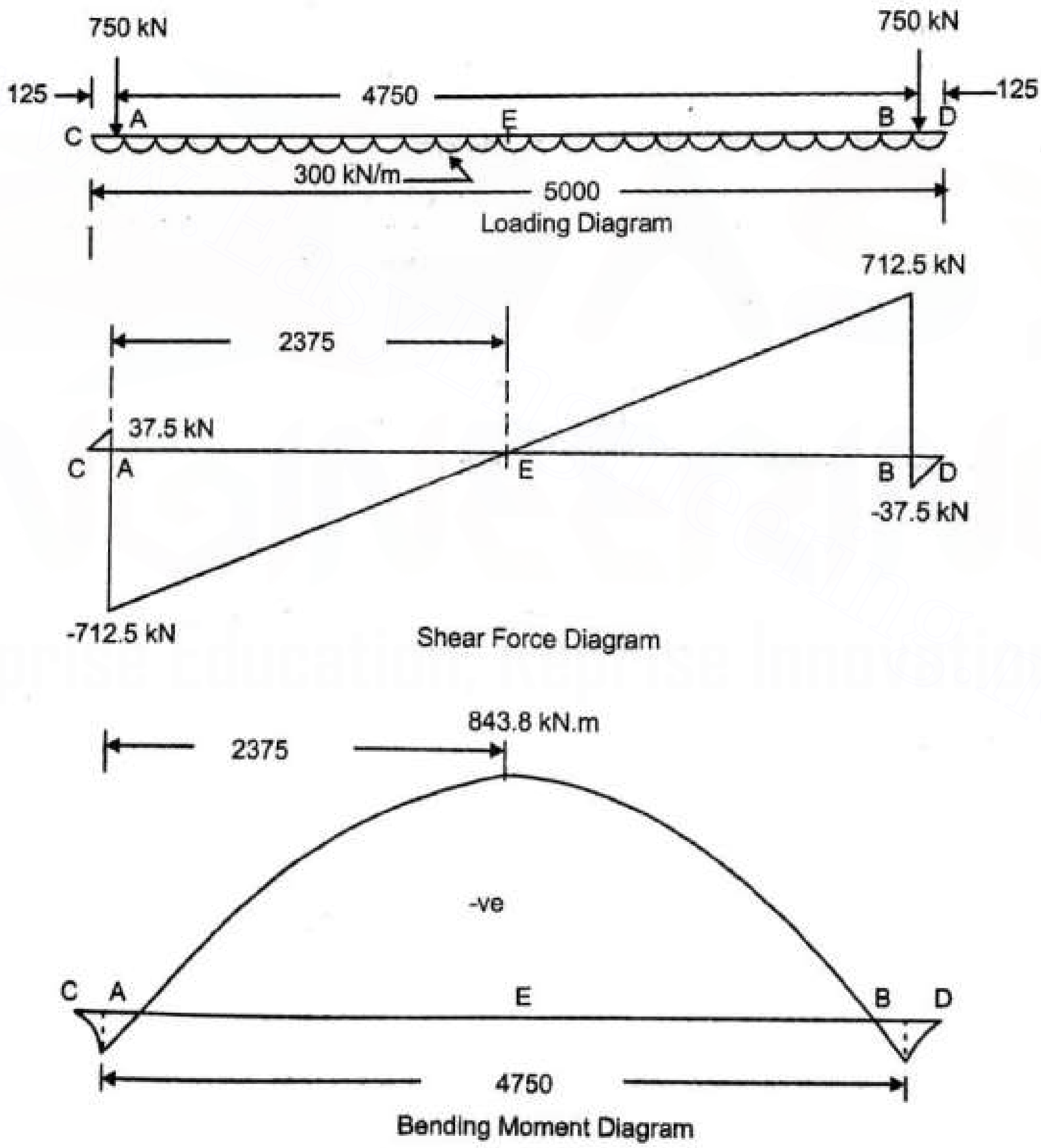


Fig. 13.9

Fig 12.9 Bending Moment and Shear Force Diagrams

As the beam is provided above the slab , the slab lies in the compression zone with respect to bending of the beam therefore, it acts as an isolated *T*-section. Even though the beam acts as a flanged section the depth required for shear in foundation beam is usually more than that required for flexure hence trial depth is assessed treating the section as rectangular.

Width of the beam = 300 *mm*

Required $d = \sqrt{843.8 \times 10^6/(2.76 \times 300)} = 1009$ *mm*

Assume the size of the beam = 300 *mm* x1000 *mm* and bar diameter 25 *mm*

Effective depth provided d = 1000 -50 -25 -25/2 = 912.5 *mm*

$$b_f = \frac{0.7 \times 4750}{[0.7 \times 4750/1470 + 4]} + 300$$

= 830 *mm*

$$\therefore \quad \text{Area of steel } A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 843.8 \times 10^6}{20 \times 830 \times 912.5^2}}\right] \times 830 \times 912.5$$

= 2773 mm^2

Provide 6 - #25 *mm* , Area provided = 2945 mm^2

Provide 2-#16 *mm* at bottom to anchor the stirrups.

The bars are curtailed at a distance of 700 *mm* from the centre of support.

Side face reinforcement = 0.1 x 300 x 1000 /100 = 300 mm^2 . Provide 4-#12*mm*

Design of shear reinforcement :

Shear at a distance *d* from the face of support = 712.5 - 300(0.9125 + 0.125)

V_{uD} = 401.25 *kN*

$$p_t = \frac{100 \times 2945}{300 \times 912.5} = 1.08\%$$

$$\tau_{uc} = 0.62 + \frac{0.05 \times 0.08}{0.25} = 0.636 \ N/mm^2 \text{. ... (Table 5.7.1)}$$

Shear resisted by concrete $V_{uc} = \tau_{uc} bd = 0.636 \times 300 \times 912.5 / 1000 = 174.1$ *kN*.

Shear resisted by minimum stirrups

$V_{usv.min} = 0.4\ bd = 0.4 \times 300 \times 912.5 \times 10^{-3} = 109.5$ *kN*

$V_{ur.min} = 174.1 + 109.5 = 283.6\ kN < V_{uD}$ (= 401.25 *kN*)

Shear reinforcement is required.

Shear to be resisted by stirrups

$= V_{usv} = V_{uD} - V_{uc} = 401.25 - 174.1 = 227.15$ *kN*

Using 8 *mm* diameter 2 – legged stirrups,

$$\frac{V_{usv}}{d} = \frac{1000 \times 227.15}{912.5} = 249 \ N/mm$$

Spacing s = 145 *mm* ... ... (Table. F-7)

$$\text{or} \quad \text{Spacing } s = \frac{0.87 \times 415 \times (2 \times 50) \times 912.5}{(227.15 \times 1000)} = 143 \ mm$$

Provide #8 *mm* stirrups at 140 *mm* c/c

Zone of shear reinforcement

$$L_{s1} = \frac{(V_{u.max} - V_{ur.min})}{w_u} = \frac{(712.5 - 283.6)}{300} = 1.43\ m$$

$$\textit{Spacing of minimum shear reinforcement} = \frac{0.87 \times 415 \times 100}{(0.4 \times 300)} = 300\ mm$$

Provide #8 *mm* 2-legged stirrups at 140 *mm* c/c from each end for a distance of 1.43 *m* and remaining portion provide #8 *mm* at 300 *mm* c/c

The details of Reinforcement are shown in *Fig. 12.10*

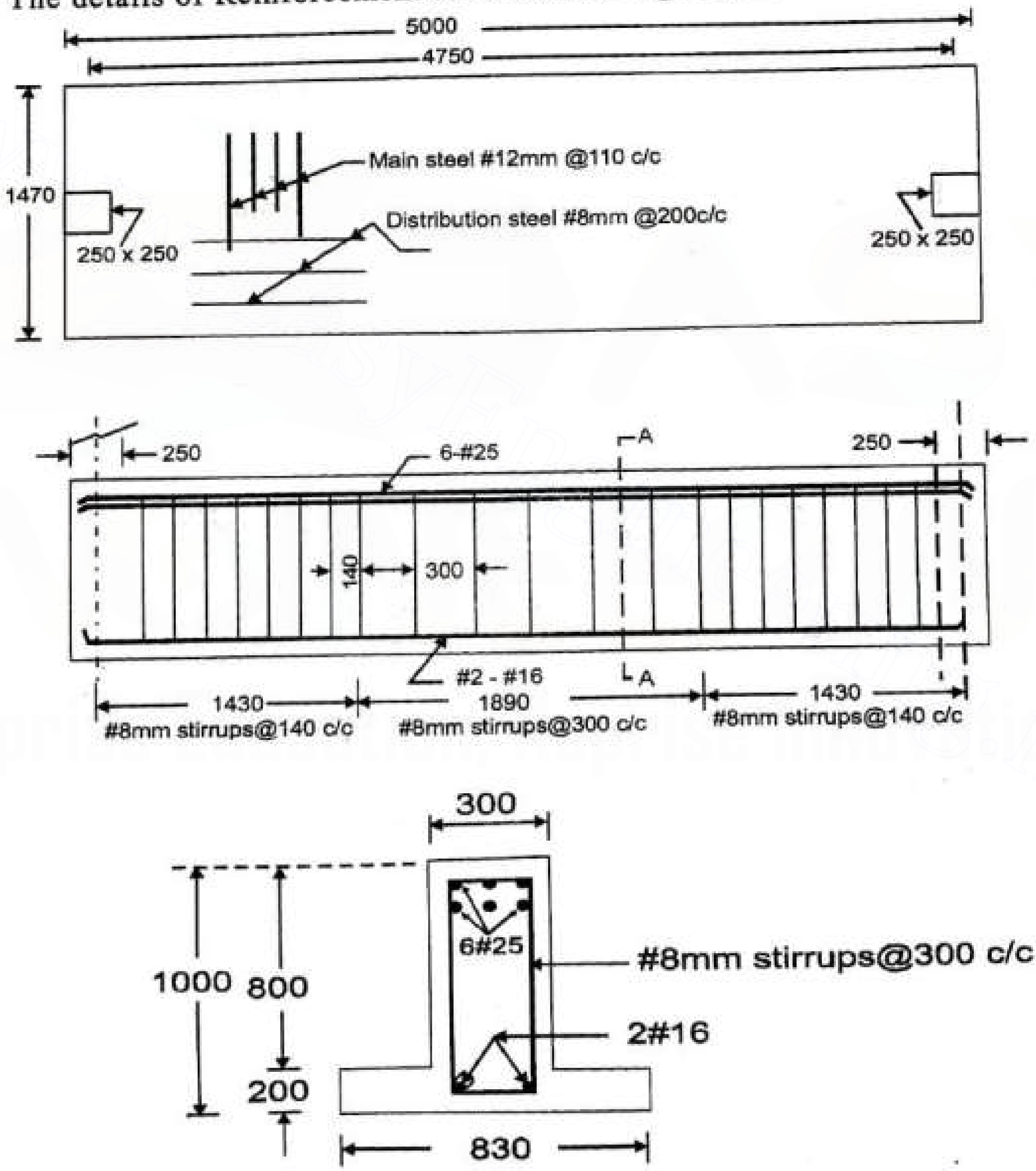


Fig.12.10 Details of Reinforcement

References:

12.1 Shah,V. L..and Karve, S.R. " Limit State Theory and Design of Reinforced Concrete ", Structures Publications, Parvati , Pune 411009, Chapter -12, Sect.12.9,

12.2 Shah,V. L..and Karve, S.R. " Limit State Theory and Design of Reinforced Concrete ", Structures Publications, Parvati , Pune 411009, Chapter -12, Sect.12.9.3,

CHAPTER - 13

EARTHQUAKE ANALYSIS AND DESIGN

13.1 WHAT IS EARTHQUAKE ?

An ***earthquake*** is vibration of earth surface by waves emerging from the source of disturbance in the earth by virtue of release of energy in the earth's crest.

The earthquake can be divided into two categories depending upon their origin *viz.* ***Tectonic*** and ***Volcanic***. *Volcanic* earthquakes are those associated with volcanic eruptions and have limited field. Tectonic earthquakes are associated with the sudden dislocation of large rock masses along the geological fault. When the earthquake occurs beneath the sea, the water above the deformed area gets displaced from equilibrium position. Series of waves or wave train, called earthquake generated tsunami waves are formed as huge water columns which can have vertical run-up height of 10m, 20m, and even 30m, off shore above sea level. Tsunamis have great erosional potential, stripping beaches of sand, inundating costal areas, and crushing homes leading to great loss of life.

Rocks being elastic material strain energy gets stored in them during the deformation that take place due to the gigantic tectonic plate actions (sliding of Earth's mass in pieces) that occur in the earth. In this process tremendous amount of strain energy stored in the rock gets suddenly released and waves emanate and propagate in all directions as shock waves. The horizontal component is generally more intense than the vertical component. Since the structures are to be designed to withstand their own weight the vertical seismic forces do not play important part except when stability forms the criteria for design.

The point from which the waves first originate is called *Focus. The* geographical point on the surface of earth vertically above the focus is called *Epicenter. The* distance between epicenter and recording station is called *epicentral distance. (see Fig.13.1.1.)*

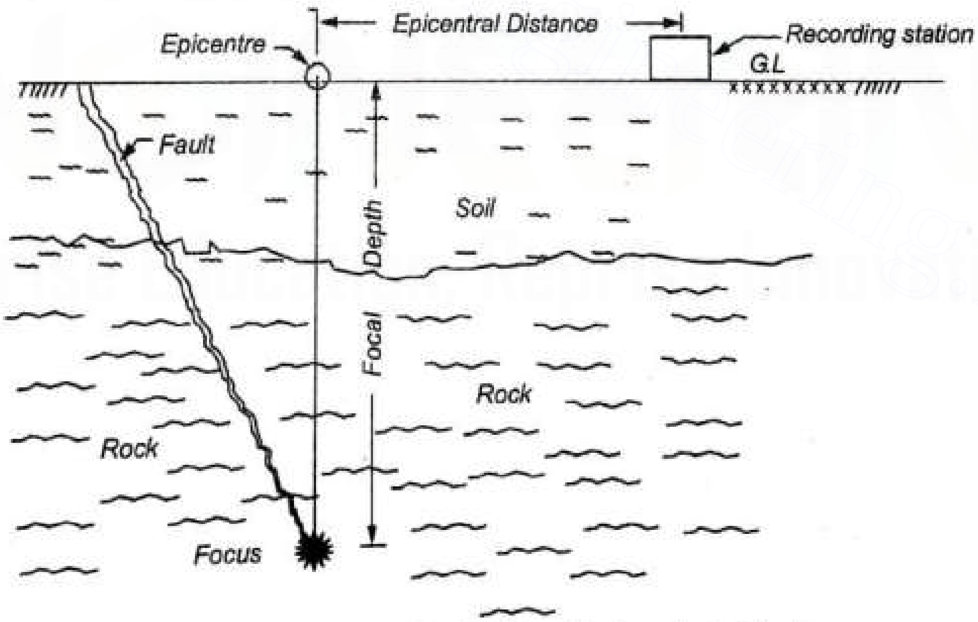


Fig. 13.1.1 Focus, Epicenter, Epicentral Distance

13.2 EARTHQUAKE MAGNITUDE AND INTENSITY

The *magnitude* of earthquake is a quantitative measure of size of earthquake and the strain energy released.

The commonly used magnitude scale is *Richter scale. The* earthquake ground motion is measured by an instrument called a *Seismograph.* The time, location and magnitude of an earthquake can be determined from the data recorded by seismograph stations. At present digital instruments using modern computer technology are used.

Intensity is the measure of the strength of grounding shaking manifested at a place during earthquake. It is recorded by Modified Mercalli Intensity **(MMI)** scale and the MSK scale. The lines joining places with equal seismic intensity are called *isoseismal*. It may be noted that for a certain magnitude of earthquake different locations experience different levels of intensity. To elaborate the distinction between magnitude and intensity consider the analogy of an electric bulb. The size of bulb ***say*** 100 watts is like the magnitude of an earthquake, and the illumination *or* brightness which varies with the location is like the intensity of shaking at that location. It may be noted that destruction caused due to an earthquake, even though partly depends on the magnitude, it is mainly a function of intensity *or* severity of shaking which depends on factors such as focal depth, epicentral distance, local geology and structural characteristics.

13.3 OBJECTIVE OF DESIGN

The design philosophy should be based on consequences of damage. It will depend on the importance of the structure, impact and psychological effects on the minds of the people, and wide spread disasters. For example service structures such as water pipe line, hospitals, fire stations *etc*. must remain functional immediately after the earthquake, and therefore, need higher level of factor of safety . But the monumental structures, dams which can cause horror, major loss of property and loss of life should be designed for still higher level of earthquake protection.

In general the design philosophy may be categorized into three limit states.

i) Serviceability limit state *or* Minor damage philosophy
ii) Damage controlled limit state *or* Reparable damage philosophy
iii) Survival limit State *or* Ir - reparable damage philosophy.

Serviceability Limit State :- Under minor tremor the load carrying structures (such as slab, beam, column, footing) should not be damaged. But non - load bearing structures may sustain reparable damage.

Damage Controlled Limit State : - Under moderate earthquake the load bearing structures may suffer reparable damage while the other structures may get badly damaged but can be easily replaced after the earthquake.

Survival Limit State :-Under strong shaking the building may suffer reparable damage but should be able to respond without total collapse to avoid devastation of life and property (so that the life and property may be saved.)

It is not possible to design the structure to remain totally safe during the tremor. This is because in that case the cost of construction will be high while the earthquake is a rare phenomenon. Therefore, damage in the building is unavoidable. The only alternative left is to control the damage to the acceptable level so that cost of repairs involved is reasonable.

13.4 BEHAVIOR OF A STRUCTURE AND FACTORS AFFECTING

Consider a single storey building with columns fixed at the base and having larger roof mass which is much greater than the column mass. As the ground shakes (say from right to left) due to earthquake, the base will experience the horizontal motion and roof slab will get displaced to right through a distance equal to say 'u' *(see Fig. 13.4.1)*. In other words the effect of ground motion can be considered as applying an external force f producing displacement u.

Such type of motion one can experience when the bus suddenly starts, your body is thrown backwards as if some force is applied in the opposite direction of motion of the bus but you may not fall down because your body resists this force. Similarly the single storey building resists this outward motion. The resisting force is called the inertia force which is equal to mass times the acceleration. The above system can be visualized as an idealized one storey frame subjected to an external force f which can be described as single degree freedom system. Thus the degree of freedom is a number of independent displacement components describing the motion of the system.

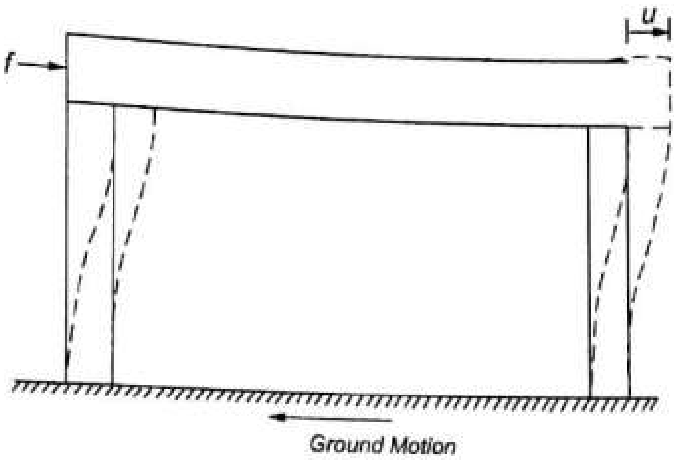


Fig. 13.4.1 Deformation due to Ground Motion

13.4.1 Free Vibration of Single Degree Freedom System (SDF).

As the motion of single degree of freedom system can be visualized as an idealized one storey frame or spring mass system, subjected to external force $f(t)$. If the spring system having stiffness k shown in *Fig. 13.4.2a* is pulled through distance $u(0)$ and let go that in setting time t, $f(t) = 0$, it will continue to vibrate about its central position.

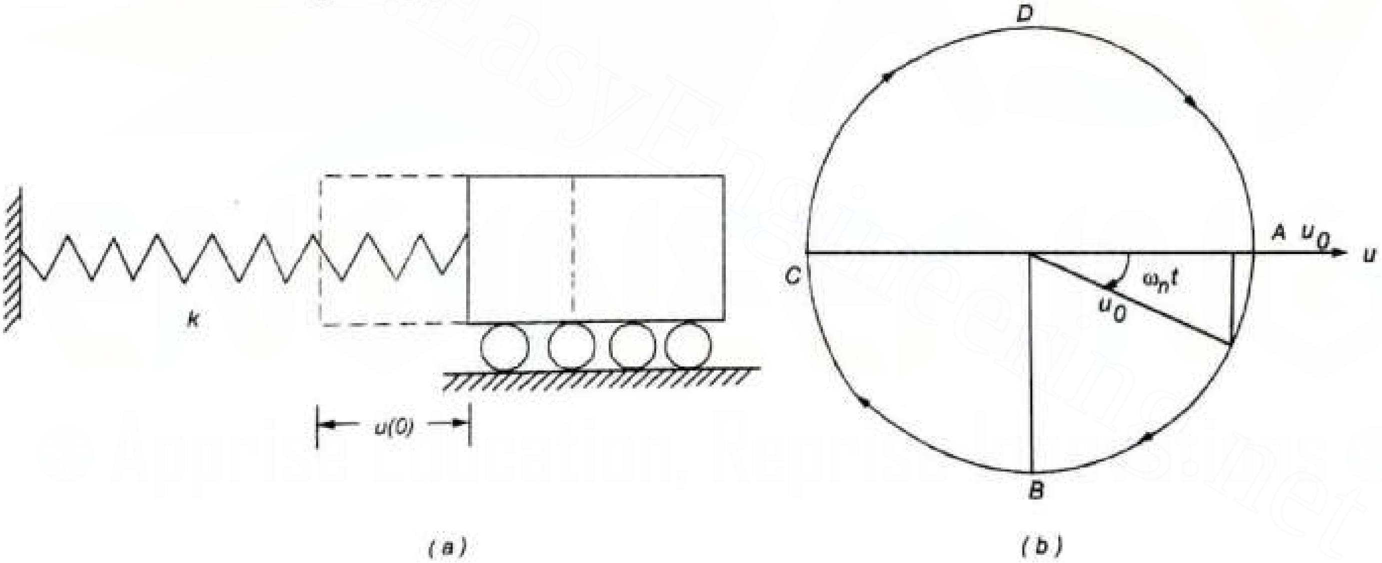


Fig. 13.4.2 Spring Mass System

The internal force resisting deformation = $f_s = k \times u$, where, k = spring stiffness .
For dynamic equilibrium the internal force must be equal to the inertia force which is equal to mass x acceleration *or* $m\ddot{u}$
Mathematically this may be expressed as :

$$m\ddot{u} + ku = 0 \qquad \ldots\ \ldots \quad (13.4.1)$$

$$\text{or} \quad m\frac{d^2u}{dt^2} + ku = 0 \ , \quad \text{Putting} \quad \frac{k}{m} = \omega^2_n \quad \text{or} \quad \omega_n = \sqrt{\frac{k}{m}}$$

$$\frac{d^2u}{dt^2} + \omega^2_n u = 0$$

This is linear differential equation with constant coefficients which is of the form $(D^2 + \omega^2_n) u = 0$

The general solution of the equation is given by :

$$u = A\cos\omega_n t + B\sin\omega_n t \quad \ldots\ \ldots (13.4.2)$$

Also , $$\frac{du}{dt} = -A\,\omega_n \sin\omega_n t + B\,\omega_n \cos\omega_n t$$

where , A and B are arbitrary constants to be determined from initial conditions.

At $t = 0$, $u = u(0)$

$\therefore\ u(0) = A\cos(0) + 0 \qquad \therefore\ A = u(0)$

Again at $t = 0$, velocity $\frac{du}{dt} = 0 \quad \therefore\ B = 0$

The solution of *Eq.13.4.1* is ;

$$u = u(0)\cos\omega_n t \quad \ldots\ \ldots (13.4.3)$$

At initial position, $u = u(0)$

$\therefore\ u(0) = u(0)\cos\omega_n t$ i.e. $\cos\omega_n t = 1$

To satisfy this condition, either $\omega_n t = 0$ or $\omega_n t = 2\pi$, But $\omega_n > 0$

$\therefore$ either $t = 0$ or $t = 2\pi/\omega_n$

At $t = 0$, $u = u(0)$ the particle will be at A, the maximum distance from mean position ... *(Eq.13.4.3)*

At $t = \pi/(2\omega_n)$, $u(0) = 0$ it is at position B.

At $t = 2\pi/\omega_n$, $u = u(0)$, again it will be at the same position *i.e.* it completes one oscillation.

It is clear that the mass is executing *simple harmonic motion* and moving with an angular velocity ω_n

The displacement of the mass is the projection on the diameter.

The time required for an undamped system to complete one cycle of free vibration is called *natural period of vibration* of the system and denoted as T_n having unit of seconds. It is related to natural circular frequency of vibration ω_n radians/sec. The natural vibration properties *viz.* ω_n, T_n, and f_n depend only on the mass and stiffness of the structure . The term natural frequency of vibration is applied to both ω_n and f_n.

Thus, the time required for one oscillation or natural period of vibration is :

$$T_n = \frac{2\pi}{\omega_n} \quad \ldots (13.4.4a)$$

$$\frac{1}{T_n} = f_n = \frac{\omega_n}{2\pi} = \frac{\sqrt{k/m}}{2\pi} \quad \ldots\ \ldots (13.4.4b)$$

f_n is natural frequency of vibration has unit of cycles per second *or* hertz (Hz) ... ... *(13.4.4c)*

$$\omega_n^2 = \frac{k}{m} \quad or \quad \omega_n = \sqrt{\frac{k}{m}}$$

In general free vibration is initiated by disturbing any system from its static equilibrium position and imparting the mass some displacement, $u(0)$ and velocity $\dot{u}(0)$ at time $t = 0$

Eq.13.4.2 can be written as

$$u = A\cos\omega_n t + B\sin\omega_n t$$

$$\dot{u} = \frac{du}{dt} = -A\,\omega_n \sin\omega_n t + B\,\omega_n \cos\omega_n t$$

At $t = 0$, $u = u(0)$, $A = u(0)$

At also at $t = 0$, velocity $\dot{u} = \dot{u}(0)$

$$\therefore\ \dot{u} = \dot{u}(0) = \omega_n B \quad or \quad B = \frac{\dot{u}(0)}{\omega_n}$$

In general *Eq. 13.4.2* can be rewritten as :

$$u = u(0)\cos\omega_n t + \frac{\dot{u}(0)}{\omega_n}\sin\omega_n t \qquad (13.4.5)$$

The displacement time curve is plotted as per *Eq. 13.4.5* and shown in *Fig. 13.4.3*

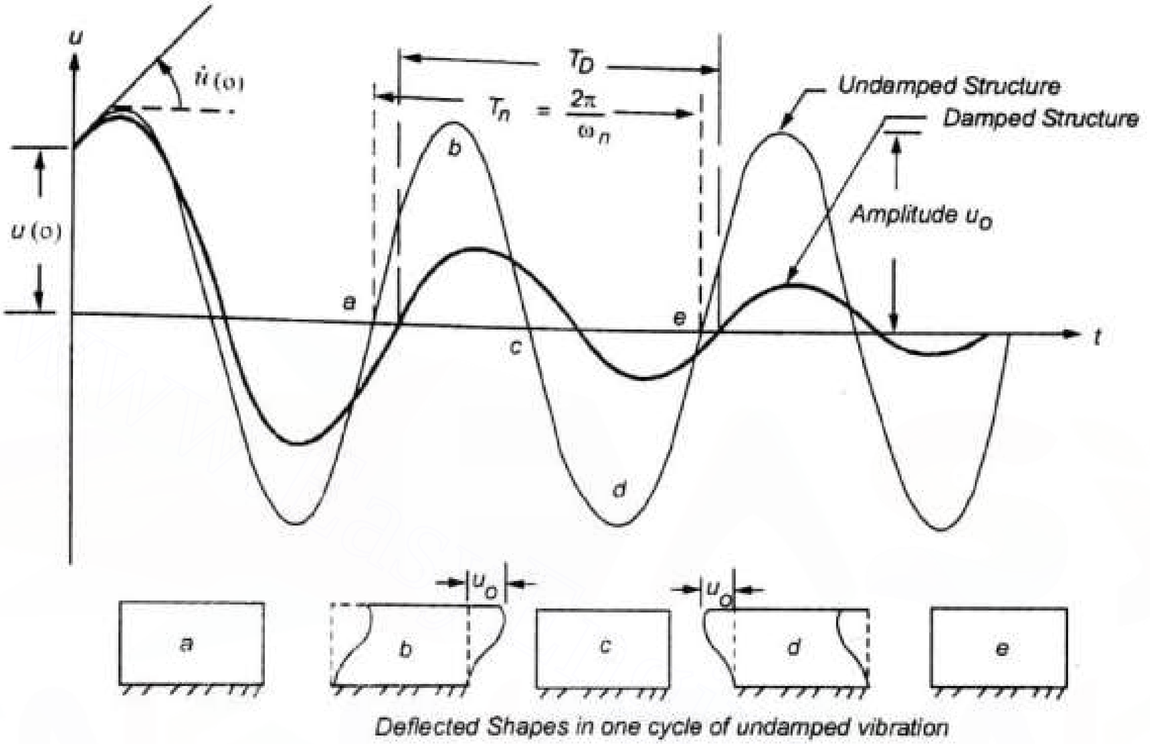


Fig. 13.4.3 Undamped and Damped Structure

It shows that the system vibrates about its static equilibrium position and repeats itself after its natural period of vibration (T_n) of $2\pi/w_n$ seconds describing simple harmonic motion.

The portion *a-b-c-d-e* describes one cycle of free vibration from its static equilibrium position (*or* undeformed position) at *"a"*. Further, the mass moves with decreasing velocity and reaching maximum positive displacement u_o at *"b"* at which time the velocity is zero. The displacement begins to decrease while velocity increasing reaching its equilibrium position at *"c"*. At that instant velocity is maximum and hence mass continues to move to its left reaching negative displacement u_o at *"d"*. Again the displacement decreases with mass reaching equilibrium position at *"e"*. The total time taken for one cycle is $2\pi/w_n$. The velocity at that instant is maximum and the mass begins its another cycle of vibration.

13.4.2 Free Vibration with Damping of SDOF

When buildings are mildly shaken and let go (called free vibration,) the amplitude of peak lateral displacements continue decreasing and finally come to rest. This phenomenon is called *Damping*. *Damping is defined as dissipation of energy in a free vibration system*. It is the reduction in amplitude of vibration expressed as a percentage of critical damping. ***Critical Damping** is the damping beyond which the free vibration motion will NOT be oscillatory*. The loss of heat in friction, air resistance, imperfect elasticity of material, slipping, sliding, cracking etc. effects in reducing the amplitude of vibration *or* damping. The common practice is to assume this damping as viscous (*i.e.* dissipation associated with moving of a plunger in oil). Physically the energy dissipation mechanism is represented by dashpot.

Viscously Damped Free Vibration.

Consider a structure shown in *Fig.13.4.4* under single degree of freedom (*SDF*) with damping. It is in dynamic equilibrium under the applied force $f(t)$ and internal resisting inertia force f_i, damping force f_d and stiffness force f_s. The free body diagram is shown in *Fig. 13.4.4b*.

The equation of motion can be rewritten in terms structural properties as :

$$f_i + f_d + f_s = f(t)$$

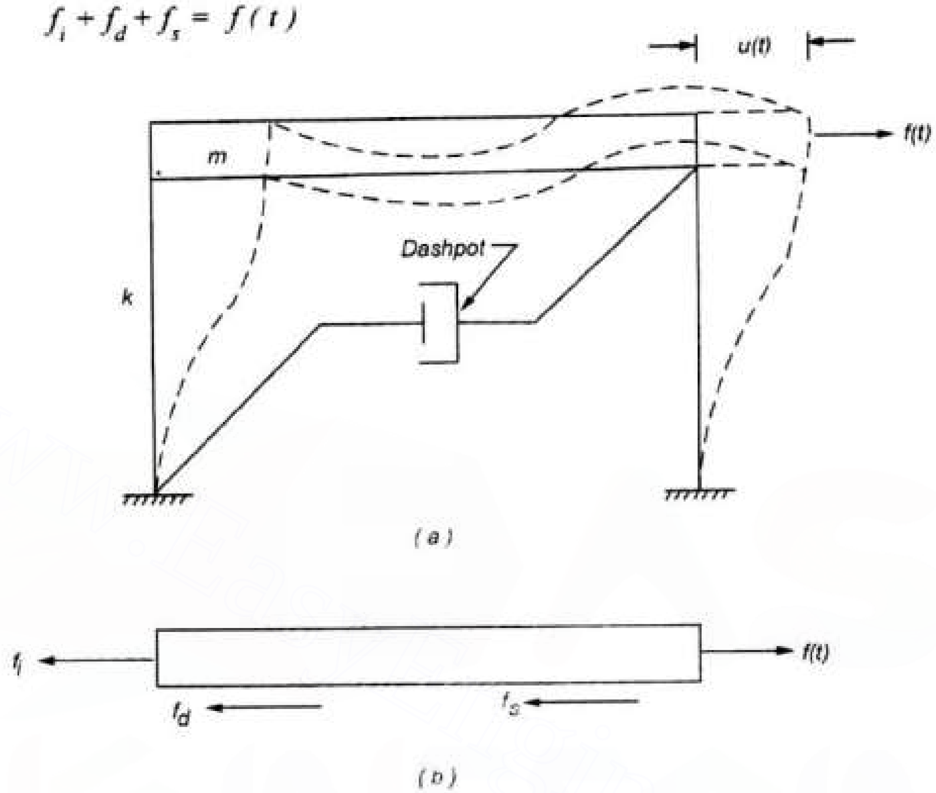


Fig. 13.4.4 Viscously Damped free Vibration

$$m\,\ddot{u}(t) + c\,\dot{u}(t) + k\,u(t) = f(t)$$

putting $f(t) = 0$, the governing equation of free vibration system can be written as :

$$\ddot{u}(t) + 2\xi\omega_n \dot{u}(t) + \omega_n^2 u(t) = 0 \qquad \text{... ... 13.4.6}$$

where, $c = 2\xi m\,\omega_n$ is a damping constant which is a measure of energy dissipation in a cycle of free vibration

ξ = damping ratio.

$$\text{damping ratio } \xi = \frac{c}{2m\omega_n} = \frac{c}{c_{cr}}$$

c_{cr} = critical damping coefficient.

The damping beyond which the free vibration ceases to be oscillatory is called **critical damping.**

The damping ratio is a dimensionless measure of damping and it depends on mass and stiffness.

If $c = c_{cr}$ then $\xi = 1$ *i.e.* the system returns to its equilibrium position without oscillation

$\xi = 0$ represents free vibration system without damping.

If $\xi > 1$, In this case also the system returns to its equilibrium position but at a slower rate.

It $\xi < 1$, The system continues to oscillate about its equilibrium position with a gradually decreasing magnitude.

These responses are shown in *Fig.13.4.5*

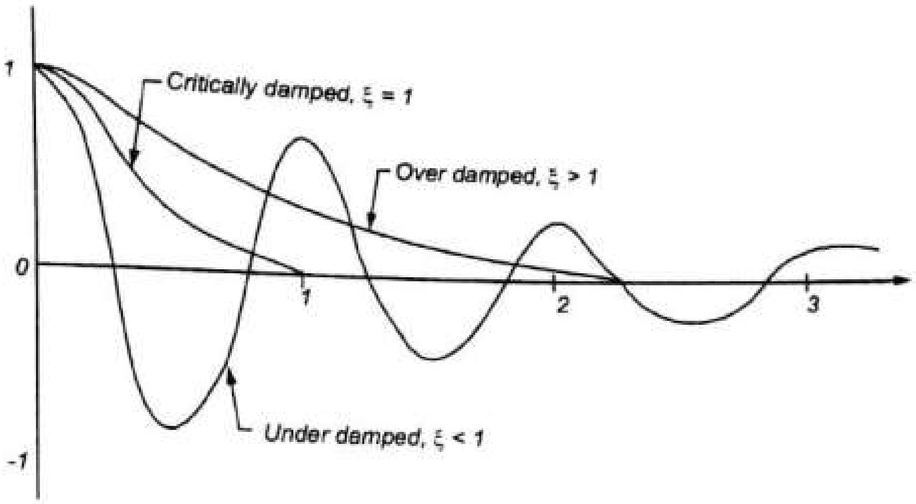


Fig. 13.4.5 Damping Effects on Free Vibrations

The effect of damping on free vibration is shown in Fig. 13.4.3 by bold lines.

It will be seen that the displacement amplitude of undamped system remains the same in all cycles of vibrations while the damped system oscillates with decreasing amplitude with every cycle of vibration.

The natural frequency of damped vibration ω_D is a function of ω_n and ξ and is given by :

$$\omega_D = \omega_n \sqrt{1-\xi^2} \qquad \text{... (13.4.7)}$$

The natural period of damped vibration is :

$$T_D = \frac{2\pi}{\omega_D} = \frac{T_n}{\sqrt{1-\xi^2}} \qquad \text{... (13.4.8)}$$

Damping has the effect of lowering the natural frequency of vibration from w_n to w_D and lengthening the time period from T_n to T_D. However, these effects are negligible for damping ratio below 20%, a range in which most structures lie. The value of damping for buildings may be taken between 2% to 5% of the critical for dynamic analysis of steel and RCC buildings respectively.

13.4.3 Equation of Motion – Earthquake Ground Motion of SDOF

During earthquake, the motion due horizontal acceleration is induced at the base of the structure. Earthquake vibration gives swing to the mass of the structure causing lateral displacement $u_g(t)$, the structure also moves along with it but due to its mass its displacement $u^t(t)$ is more than the ground displacement $u_g(t)$. (*Fig. 13.4.6*).

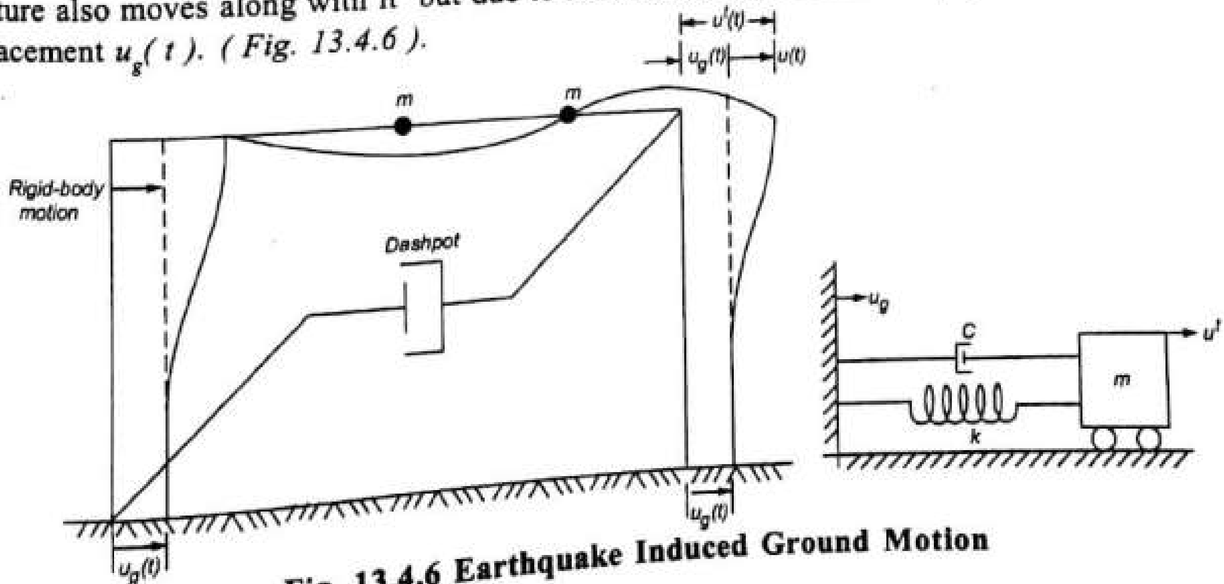


Fig. 13.4.6 Earthquake Induced Ground Motion

The displacement including that of the structure is given by :

$$u(t) = u^t(t) - u_g(t)$$

$$m[\ddot{u}(t) + \ddot{u}_g(t)] + c\ \dot{u}(t) + k\,u(t) = 0$$

where, $\ddot{u}_g(t)$ is ground acceleration measured with respect to fixed coordinates.

$$\text{or} \quad m\ddot{u}(t) + c\dot{u}(t) + k\,u(t) = -\ddot{u}_g(t)$$

But $c = 2\xi\, m\omega_n$ and $k = m\omega^2_n$, substituting these values in above equation we get

$$\ddot{u}(t) + 2\xi\omega_n\ \dot{u}(t) + \omega^2_n u(t) = 0$$

where, $\ddot{u}(t), \dot{u}(t), u(t)$ are respectively acceleration, velocity and displacement vectors measured relative to ground

$\ddot{u}_g(t)$ is ground acceleration measured with respect to fixed coordinates.

It will be seen that for a given base motion $\ddot{u}_g(t)$, the deformation response $u_g(t)$ of the system depends on the natural frequency ω_n *or* natural period T_n ($=T/\omega_n$) of the system and the damping ratio. Thus, any two systems having the same natural period T_n and damping ratio ξ will have the same deformation response *u(t), even though* one system may be stiffer than the other.

For the same vibration period T_n but with variable damping ratio it will be observed that systems with more damping respond less than lightly damped system *or* in other words lightly damped systems are subjected to more peak deformation than more damped systems.

Damping has been observed to increase with increase in response and damage during the earthquake.

13.5 MULTI - DEGREE OF FREEDOM SYSTEM (MDOF)

A multi - storey building is a three dimensional structure. It has six degrees of freedom at each node (namely 3 displacements / forces and three rotations / moments). However, for developing the basic concept of dynamics of multi-storey building, the number of degrees of freedom are reduced by making the following assumptions:

1. The lateral displacement at all points at a floor level are the same.
2. The columns are axially rigid and no axial deformation can occur.
3. The mass of the building is concentrated at floor level.

Using these assumptions the equations for multi-degree freedom (*MDF*) system can be developed.

Consider an idealized two storey frame subjected to external forces $f_1(t)$ and $f_2(t)$ acting at the floor level subjected to displacement u as shown in *Fig. 13.5.1*

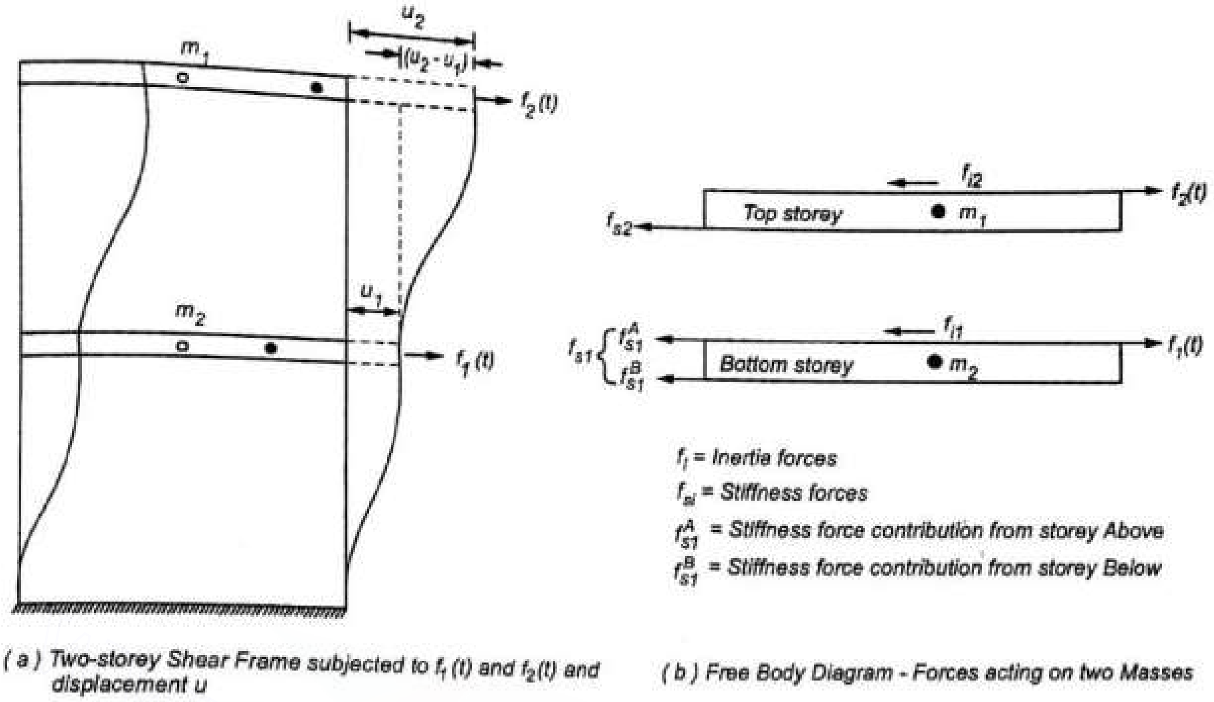
Neglecting the *effect of damping*, the general equation of motion for SDOF can be written as :

$$f_j + f_{sj} = f_j(t) \quad \text{or} \quad m_j\,\ddot{u}_j + f_{sj} = f_j(t) \qquad (13.5.1)$$

where, f_j = *inertia force*
f_s = stiffness force

For a two storey building $j = 1$ and 2, *Eq. 12.5.1* can be written in matrix from as :

$$\begin{bmatrix} m_1 & 0 \\ 0 & m_2 \end{bmatrix} \begin{Bmatrix} u_1 \\ u_2 \end{Bmatrix} + \begin{Bmatrix} f_{s1} \\ f_{s2} \end{Bmatrix} = \begin{Bmatrix} f_1(t) \\ f_2(t) \end{Bmatrix}$$



(a) Two-storey Shear Frame subjected to $f_1(t)$ and $f_2(t)$ and displacement u

(b) Free Body Diagram - Forces acting on two Masses

Fig. 13.5.1 Two Storey Frame

(A) Force Displacement Relationship

The force f_s is related to displacement u ,

At the j^{th} storey, storey shear s_j to storey deformation δ_j *as* :

$$\delta_j = u_j - u_{j-1} \quad or \quad S_j = k_j \delta_j \qquad \text{... (13.5.3)}$$

The storey stiffness k_j is the sum of lateral stiffnesses of all the columns in the storey

For a column with fixed ends the lateral stiffness

$$k_j = \sum_{j=1}^{n} \frac{12EI_c}{h^3} \qquad \text{...(13.5.4)}$$

where, I_c = moment of inertia of the column in the storey

n = number of storeys

h = height of storey.

For a two storey single bay frame let m_1 and m_2 be the lumped floor masses under going displeacements u_1 and u_2 for bottom and top storey respectively.

Now , the resisting forces f_{s1} *and* f_{s2} *can* be related to displacements u_1 *and* u_2.

The force f_{s1} at the first floor consists of contribution f_{s1}^A from storey above f_{s1}^B from storey below

$$f_{s1} = f_{s1}^A + f_{s1}^B$$

The displacements are :

$\delta_2 = u_2 - u_1$ and $\delta_1 = u_1$ *(see Fig. 13.5.1)*

The stiffness forces in the storey are proportional to the relative displacement in the storey. If the lateral stiffness of all columns in the two storeys are k_1 and k_2 then the stiffness forces in each storey can be written as :

∴ At first floor , (13.5.5a)

$$f_{s1} = k_1(u_1 - u_o) - k_2(u_2 - u_1)$$

At second floor , (13.5.5b)

$$f_{s2} = k_2(u_2 - u_1)$$

From *Fig. 13.5.1b* it will be seen that f_{s1}^A and f_{s2} are equal in *magnitude* because both represent the shear in the second storey.

Eq. 13.5.5a and Eq 13.5.5b in the matrix form are :

$$\begin{Bmatrix} f_{s1} \\ \hline f_{s2} \end{Bmatrix} = \begin{bmatrix} k_1+k_2 & -k_2 \\ -k_2 & k_2 \end{bmatrix} \begin{Bmatrix} u_1 \\ \hline u_2 \end{Bmatrix} \quad or \quad \{f_s\} = [k]\ \{u\} \qquad \ldots (13.5.6)$$

This gives the relation between force and displacement through the stiffness matrix for a two - storey shear bulding.

(B) Dynamic Equilibrium

Neglecting damping, the system shown in *Fig.13.5.1*, is in dynamic equilibrium under the action of inertia forces f_i and stiffness forces f_{si}

For undamped free vibration $f(t) = 0$, the equation of dynamic equilibrium is wirtten as :

$$m\,\ddot{u} + k\,u = 0 \quad or \quad [m]\ \{\ddot{u}\} + [k]\ \{u\} = 0 \qquad \ldots (13.5.7)$$

As the mass is assumed to execute simple harmonic motion. The solution of the equation can be expressed as :

$$u = \phi_n \sin\omega_n t \text{ then } \ddot{u} = -\phi_n \omega_n^2 \sin\omega_n t$$

where , ϕ_n represent displaced shapes of the vibrating system (*i.e. mode shapes*) which do not change with time t but varies only with the amplitude, and ω_n is the circular frequency.

Substituting the above values in *Eq.12.5.7* we get,

$$m\ (-\phi_n\ \omega_n^2\ \sin\omega_n t) + k\,(\phi_n \sin\omega_n t) = 0$$

$$[(\ k - \omega_n^2\, m)\ \phi_n\ ]\ \sin\omega_n t = 0 \quad or \quad [\omega_n^2 m]\ [\phi_n] = [k]\ [\phi_n]$$

Therefore , either $\sin\,\omega_n t = 0$ or $[\ k\ \phi_n - \omega_n^2 m\ \phi_n\ ] = 0$

But $\sin\,\omega_n t$ cannot be equal to zero because in that case $u=0$, which means there is no motion.

$$\therefore \quad k\ \phi_n - \omega_n^2\ \ m\ \phi_n = 0$$

$$or \quad k\ \phi_n = \ \omega_n^2\ \ m\ \phi_n$$

$$or \quad [\,k - \omega_n^2 m]\ \ \phi_n = 0 \qquad \ldots (13.5.8)$$

Again ϕ_n cannot be zero because it leads to trivial solution implying $u = 0$

$\therefore$ for nontrivial solution ,

$$\det|\ k - \omega_n^2 m\,| = 0 \qquad \ldots (13.5.9)$$

This equation is known as *characteristic equation*.

The equation has n real roots for ω_n^2 because mass matrix and stiffness matrix are symmetric and positive definite having strong diagonals.

The solution of *Eq.13.5.8* gives n natural frequencies ω_n ($n = 1\ ,\ 2\ ,\ \ldots N$). These roots of characteristic equations are called as *eigenvalues*. Once the natural frequency ω_n is known *Eq.13.5.6* can be solved for corresponding vectors ϕ_n which are called *eigenvectors*. Vectors ϕ_n represent natural mode shapes of vibration. It is not the absolute amplitude but the relative values of shape of the vector given by relative values of n displacements ϕ_{jn} ($j = 1, 2, 3, \ldots n$). The subscript denotes the mode number and the first mode (n=1) is called *fundamental mode* corresponding to ω_1 ($\omega_1 < \omega_2 < \omega_3$ *etc.*) .

Thus, for a *two storey frame* shown in *Fig.13.5.1* , *Eq. 13.5.8* can be written using *Eq.13.5.6* as :

$$\begin{bmatrix} k_1+k_2 & -k_2 \\ -k_2 & k_2 \end{bmatrix} \begin{Bmatrix} \phi_{11} \\ \hline \phi_{21} \end{Bmatrix} - \omega^2 \begin{bmatrix} m_1 & 0 \\ 0 & m_2 \end{bmatrix} \begin{Bmatrix} \phi_{11} \\ \hline \phi_{21} \end{Bmatrix} = 0$$

$$\begin{bmatrix} k_1+k_2-\omega^2 m_1 & -k_2 \\ -k_2 & k_2-\omega^2 m_2 \end{bmatrix} \begin{Bmatrix} \phi_{11} \\ \hline \phi_{21} \end{Bmatrix} = 0 \qquad \ldots (13.5.10)$$

And for nontrivial solution :

$$det \begin{vmatrix} k_1+k_2-\omega^2 m_1 & -k_2 \\ -k_2 & k_2-\omega^2 m_2 \end{vmatrix} = 0 \qquad \ldots (13.5.11)$$

The solution will give values of ω from which natural period of vibration and mode shapes can be obtained and further dynamic analysis can be carried out.

13.6 RESONANCE

The earthquake motion can be considered to consist of combination of harmonic motions of different frequencies and of different magnitude. When any of the frequencies in the ground motion matches with that of natural frequency of the structure resonance occurs (*i.e.* manifold increase in response) leading to complete collapse. For resonance to build up sustained harmonic force is required but fotunately such sustained harmonic excitations do not occur. Therefore, the complete collapse of the structure is a very rare but significant damages occur when ground motion has reasonable amount of energy associated with the frequencies closure to its natural frequency.

13.7 STRUCTURAL RESPONSE TO EARTHQUAKE

13.7.1 Introduction

During earthquake ground vibrates in all directions but the horizontal component of the ground motion mainly causes the damages to the structures. This is because the structure does not move along ground motion but responses it thereby stresses and strains are induced in the structure.

13.7.2. Structural Response

The response of the structure to the ground motion depends on the following factors:

1. Acceleration to which it is subjected. This in turn depends on frequency *or* time period of vibration and damping ratio.
2. Materials, form, size and mode of construction of the structure.
3. Soil - structure interaction
4. Post - yeild behaviour of the structure.

13.8 FACTORS GOVERNING SEISMIC DESIGN

The parameters which charaterizes the seismic design are as under:

1. Design load
2. Seismic weight
3. Zone factor.
4. Importance factor.
5 Response reduction factor.
6. Design acceleration spectrum.

13.8.1 Design Loads

The earthquake being a rare phenomenon in which a structure is mainly subjected to horizontal forces, it is unlikly (very less probability) that both wind and earthquake will occur simutaneously. Therefore, load factors specified in the code are to be used either for wind load *or* for earthquake load. The various load combinations specified in IS :1893 *clause 6.3* shall be considered for regorous design. For various loading combinations specified in the code, the earthquake force shall be calculated for the full dead load plus percentage of live load given in *Table 13.8.1*. But where the probable loads at the time of earthquake are more accurately assessed, the designer may alter the proportions *or* even replace the imposed load proportions by actual assessed load. Lateral design force for earthquake shall not be calculated on contribution of impact effects from imposed loads.

Table 13.8.1 Percentage of Imposed Load to be Considered in Seismic Weight Calculations	
Imposed Uniformly Distributed Floor Loads kN/m^2	*Percentage of Imposed Load*
Up to and including 3 kN/m^2	25%
Above 3 kN/m^2	50%

The percentage of imposed load be estimated using full dead load plus % of imposed load as specified by IS: 1893, clause. 7.3.1 and is given in *Table 13.8.1*.

In using working stress method the permissible stresses in material shall be increased by one - third limiting it to yield stress of steel. Depending upon type of foundation of the structure and the type of soil the allowable bearing pressure in soil shall be increased as specified in *Table -1 of IS:1893,clause.6.3.5.2.* The classification of the soil is based on the average shear wave velocity for top 30*m* of rock / soil layers *or* based on average standard penetration test *(N)* values for top 30*m*.

13.8.2 Seismic Weight

Since the seismic force is due to inertia of mass, the imposed load which includes impact effects does not contribute fully to the seismic load. In addition to this the probability that the building will be loaded to its full design load during earthquake is rare. These facts are taken into consideration and the code incorporates only part of the loads to be taken under earthquake. Since the reduction of load has already been made no further reduction in imposed load shall be made as specified in *Sect. 3.2.1.2 of IS : 875* (part 2).

Thus, the seismic weight of each floor consists of full dead plus appropriate amount of imposed load specified above while computing the seismic weight of each floor, the weight of columns and walls in each storey shall be equally distributed to floors above and below the storey.

The seismic weight of the whole building is the sum of the seismic weight of all the floors. Any weight supported in between storeys shall be distributed to the floors above and below in inverse proportion to its distance from the floors.

13.8.3 Zone factor *(IS:1893, clause.6.4.2)*

Earthquake severity has been classified into zones on the basis of maximum ground acceleration based on past earthquake data. India has been divided into four seismic zones (shown in *Fig. 13.8.1*) for the Maximum Considered Earthquake *(MCE)* and service life of the structure in a zone .

The map is based on expected intensity of ground shaking but does not consider the frequency of .occurrence. In seismic zone map, zone-I and zone - II of the contemporary map have been merged and assigned the level of zone-II

Zone II has lowest danger *or* risk while *Zone - V* has highest hazards.

The zone factors are given in *Table 13.8.2*

Table 13.8.2 Zone Factor , z				
Seismic Zone	*II*	*III*	*IV*	*V*
Seismic intensity	Low	Moderate	Severe	Very severe
z	0.10	0.16	0.24	0.36

Since damage controlled limit state method has been accepted, the zone factor, *z* has been reduce to half ***(z/2)*** of Maximum Considered Earthquake *(MCE)* for Design Basis Earthquake *(DBE)*. Structures are explicitly designed for *DBE* and maximum considered earthquake is taken care of through over strength and *ductility* provisions.

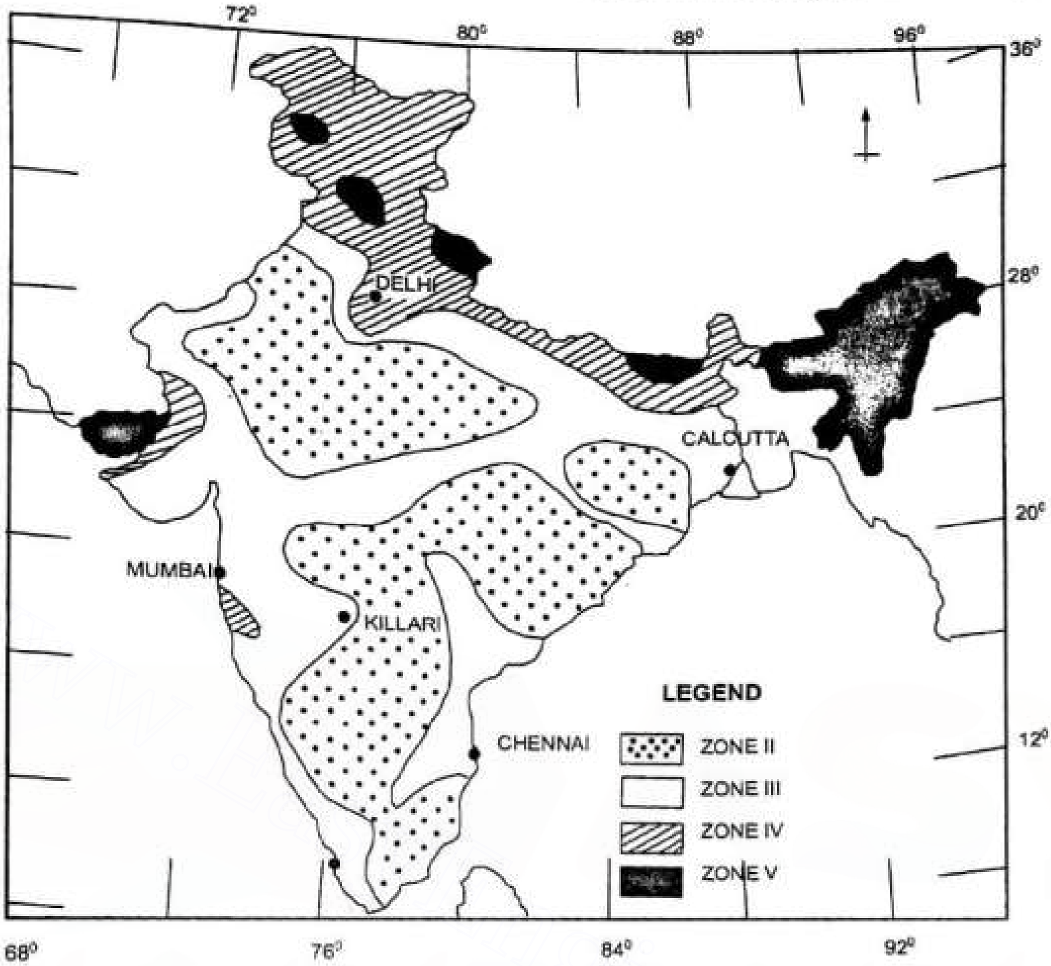


Fig. 13.8.1 Seismic Zone Map

13.8.4 Importance of Structure *(IS:1893, clause.7.2.3)*

Importance of a structure depends upon the functional use of the structures, characterized by hazardous consequence of its failure, post- earthquake functional needs, historical value, *or* economic importance. In estimating lateral force V_B of buildings , the importance factor I of building shall be taken as per *Table 13.8.3*

Table 13.8.3 Importance Factor , I

Sr. No.	*Structure*	*Importance Factor I*
1	Important service and community buildings, or structures,(for example, critical governance buildings, schools), signature buildings, monument buildings, life line and emergency buildings (for example, hospital buildings, telephone exchange buildings, television stations , bus station buildings, metrorail and metro station buildings),railway station , airports, food storage buildings (such as warehouses, fuel stations buildings, and fire station buildings), large community hall buildings, (for example cinema halls, shopping malls, assembly halls and subway stations).	1.5
2.	Residential or commercial buildings excepting the one stated above	1.2
3.	All other Buildings	1.0

13.8.5 Response Reduction Factor R (IS:1893, clause.7.2.6)

The response reduction factor takes into account the *ductility* of the structural system, and *over strength* so that the structure can be designed to the level of yield force of the structure and rely on the nonlinear response of the structures in the case of severe earthquake. It is, therefore, obvious that structure having low over strength *or* low ductility should be designed for higher seismic coefficients. This means that design seismic coefficients for buildings such as steel stacks, overhead water tanks etc. should be designed for higher design seismic coefficient than that for the buildings.

If the importance factor, *I* taken greater than 1.5 (see *Note - l* and *2* in *Table 13.8.3*) and lower value of response reduction factor ,*R*, then *I/R* will be greater than unity. In such a case the base shear to which structure will be subjected will be very large. Therefore, the code restricts the ratio *I/R* to unity However, for building frame system the maximum value of *I* in *Table 13.8.3* is 1.5 and the lowest value of *R is 1.0. Therefore the ratio I/R* will not exceed unity. The response reduction factor is also important from viewpoint of discouraging the construction of ordinary moment resisting frames and ordinary reinforced concrete shear wall, by considering higher level forces in the design of earthquake resisting structures.

The response reduction factors are given in *Table 13.8.4*

Table 13.8.4 Response Reduction Factor, R for Building Frame Systems

Sr. No.	Lateral Load Resisting System	R
i)	**Moment Frame System**	
	(a) RC buildings with ordinary moment resisting frame (OMRF)	3
	(b) RC buildings with special moment resisting frame (SMRF)	5
	(c) Steel buildings with ordinary moment resisting frame (OMRF)	3
	(d) Steel buildings with special moment resisting frame (SMRF)	5
ii)	**Braced Frame Systems**	
	(a) Buildings with ordinary braced frame (OBF)	4
	(b) Buildings with special braced frame (SBF)	4.5
	(c) Buildings with special braced frame having eccentrc braces(SBF)	5
iii)	**Structural Wall System (Note-3)**	
	(a) Load bearing masonry buildings	
	1) Unreinforced masonry without horizontal seismic bonds(Note-1)	1.5
	2) Unreinforced masonry with horizontal seismic bonds	2.0
	3)Unreinforced masonry with horizontal seismic bonds and vertical reinforcing bars at corners of rooms and jambs of openings	2.5
	4) Reinfoedced masaonry	3.0
	5) Cofined masonry	3.0
	(b) Buildings with ordinary RC structural walls (Note-1)	3.0
	(c) Buildings with ductile RC structural walls	4.0
iv)	**Duel systems (Note-3)**	
	(a)Buildings with ordinary RC structural walls and RC with OMRFs (Note-1)	3.0
	(b)Buildings with ordinary RC structural walls and RC with SMRFs (Note-1)	4.0
	(c)Buildings with ductile RC structural walls and RC with OMRFs (Note-1)	4.0
	(d)Buildings with ductile RC structural walls and RC with SMRFs (Note-1)	5.0
v)	**Flat Slab - Structural Wall System** (Note-4)	3.

Notes: 1) RC and steel structures in seismic zone III,IV,and V shall be designed to be duuctile. Hence , this system is not allowed in these sysmic zones.

2) Eccentric braces shall be used only with SBFs.

3) Buildings with structural wqalls also include buildings having structural walls and moment frames, but where

(a) frames are not designed to carry design lateral loads, or

(b) frames are designed to carry design design loads, but do not fulfillthe requirment of s of Duel Systems.

4) In these buildings, (a) punching shear failure shall be avoided, and

(b) lateral drift at the roof under design lateral force shall not exceed 0.1%

13.8.6 Design Acceleration Spectrum (IS:1893, clause.6.4)

For the purpose of determinimg design seismic force, the country is classified into four seismic zones as shown in Fig.13.8.1

The design horizontal seismic coefficient A_h for a structure shall be determined by:

$$A_h = \frac{\frac{z}{2} \times \frac{S_a}{g}}{\frac{R}{I}} \qquad (13.8.1)$$

where, z = seismic zone factor.

I = importance factor given in IS: 1893, when not specified, the minimum value of I shall be taken.
(a) 1.5 for crtical and life line structures
(b) 1.2 for business continuity structures; and
(c) 1.0 for rest

R = response reduction factor given in IS: 1893 for corresponding structure.

S_a/g = design acceleration coefficient for different types of soils, nomalizeed with peak ground acceleration, corresponding natural period T of structure given in *Table.13.8.5.*

Table 13.8.5 Design Acceleration Coefficients S_a/g

(a) For Use in Equivalent Static Method

	For rocky hard soil		For medium stiff soil		For soft soil	
$S_a/g=$	2.5	0.00 < T < 0.40 s	2.5	0.00 < T < 0.55 s	2.5	0.00 < T < 0.67 s
	1/T	0.40 s < T < 4.00 s	1.36/T	0.55 s < T < 4.00 s	1.67/T	0.67 s < T < 4.00 s
	0.25	T > 4.00 s	0.34	T > 4.00 s	0.42	T > 4.00 s

(b) For Use in Response Spectrum Method

	For rocky hard soil		For medium stiff soil		For soft soil	
$S_a/g=$	1+15T	T < 0.10 s	1+15T	T < 0.10 s	1+15T	T < 0.10 s
	2.5	0.10 s < T < 0.40 s	2.5	0.10 s < T < 0.55 s	2.5	0.10 s < T < 0.67 s
	1/T	0.4 s < T < 4.00 s	1.36/T	0.55 s <T < 4.00 s	1.67/T	0.67 s < T < 4.00 s
	0.25	T > 4.00 s	0.34	T > 4.00 s	0.42	T > 4.00 s

13.8.7 Over Strength

The additional strength over the design force incorporated in design codes is called over - strength. This is due to ignorance about the actual behavior of the structure subjected to lateral forces.

The factors that contribute to over - strength are :

1. Partial safety factors applied on material properties.
2. Load factors used for design forces.
3. The contribution of non - structural elements neglected.
4. The reserve strength of redundant members not taken into account.
5. Larger sizes of members and more reinforcement provided than required as per design.

13.8.8 Ductility

Ductility of a structure *or* its member is the capacity to undergo large inelastic deformations without significant loss of strength *or* stiffness. It can also be defined as the ratio of maximum displacement (δ_{max}) at ultimate strength to displacement at yield (d_y). The measure of the structured ductility is the ductility factor defined as $\upsilon = \delta_{max} / \delta_y$. Usually displacements are measured at roof level.

Main important factors affecting ductility are :

1. Axial load in members reduces ductility at column ends.
   A structure with a strong column and weak beam improves ductility.
2. Flexural members exhibit large ductility before collapse if failure is initiated by steel.
   Thus, ductility can be improved by providing under - reinforced flexural members.
3. Avoid failure of a member in diagonal shear.
4. Crushing strain in concrete can be improved considerably by confining the concrete by providing closely spaced stirrups.
5. Curvature ductility increases with increase in compression steel.
6. High strength concrete is less ductile. Therefore, as far as possible very high strength concrete should not be selected for earthquake resistant structures.
   The requirements for ductile detailing are given in *Sect. 13.12*.

13.8.9 Soft Storey

Soft storey *or* flexible storey is one in which the lateral stiffness is less than 70% of that in the storey above or less than 80% of the average lateral stiffness of the three storeys above.

In case of buildings with a flexible storey, such as ground storey consisting of open spaces for parking *i.e.* stilt buildings, special arrangement needs to be made to increase the lateral strength and stiffness of the soft/open storey.

For such buildings dynamic analysis is carried out including the strength and stiffness effects of infills and inelastic deformations in the members, particularly, those in the soft storey and the members designed accordingly.

Alternatively, the following design criteria are to be adopted after carrying out the earthquake analysis, neglecting the effect of infill walls in other storeys:

a) the columns and beams of the soft storey are to be designed for 2.5 times the storey shear and moments calculated under seismic loads specified in the other relevant clauses, *or*

(b) besides the columns designed and detailed for calculated storey shears and moments, shear walls placed symmetrically in both directions of the building as far away from the centre of the building as feasible: to be designed exclusively for 1.5 times the lateral storey shear calculated as before.

13.8.10 Drift

Drift is the maximum lateral displacement of the structure with respect to total height *or* relative inter-storey displacement. The overall drifts index is the ratio of maximum roof displacement to the height of the structure, and inter-storey drift is the ratio of maximum difference of lateral displacements at top and bottom of the storey divided by the storey height.

Non structural elements and structural non-seismic members primarily get damaged due to drift. Higher the lateral stiffness lesser is the likely damage. The storey drift in any storey due to minimum specified design lateral force, with partial safety factor of unity, shall not exceed 0.004 times the storey height.

Separation Between Adjacent Units or Buildings

Two adjacent buildings, *or* two adjacent units of the same building with separation joint in between shall be separated by a distance equal to the amount *R* times the sum of the calculated storey displacements as specified above of each of them, to avoid damaging contact when the two units deflect towards each other. When the floor levels of two similar adjacent units *or* buildings are at the same elevation levels, factor *R* in this case may be replaced by *R/2*.

13.8.11 Foundations

The use of foundations vulnerable to significant differential settlement due to ground shaking shall be avoided for structures in seismic *Zones III, IV, V*. In seismic *Zone IV* and *V*, individual spread footings *or* pile caps shall be interconnected with ties, except when individual spread footings are directly supported on rock. All ties shall be capable of carrying, in tension and in compression, an axial force equal to $A_h/4$ times the larger of the column *or* pile cap load, in addition to the otherwise computed forces, where A_h is design horizontal spectrum value.

13.8.12 Projections

a) Vertical Projections

Tanks, tower, parapets, smoke stacks (chimneys) and other vertical cantilever projections attached to buildings and projecting above the roof, shall be designed and checked for stability for five times the design horizontal seismic coefficient A_h. In the analysis of the building, the weight of these projecting elements will be lumped with the roof weight.

a) the columns and beams of the soft storey are to be designed for 2.5 times the storey shear and moments calculated under seismic loads specified in the other relevant clauses, *or*

b) Horizontal Projection

All horizontal projections like cornices and balconies shall be designed and checked for stability for five times the design vertical coefficient equal to 10/3 A_h. These increased designed forces either for vertical projection or horizontal projection are only for designing the projecting parts and their connection with the main structures. This means that for the design of main structure, such increase need not be considered.

13.9 METHODS OF ANALYSIS

The effects of design earthquake loads as applied to structure can be considered by two methods as:

(a) Equivalent static method or Seismic coefficient method.

(b) Dynamic analysis method.

13.9.1 Equivalent static method or Seismic Coefficient Method

The method is based of static approach normally referred to as pseudo static approach employing use of seismic coefficients. It may be used for regular structure.

The assumptions involved in the method are :

a) Major contribution made to base shear is by fundamental mode of the building.

b) The total building mass is considered as against the model mass used in dynamic analysis.

In this method the total design lateral force *or* seismic is determined by equation :

Seismic base shear $V_B = A_h \times W$ ... (13.9.1)

where, A_h = Design horizontal spectrum value using fundamental natural period in the considered direction of vibration.

$$A_h = \frac{\frac{z}{2} \times \frac{S_a}{g}}{\frac{R}{I}} \qquad \textit{(Eq.13.8.1)}$$

where,
z = Zone factor (*Table 13.8.2*).
I = Importance factors *(Table 13.8.3)*.
R = Response reduction factor *(Table 13.8.4)*.
S_a/g = Average acceleration response coefficient for approximate, natural period of vibration T_a to be determined as per *(Table 13.8.5)*
W = seismic weight of building (see *Sect.13.8.2*).

The, seismic coefficient method does not need theoretical concepts of structural dynamics and modal analysis.

The lateral distribution of the base shear to different floor levels along the height of the building is giving by :

$$Q_i = V_B \times \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2} \qquad \text{... (13.9.2)}$$

where,
Q_i = Design lateral force at floor i ,
W_i = Seismic weight of floor i ,
h_i = Height of floor measured from the base ,
n = Number of levels at which masses are located.

Once the design lateral force at floor levels are known any method of frame analysis *or* approximate methods such as *Portal method or Cantilever method*, can be used to calculate forces in the member due to earthquake.

13.9.1.1 Fundamental Natural Period .

The approximate fundamental natural period of vibration T_a in seconds of a moment resisting frame building *without brick infill panels* is given by :

T_a = $0.075\, h^{0.75}$ for R.C. frame building.
= $0.085\, h^{0.75}$ for Steel frame building. ...(13.9.3)

where, h = height of building in m.

For other buildings including moment - resisting frame buildings *with brick infill panels*, the fundamental natural period of vibration (T_a) in seconds is given by :

$$T_a = \frac{0.09\,h}{\sqrt{d}} \qquad \text{....(13.9.4)}$$

where, h = height of building in m
d = base dimension of the building at the plinth level in m, along the considered direction of the lateral force.

13.9.2 Dynamic Analysis

Dynamic analysis is carried out by the *Time history method or Response spectrum method.*

13.9.2.1 Response spectrum

Response spectrum of any earthquake ground motion is a plot *of peak (or maximum)* values of response quantities (*viz.* displacement, velocity and acceleration) as a function of the natural vibration period *or* frequency and damping ratio of single degree freedom system *(SDOF)*

The maximum stiffness force to which to the structure is subjected during ground motion depends on maximum displacement response. The maximum displacement called as *Spectral Displacement* S_d of the structure corresponds to a condition of zero kinetic energy and maximum strain energy.

The maximum strain energy given to *SDOF* system can written as :

Maximum strain energy = $E_{max} = 1/2\; k\, S_d^2$

The maximum velocity response is approximated by multiplying the spectral displacement S_d by circular frequency ω .

The maximum kinetic energy is given by :

$$E_{max} = 1/2\; k\, S_d^2 = 1/2\; m\,(\omega\; S_d)^2 = 1/2\; m\; S_{pv}^2$$

where, $S_{pv} = \omega\, S_d$ is called Pseudo Spectral Velocity

A plot of S_{pv} with respect to time T *or* ω is called Pseudo - Velocity Response Spectrum.

The maximum base shear in *SDOF* system can be obtained as :

$$Q_{max} = k\; S_d = m\,\omega^2 S_d \qquad (\text{because } k = m\,\omega^2)$$

$$= m\;(\omega^2 S_d) = m S_{pa} \text{ is called Pseudo Spectral Acceleration.}$$

The Pseudo - spectrum acceleration has the unit as acceleration which when multipled with the mass gives maximum base shear.

The spectral values of S_d , S_{pv} , S_{pa} are related by :

$$S_{pa} = \omega^2\; S_d = \omega\,(\omega\, S_d) = \omega\, S_{pv} = \frac{2\,\pi}{T} S_{pv} = \left(\frac{2\,\pi}{T}\right)^2 S_d$$

They are normally plotted on single graph with log scale on each axis and is called **'tripartite'** log plot.

The response spectra for a single earthquake record is used for analysis but they are not suitable for purpose of design. This is because the information regarding the effect of near and distant earthquake is required in the design response spectra. It also does not account for the inherent variability of earthquake motions with repect to both frequency and amplitudes at a given site.

13.9.2.2 Response Spectrum Method.

Response Spectrum represents maximum resposes of indealised SDF systems of different natural periods but having the same damping under the action of the same earthquake ground motion at their bases. In this method, the maximum modal response is obtained for each mode using response spectrum. The number of modes to be combined in the analysis are such that the sum total of all modes considered is at least 90 % of total seismic mass.

Mode Shapes

Mode shapes are the displacement shapes of a vibrating system corresponding to the natural frequencies.

Under the undamped free vibration of *MDOF* buildings having N degrees of freedom, the building will be vibrate in N modes of vibration.

The equation for multi - degree, undamped free vibration system is given by *Eq. 12.5.7* and is rewritten as :

$$m\,\ddot{u} + k\,u = 0 \quad or \quad [m]\,\{\ddot{u}\} + [k]\,\{u\} = 0 \qquad (Eq.\ 13.5.7)$$

As the mass is assumed to execute simple harmonic motion the solution of *Eq. 12.5.7* is given as :

$$[\,k - \omega_n^{\,2}\,m\,]\ \ \phi_n = 0 \qquad (Eq.\ 13.5.8)$$

for non - trivial solution,

$$\det\ |\ k - \omega_n^{\,2}\,m\,| = 0 \qquad (see\ Eq. 13.5.9)$$

The solution of equation give N roots representing the frequencies of N modes of vibrations.

The storey shear for each mode can be calculated, the details of which are beyond the scope of this book for which specialized literature on Structural Dynamics with application to earthquake may be referred.

13.9.3 Time History Analysis

It is an analysis of the dynamic response of the structure at each increment of time, when its base is subjected to a specific ground motion history. This means the method requires site specific ground motion studies. However, in majority of cases the time history method is not warranted.

13.9.4 Remarks on Selection of Method.

Dynamic analysis using either *Time history method or Response spectrum method* shall be performed for the following buildings.

a) Regular buildings

These buildings having height greater than 40 m in *Zone IV* and *V*, and those greater than 90 m height in *Zone II* and *III*.

b) Irregular buildings

All buildings higher than 12 m in *Zone IV* and *V*, and those greater than 40 m in height *Zone II* and *III*.

However, for *irregular buildings* having height less than 40 m in *Zone II* and *III*, even though not mandatory, dynamic analysis is recommended.

Thus, in general dynamic analysis shall be performed for building in *Zone IV* and *V* while for buildings having height less than 40 m in *Zone II* and *III* seismic coefficient method which is simple in application can be used.

13.10 DESIGN EXAMPLES

13.10.1 Solution Using Seismic Coefficient Method

The plan of the commercial building is shown *Fig. 13.10.1*. It consists of RCC frame infilled with brick masonry. The building is a perfectly symmetrical structure which normally is the case of commercial buildings. Even though perfect symmetry and uniformity are considered to be better planning for seismic resisting structures, it may not be always possible for residential buildings and for some commercial buildings also. In such cases adjust the size of columns and shear walls such that the centre of mass and centre of stiffness come as close as possible to reduce the torsional effect.

Data :

The RCC frames are infilled with brick masonry

Thickness of slab	=	130*mm*
Load due to roof finish	=	2 kN/m^2
Load due to floor finish	=	1 kN/m^2
Thickness of outer walls	=	250*mm* (including plaster)
Thickness of partition walls	=	175*mm* (including plaster)
Imposed Load	=	4 kN/m^2
Size of column at ground level		250*mm* x 400*mm*
Type of Foundation		isolated footing
Soil condition		Hard murum available at depth of 1.5*m* below *G.L.*
Seismic zone		III

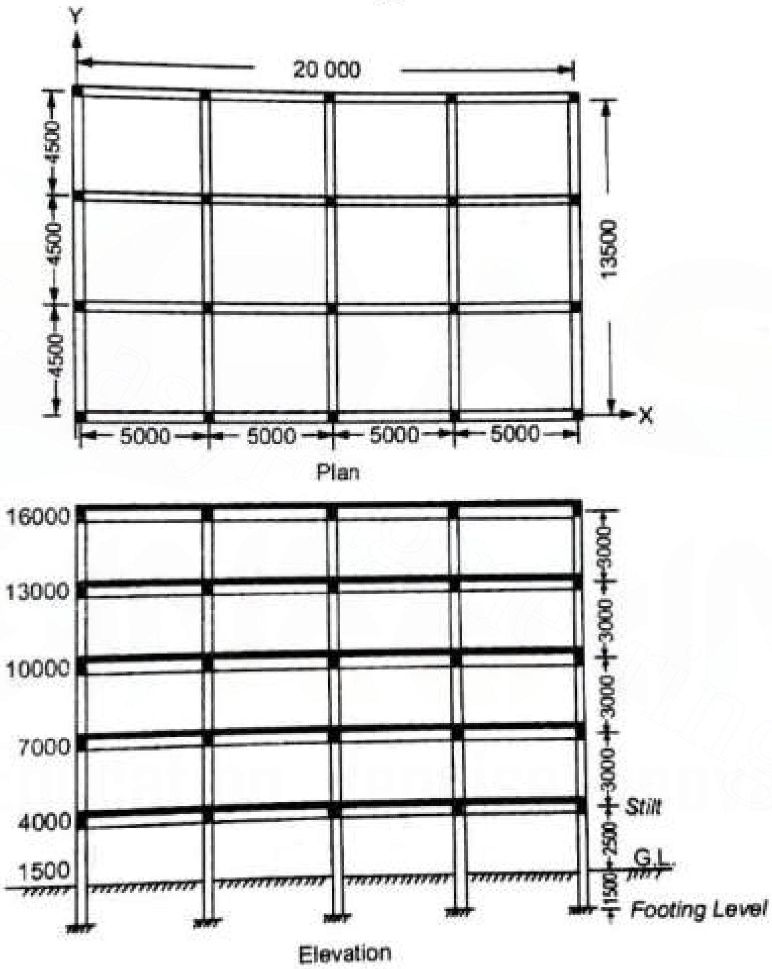


Fig 13.10.1 RCC Frame with Infilled Brick Masonry

Seismic weights

Total floor/ room area $A_f = 20 \times 13.5 = 270 m^2$

At roof, no imposed load to be lumped.

The roof load consists of self weight of slab + 50% load due to weight of wall below the storey

Weight of wall at each floor level :

= Total length of outer walls x thickness x storey height x unit weight of masonry

+ Total length of inner wall x thickness x storey height x unit weight of masonry

$= 2 \times (20 + 13.5) \times 0.25 \times 3 \times 20 + (2 \times 20 + 3 \times 13.5) \times 0.175 \times 3 \times 20$

$= 1005 + 845 = 1850\ kN$

***Note :** Since length of wall is taken equal to centre distance the self weight of the column is not taken. The difference between the unit weight of concrete and unit weight of masonry is neglected.*

∴ Roof load W_4 = (self wt. of slab + *FF*) x A_f + half wall load
= (25 x 0.13 + 2) x 270 + 1850/2 = 2343 *kN*

At floor level, only 50% of imposed load is lumped *(see Table 12.6.1)*

Floor loads = (self wt. of slab + *FF* + 50% *I.L.*) x A_f
= (25 x 0.13 + 1 + 0.5 x 4) x 270 = 1688 *kN*

Wall load = = 1850 *kN*

Assess load at each floor W_3, W_2 *and* W_1 = 1688 + 1850 = 3538 *kN*

Load at plinth W_o = 1850/2 + (25 x 0.23 x 0.45 x20) 4/2 +1688 = 2716 *kN*

Total seismic weight of building = $W=\Sigma W_i$ = 2343 + 3 x 3538 + 1028 = 15673 *kN*

***Note :** Normally water tank is provided over the roof. In such a case additional total due to water tank should be taken over the roof.*

Fundamental Natural Period

Natural period is the time taken (in seconds) by the structure to complete one cycle of oscillation in its natual specified mode of oscillation.

Earthquake load in x - direction:

The structure is considered under the category " (c) All other Buildings,vide *clause* 7.6.2[04] "
As per above mentioned clause, the fundamental natural period (*vide Table. 13.8.5a*) is given by:

$$T_a = \frac{0.09\,h}{\sqrt{d}}$$

where , h = height of building in m = 16m
d = base dimension of the building at plinth in x- direction = 20 m
for T_a = 0.32 sec,average response acceleration coefficient for Type-1 hard soil

$$= \frac{S_a}{g} = 2.5$$

I = importance factor from Table 13.6.3 for buildings = 1.0
R = responce reduction factor from Table 13.6.4
I/R = 1/5 = 0.2 >1
z = zone factor from Table 13.6.2

Design horizontal coefficient

$$A_h = \frac{\frac{z}{2} \times \frac{S_a}{g}}{\frac{R}{I}} \qquad \text{(Eq.13.8.2)}$$

= (0.16/2) x 2.5 x (1/3) (Table.13.8.5a)
= 0.66

Design seismic base shear along x- axis :

$V_B = A_h \times w$ = 0.66 x 15673 = 1044.8 kN

The vertical distribution of base shear to different floor level is given by:

Story level	W_i kN	h_i m	$W_i h_i^2$ kN m²	$\frac{W_i h_i^2}{\sum_{j=1}^{5} W_j h_j^2}$	Lateral force at i^{th} level for EL in x or y direction kN $Q_i = 1044.86 \times \frac{W_i h_i^2}{\Sigma W_j h_j^2}$
4	2343	16	599808	0.339	354.2
3	3538	13	597922	0.338	353.1
2	3538	10	353800	0.200	209.0
1	3538	7	173362	0.098	102.4
Stilt	2716	4	43456	0.025	26.12
	ΣW_i=15673		$\sum_{j=1}^{5} W_j h_j^2$=1768348	1.000	1044.8

Earthquake load in y - direction:

$$T_a = 0.09 \times 16 / \sqrt{13.5} = 0.39$$

T_a = 0.39, average acceleration response coefficient $\frac{S_a}{g}$ *for hard soil* = 2.5.

Importance factor, response reduction factor, zone factor remain the same as obtained for earthquake in *x* - direction. Therefore, for this building the design seismic force in *y*-direction is the same as in *x* - direction.

Fig.13.10.2a and *Fig.13.10.2b* show the design seismic force in *x*-direction and *in y* - direction respectively

Once the distribution of lateral forces to various lateral force resisting elements are determined, any method of frame analysis *or* approximate methods (*viz. Portal* method *or* Cantilever method) can be used to compute forces in members.

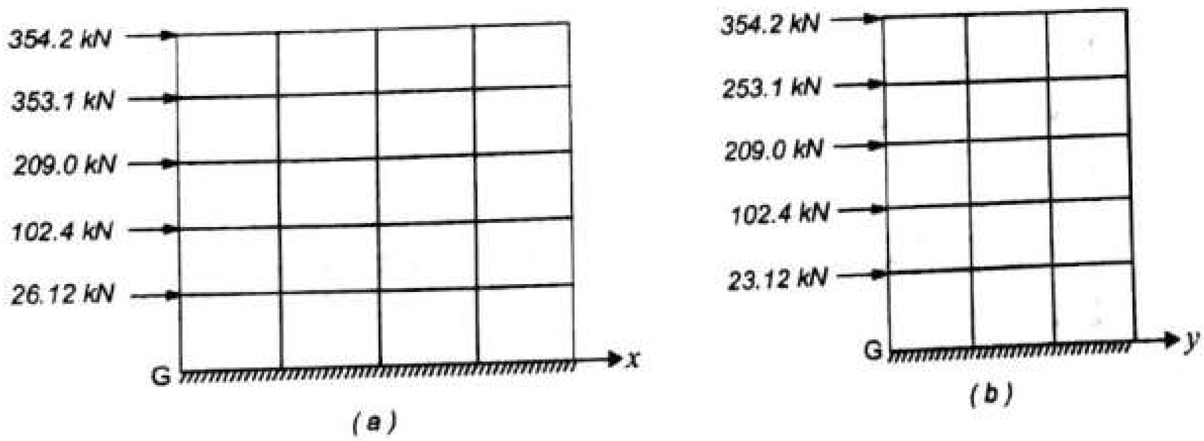


Fig. 13.10.2 Design Seismic Force on Building in (a) *x* - direction (b) *y*- direction

Ex. 13.10.2 A three storey, single bay, commercial building is shown in *Fig. 13.10.3*. It has infilled masonry walls. Calculate the distribution of lateral forces to various lateral force reisting elements.

Seismic Zone		Zone IV (*Table 2 IS: 1893*)		
No. of storeys		Three (G + 2)		
Floor to floor height		3.2 *m*		
Imposed Load :	- Roof slab	1.5 kN/m^2	- Floor slab	4.0 kN/m^2
Floor finish :	- Roof slab	2.0 kN/m^2	- Floor slab	1.0 kN/m^2
Thickness of infilled wall	250 *mm* (including plaster)			
Beam sizes		250mm x 350 *mm*		
Column sizes :	- Ground floor	250mm x 450mm		
	- First floor	250 *mm* x 350 *mm*	- Second floor	250 *mm* x 250 *mm*
Thickness of slab		140 *mm*		
Parapet		1 *m* high and 150 *mm* thick		
Plinth level		1 *m* above ground level (Medium soil condition)		
Depth of foundation for column		1.5 *m* below GL.		
Depth of footing		0.6 *m*		
Depth of foundation for wall		1 *m* below GL..		
Materials		Concrete (M 20), Steel (Fe415)		
Specific weight of concrete		25 kN/m^3		
Specific weight of Masonry		20 kN/m^3		

The tie connecting the footings which is mandatory in Zone IV and V is provided.

Soil type — medium stiff soil

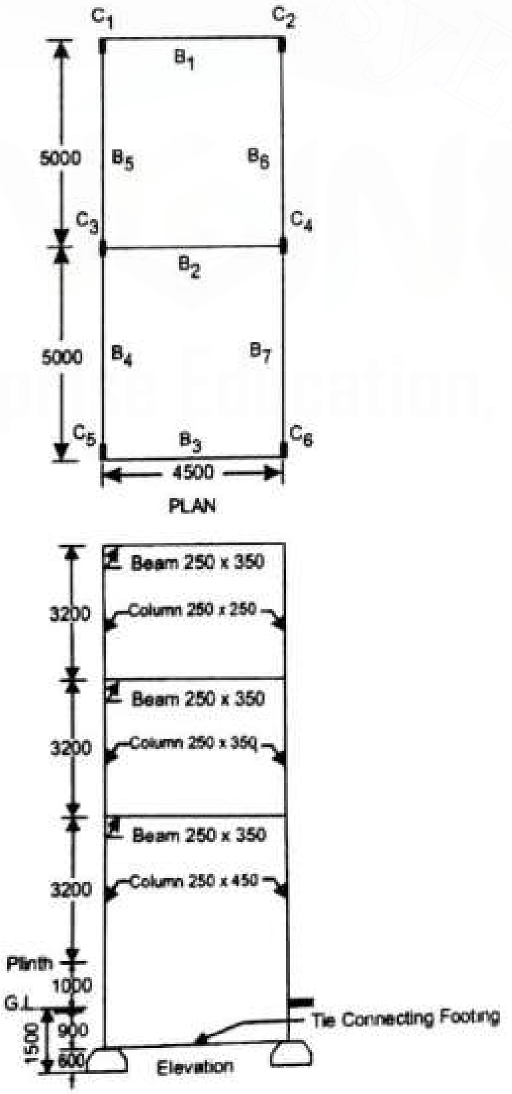


Fig. 13.10.3 **Three Storey, Single Bay, Commercial building**

Remarks : *The example of a single bay, three storey symmetrical building is solved for determination of lateral forces as per IS: 1893 (Part -1) 2016. Both equivalent static method and response spectrum method have been used. The plinth beam has not been provided so as to keep the degree of freedom to three, so that the example can be solved by hand computation. The aim of giving worked out example is to present a clause wise approach for determination of lateral forces as per IS : 1893. Once the detailed procedure is understood it will give insight in the method of design and the programs for multi-storeyed buildings can be prepared.*

Calculation of Seismic Weights at various levels

While calculating the seismic weight of each floor the weight of column and wall in any story is equally distributed to the floors above and below that storey.

Sr. No	Mass of	Calculation	Seismic wt. kN
	ROOF		
1	Slab	(Slab area) x (γ x t +FF+LL) = (10.25x4.75) x (25 x 0.14 + 2 + 0*) = 267.8	268
2	Parapet	(Perimeter) x (γ x t x *height*) = (2 x 14.5) x (25 x 0.15 x 1) = 108.7	108
3	Beam (expect B_2) Beam B_2	(Perimeter) x width x (depth - t) x 25 + width (depth - 0.14) x 25 = (2 x 14.5) x 0.25 x (0.35 - 0.14) x 25 + 4.5 x 0.25 x (0.35 - 0.14) x 25 = 43.9	44
4	Wall	(Perimeter) x γ x t x net height / 2 + length x width x γ x net height/ 2 = (2 x 14.5) x [20 x 0.25 x (3.2/2 - 0.35)] + 4.5 x 0.25 x 20 x (3.2/2 - 0.35) = 209.4	210
		Total Load for Roof =	**630**
		* *Imposed Load on roof is not considered (clause 7.3.2)*	
	II FLOOR		
1	Slab	(10.25 x 4.75) x (25 x 0.14 + 1 + 4/2 *) = 316.47	317
2	Beam (except B_2) Beam B_2	(2 x 14.5) x 0.25 x (0.35 - 0.14) x 25 + 4.5 x 0.25 x (0.35 - 0.14) x 25 = 43.9	44
3	Wall	(2 x 14.5) x [20 x 0.25 x (3.2 - 0.35)] + 4.5 x 20 x 0.25 x (3.2 - 0.35) = 477.4	478
		Total Load for II floor =	**839**
		* *50% of Imposed Load is considered (clause 7.3.2)*	
	I FLOOR		
1	Slab	(10.25 x 4.75) x (25 x 0.14 + 1 + 4/2) = 316.47	317
2	Beam	(2 x 14.5) x 0.25 x (0.35 - 0.14) x 25 + 4.5 x 0.25 x (0.35 - 0.14) x 25 = 43.9	44
3	Wall	(2 x 14.5) x [20 x 0.25 x (4.2 * - 0.35)] + 4.5 x 20 x 0.25 x (4.2 * - 0.35) = 644.87	645
		Total load for I - Floor =	**1006**

Note : Wall foundation is taken 1m below G.L. Hence total height of wall below II floor = 3.2 + 1 +1 = 5.2 m

Therefore, Height of wall for computation of seismic weight = 5.2/2 + 3.2/2 = 4.2 m .

Seismic weight of building W = 630 + 839 - 1006 = 2475 kN

(1) EQUIVALENT STATIC METHOD

Determination of Fundamental Natural Period :

The approximate fundamental natural period of vibration T_a in seconds of a moment resisting frame building without brick infill panels is estimated by formula :

$T_a = 0.075 \times h^{0.75}$, where , h = Total height of the building $= 3.2 + 3.2 + 5.1 = 11.5m$

$= 0.075 \times 11.5^{0.75} = 0.468$ sec. (Assuming that infill panels will not resist any load)

Computation of Design Base Shear: Design base shear $V_b = A_h \times W$ *(Eq. 13.9.1)*

$$A_h = \frac{\frac{Z}{2} \times \frac{S_a}{g}}{\frac{R}{I}} = \frac{\frac{0.24}{2} \times 2.5}{\frac{5}{2}} = 0.09 \qquad (Eq.\ 13.9.2)$$

where , Z = 0.24 for severe seismic intensity (*Table* 13.8.2)

I = Importance factor = 1.5 , for public building (*Table* 13.8.3)

R = Response reduction factor = 5 for ductile detailing (*Table* 13.8.4)

S_a/g = 2.5 for medium soil ($0.1 \le T \le 0.55$) (*Table* 13.8.5)

Design Base Shear $V_B = A_h \times W = 0.09 \times 2475 =$ 222.75 kN

Vertical Distribution of Base Shear to different Floor Levels

$$Q_i = V_B \times \frac{W_i h_i^2}{\sum_{j=1}^{3} W_j h_j^2} \qquad (Eq.\ 13.9.3)$$

where, Q_i = Design lateral force at floor i ,

W_i = Seismic weight of floor i ,

h_i = Height of floor measured from the base ,

n = Number of levels at which masses are located.

Note : *Height of column above the footing = 3.2 + 1 + 1.5 - 0.6 = 5.1 m*

For I-floor h1=5.1 m , For II - floor h2=5.1+3.2 = 8.3 m , For Roof h3= 8.3+3.2 = 11.5

Story	W_i	h_i (m)	$W_i h_i^2$	$W_i h_i^2 / \sum_1^n W_j h_j^2$	Q_i	V_i (kN)
Roof	630	11.5	83317.50	0.4981	110.94	110.94
IInd Floor	839	8.3	57798.71	0.3455	76.96	187.90
Ist Floor	1006	5.1	26166.06	0.1564	32.84	220.74
	2475		167282.27	1.0		

The storey shear, and design load using equivalent static method are shown in *Fig. 12.10.4.*

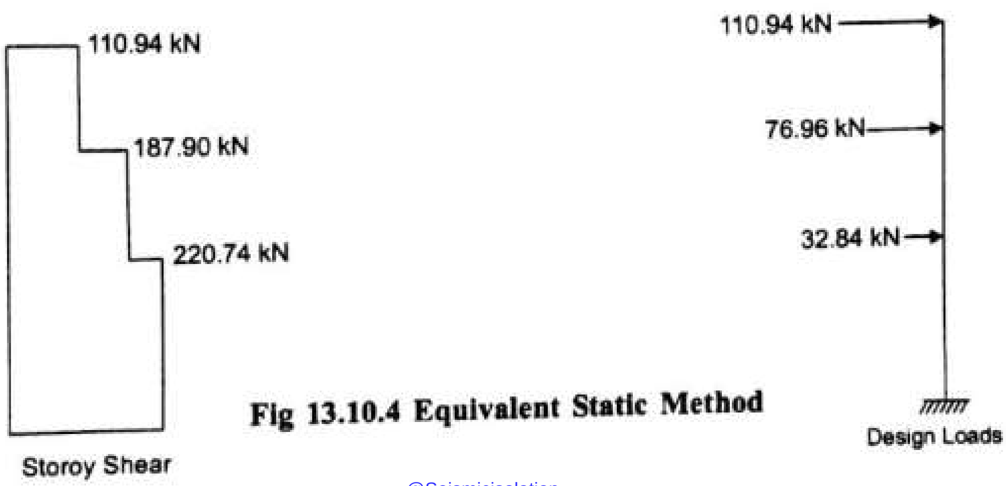


Fig 13.10.4 Equivalent Static Method

13.10.2 Response Spectrum Method

The same problem is solved using dynamic analysis using response spectrum method.

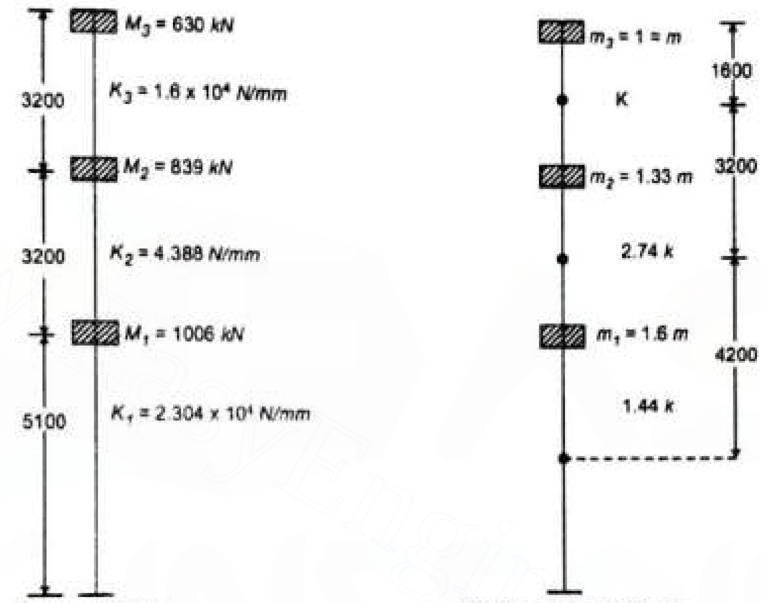
Mass Matrix : The masses at various floor level have already obtained early as :

M_3 = 630 kN , M_2 = 839 kN , M_1 = 1006 kN

To simplify the calculations the relative values of masses are taken.

Assuming , m_3 = 630/630 = 1m , m_2 = 839 / 630 = 1.33m , m_1 = 1006/630 = 1.6 m

The positive definite property of mass is assumed because the lumped masses are non zero in all degrees of freedom restrained in the analysis with zero lumped masses have been eliminated by static condensation. *Fig. 13.10.5* shows column stiffness, relative column stiffness, lumped masses and relative lumped masses.



Column stiffness and Lumped Mass — *Fig. 13.10.5* — Relative Column Stiffness and Lumped Mass

The mass matrix can be written as :

$$\text{Mass Matrix } [M] = \begin{bmatrix} m_1 & 0 & 0 \\ 0 & m_2 & 0 \\ 0 & 0 & m_3 \end{bmatrix} = m \begin{bmatrix} 1.6 & 0 & 0 \\ 0 & 1.33 & 0 \\ 0 & 0 & 1 \end{bmatrix}$$

Stiffness Matrix :

The total number of columns having the same size in each storey = 6

Therefore , $k = 6\,(12 E I / h^3)$

where , $E = 5000\sqrt{f_{ck}} = 5000\sqrt{20} = 2.236 \times 10^4\ N/mm^2$

Column stiffness for *I* - storey $k_1 = 6 \left[\dfrac{12 \times 2.236 \times 10^4 \times (1.898 \times 10^9)}{(5100)^3} \right] = 2.304 \times 10^4\ N/mm$

Column stiffness for *II* - storey $k_2 = 6 \left[\dfrac{12 \times 2.236 \times 10^4 \times (8.932 \times 10^8)}{(3200)^3} \right] = 4.388 \times 10^4\ N/mm$

Column stiffness for *Roof* $k_3 = 6 \left[\dfrac{12 \times 2.236 \times 10^4 \times (3.255 \times 10^8)}{(3200)^3} \right] = 1.6 \times 10^4\ N/mm$

To simplify the computations the relative values of stiffness are taken ,

Assuming $k_3 = 1.6 \times 10^4 = k = 1$,

Thus, $k_1 = 2.304 \times 10^4 / (1.6 \times 10^4) = 1.44\,k$

$k_2 = 4.388 \times 10^4 / (1.6 \times 10^4) = 2.74\,k$ and $k_3 = k$

The stiffness matrix $[k] = \begin{bmatrix} k_1 + k_2 & -k_2 & 0 \\ -k_2 & k_2 + k_3 & -k_3 \\ 0 & -k_3 & k_3 \end{bmatrix} = k \begin{bmatrix} 4.18 & -2.74 & 0 \\ -2.74 & 3.74 & -1 \\ 0 & -1 & 1 \end{bmatrix}$

Mode Shapes Equations (Characteristic Equation)

$$[k - \omega_n^2 . m] . \phi_n = 0 \qquad \text{(Eq. 13.5.8)}$$

The right hand side of the equation is zero.

$$\therefore \text{ either } \phi_n = 0 \text{ or } [k - \omega_n^2 . m] = 0 \qquad \text{(Eq. 13.5.9)}$$

but ϕ_n cannot be zero because it leads to trivial solution implying $u = 0$, which means no motion.

$\therefore$ $[k - \omega_n^2 . m]$ has non trivial solution if $det\,[k - \omega_n^2 . m] = 0$

$$k \begin{bmatrix} 4.18 & -2.74 & 0 \\ -2.74 & 3.74 & -1 \\ 0 & -1 & 1 \end{bmatrix} - m\,\omega_n^2 \begin{bmatrix} 1.6 & 0 & 0 \\ 0 & 1.33 & 0 \\ 0 & 0 & 1 \end{bmatrix} = 0$$

$$\text{or} \quad \begin{bmatrix} 4.18 & -2.74 & 0 \\ -2.74 & 3.74 & -1 \\ 0 & -1 & 1 \end{bmatrix} - \frac{m\,\omega_n^2}{k} \begin{bmatrix} 1.6 & 0 & 0 \\ 0 & 1.33 & 0 \\ 0 & 0 & 1 \end{bmatrix} = 0$$

Let $\dfrac{m.\omega_n^2}{k} = \lambda$ then the above equation can be written as

$$\begin{bmatrix} 4.18 & -2.74 & 0 \\ -2.74 & 3.74 & -1 \\ 0 & -1 & 1 \end{bmatrix} - \begin{bmatrix} 1.6\lambda & 0 & 0 \\ 0 & 1.33\lambda & 0 \\ 0 & 0 & \lambda \end{bmatrix} = 0$$

$$\text{or} \quad \begin{bmatrix} (4.18 - 1.6\lambda) & -2.74 & 0 \\ -2.74 & (3.74 - 1.33\lambda) & -1 \\ 0 & -1 & (1 - \lambda) \end{bmatrix} = 0$$

For non trivial solution $det. \begin{vmatrix} (4.18 - 1.6\lambda) & -2.74 & 0 \\ -2.74 & (3.74 - 1.33\lambda) & -1 \\ 0 & -1 & (1 - \lambda) \end{vmatrix} = 0$

Solving the determinant we get,

$$\lambda^3 - 6.4247\,\lambda^2 + 8.4913\,\lambda - 1.8543 = 0$$

The solution of this cubic equation gives the eigenvalues as under :

$$\lambda_1 = 0.2720 \quad , \quad \lambda_2 = 1.4497 \quad , \quad \lambda_3 = 4.7030$$

$k = 1.60 \times 10^4\ N.mm = 1.6 \times 10^7\ N/m$, $m = 630\ kN = 63000\ kg$

$$\therefore \quad \sqrt{\frac{k}{m}} = \sqrt{\frac{1.6 \times 10^7}{63000}} = 15.94$$

but $\lambda = m\,\omega^2 / k = \omega^2 (m/k)$ $\quad \therefore \omega^2 = \lambda\,(k/m)$ or $\omega = \sqrt{\lambda} \times \sqrt{k/m} = \sqrt{\lambda} \times 15.94$

i) $\omega_1^2 = \lambda_1\,(k/m)$

or $\omega_1 = \sqrt{\lambda_1} \times \sqrt{k/m}$

$= \sqrt{0.2720} \times 15.94 = 8.32$

$\therefore T_1 = 2\pi/\omega_1 = 2\pi / 8.32 = 0.755$ sec.

Similarly

ii) $\omega_2^2 = \lambda_2\,(k/m)$ or $\omega_2 = \sqrt{\lambda_2} \times \sqrt{k/m}$

$\therefore \omega_2 = \sqrt{1.4497} \times 15.94 = 19.19$ radius / sec. $\therefore T_2 = 2\pi/\omega_2 = 2\pi / 19.192 = 0.327$ sec.

iii) $\omega_3^2 = \lambda_3\,(k/m)$ or $\omega_3 = \sqrt{\lambda_3} \times \sqrt{k/m}$

$\therefore \omega_3 = \sqrt{4.7030} \times 15.94 = 34.57$ radius / sec. $\therefore T_3 = 2\pi/\omega_3 = 2\pi / 34.574 = 0.182$ sec.

Mode Shapes Coefficients (ϕ_{ik}) :

where, ϕ_{ik} = mode shape coefficients at floor i in mode k

$$\phi_1 = \begin{Bmatrix} \phi_{11} \\ \phi_{21} \\ \phi_{31} \end{Bmatrix} \quad \text{and} \quad \lambda_1 = 0.2720 \quad \text{as obtained earlier}$$

The characteristic equation is given as :

$$\begin{bmatrix} 4.18 - 1.6\,\lambda_1 & -2.74 & 0 \\ -2.74 & 3.74 - 1.33\,\lambda_1 & -1 \\ 0 & -1 & 1-\lambda_1 \end{bmatrix} \begin{Bmatrix} \phi_{11} \\ \phi_{21} \\ \phi_{31} \end{Bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix}$$

$$\begin{bmatrix} 3.745 & -2.74 & 0 \\ -2.74 & 3.378 & -1 \\ 0 & -1 & 0.728 \end{bmatrix} \begin{Bmatrix} \phi_{11} \\ \phi_{21} \\ \phi_{31} \end{Bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix}$$

For Mode - 1 Let $\phi_{31} = 1$

$\therefore \quad -1\,(\phi_{21}) + 0.728\,\phi_{31} = 0 \qquad \therefore -\phi_{21} + 0.728 = 0 \qquad \therefore\ \phi_{21} = 0.728$

again $\quad -2.74\,\phi_{11} + 3.378 \times (0.728) - 1 = 0 \qquad \therefore\ \phi_{11} = 0.533$

For Mode - 2

$$\phi_2 = \begin{Bmatrix} \phi_{12} \\ \phi_{22} \\ \phi_{32} \end{Bmatrix} \quad \text{and} \quad \lambda_2 = 1.4497$$

The characteristic equation is given as :

$$\begin{bmatrix} 4.18 - 1.6\,\lambda_2 & -2.74 & 0 \\ -2.74 & 3.74 - 1.33\,\lambda_2 & -1 \\ 0 & -1 & 1-\lambda_2 \end{bmatrix} \begin{Bmatrix} \phi_{12} \\ \phi_{22} \\ \phi_{32} \end{Bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix}$$

$$\begin{bmatrix} 1.860 & -2.74 & 0 \\ -2.74 & 1.812 & -1 \\ 0 & -1 & -0.450 \end{bmatrix} \begin{Bmatrix} \phi_{12} \\ \phi_{22} \\ \phi_{32} \end{Bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix}$$

Let $\phi_{32} = 1$

$\therefore \quad -\,(\phi_{22}) - 0.45\,\phi_{32} = 0 \qquad \therefore\ \phi_{22} = -0.45$

again $\quad -\ 2.74\,\phi_{12} + 1.812\,(-0.45) - 1 = 0 \qquad \therefore\ \phi_{12} = -0.663$

For Mode - 3

$$\phi_3 = \begin{Bmatrix} \phi_{13} \\ \phi_{23} \\ \phi_{33} \end{Bmatrix} \quad \text{and} \quad \lambda_3 = 4.7030$$

The characteristic equation is given as :

$$\begin{bmatrix} 4.18 - 1.6\,\lambda_3 & -2.74 & 0 \\ -2.74 & 3.74 - 1.33\,\lambda_3 & -1 \\ 0 & -1 & 1-\lambda_3 \end{bmatrix} \begin{Bmatrix} \phi_{13} \\ \phi_{23} \\ \phi_{33} \end{Bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix}$$

$$\begin{bmatrix} -3.345 & -2.74 & 0 \\ -2.74 & -2.515 & -1 \\ 0 & -1 & -3.703 \end{bmatrix} \begin{Bmatrix} \phi_{13} \\ \phi_{23} \\ \phi_{33} \end{Bmatrix} = \begin{bmatrix} 0 \\ 0 \\ 0 \end{bmatrix}$$

Let $\phi_{33} = 1$

$-(\phi_{23}) - 3.703\ \phi_{33} = 0$ $\quad\therefore\ \phi_{23} = -3.703$

again $\quad -2.74\ \phi_{13}\ -2.515\ \phi_{23} - \phi_{33} \quad = 0$

$-2.74\ \phi_{13} - 2.515 \times (-3.703) - 1 \quad = 0 \quad \therefore\ \phi_{13} = 3.034$

Summary of mode shape coefficients :

$$\phi_1 = \begin{Bmatrix} 0.533 \\ 0.728 \\ 1.0 \end{Bmatrix}, \quad \phi_2 = \begin{Bmatrix} -0.663 \\ -0.450 \\ 1.0 \end{Bmatrix}, \quad \phi_3 = \begin{Bmatrix} 3.034 \\ -3.703 \\ 1.0 \end{Bmatrix}$$

The mode shapes are shown in *Fig 13.10.6*

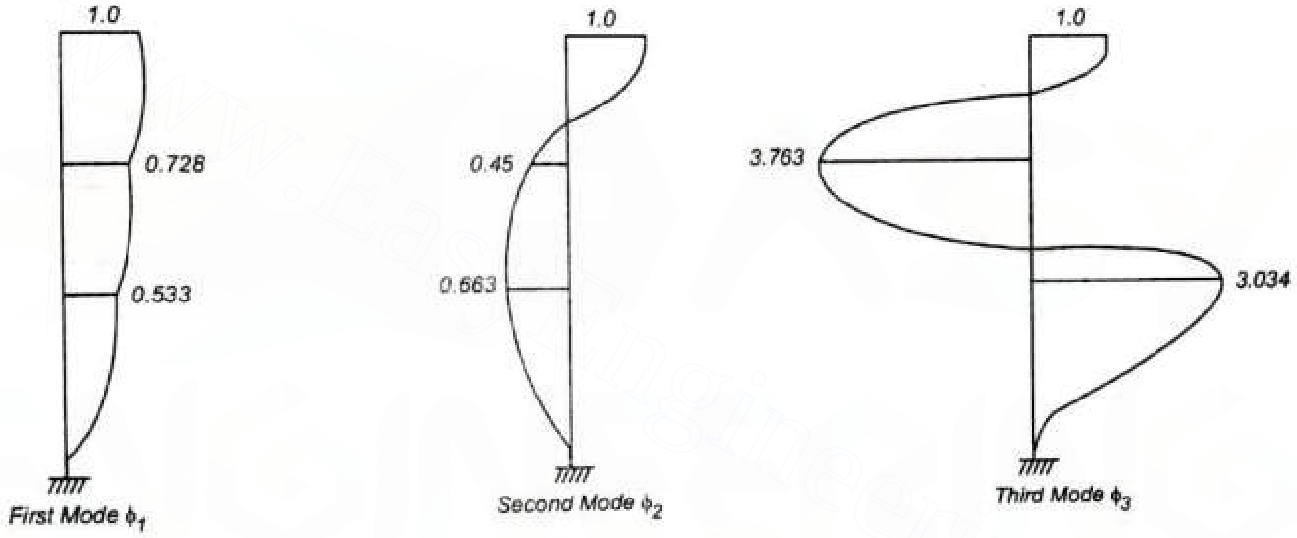


Fig. 13.10.6 Mode Shapes

ω_1 = **8.32 radian /sec.**	ω_2 = **19.192 radians/sec.**	ω_3 = **34.574 radians/sec.**
T_1 = **0.755 sec.**	T_2 = **0.327 sec.**	T_3 = **0.182 sec.**

Vibration properties of the building for vibration in x - direction.

Modal Mass (M_k) : (clause 3.20)

Model mass of a structure subjected to horizontal *or* vertical ground motion is the part of the total seismic mass of the structure that is effective in mode k of vibration. It has a unique value for a given mode irrespective of scaling of the mode shape.

The Model Mass (M_k) of mode k is given by : (clause 7.8.4.5)

$$M_k = \frac{\left[\sum_{i=1}^{n} W_i\,\phi_{ik}\right]^2}{g \sum_{i=1}^{n} W_i\,(\phi_{ik})^2} \qquad \text{... ... (13.10.1)}$$

where, W_i = seismic weight of floor i

ϕ_{ik} = mode shape coefficient at floor i in mode k

g = acceleration due to gravity

Modal Mass M_1 of Mode 1

Storey level	Weight (W_i)	ϕ_{i1}	$W_i\ \phi_{i1}$	$W_i\ \phi_{i1}^2$
Roof	630 kN	1.000	630.00	630.00
II - Floor	839 kN	0.728	610.80	444.66
I - Floor	1006 kN	0.533	536.20	285.80
	$\Sigma Wi = 2475$ kN		$\Sigma W_i\phi_{i1} = 1777.0$	$W_i\ \phi_{i1}^2 = 1360.46$

$$\therefore \quad M_1 = \frac{(\Sigma\ W_i\ \phi_{i1})^2}{g\ \Sigma\ (W_i\ \phi_{i1}^2)} = \frac{(1777)^2}{g\,(1360.46)} = \frac{2321.07}{g}$$

$$M_1 = 2321.07 \times 1000 / 9.81 = 236602.45\ kg \quad or \quad M_1 = 2321.07\ kN$$

Modal Mass M_2 of Mode 2

Storey level	Weight (W_i)	ϕ_{i2}	$W_i\ \phi_{i2}$	$W_i\ \phi_{i2}^2$
Roof	630 kN	1.00	630.00	630.00
II Floor	839 kN	-0.45	-377.55	169.90
I Floor	1006 kN	-0.663	-666.98	442.21
	$\Sigma Wi = 2475$ kN		$\Sigma W_i\phi_{i2} = -414.53$	$W_i\ \phi_{i2}^2 = 1242.11$

$$\therefore \quad M_2 = \frac{(-414.53)^2}{g\,(1242.11)} = \frac{138.34}{g}$$

$$M_2 = (138.34 \times 1000) / 9.81 = 14101.94\ kg \quad or \quad M_2 = 138.34\ kN$$

Modal Mass M_3 of Mode 3

Storey level	Weight (W_i)	ϕ_{i3}	$W_i\ \phi_{i3}$	$W_i\ \phi_{i3}^2$
Roof	630 kN	1.00	630.00	630.00
II Floor	839 kN	-3.703	-3106.82	11504.55
I Floor	1006 kN	3.034	3052.20	9260.39
	$\Sigma Wi = 2475$ kN		$\Sigma W_i\phi_{i3} = 575.38$	$W_i\ \phi_{i3}^2 = 21394.94$

$$\therefore \quad M_3 = \frac{(575.38)^2}{g\,(21394.94)} = \frac{15.47}{g}$$

$$M_3 = 15.47 \times 1000 / 9.81 = 1576.96 \quad or \quad M_3 = 15.47\ \text{kN}$$

Modal Contribution of Various Modes

$$M = 2475\ kN, \quad M_1 = 2321.07\ kN, \quad M_2 = 138.34\ kN, \quad M_3 = 15.47\ kN$$

Modal Contribution Factor :

Modal contribution factors are dimensionless quantities. They are independent of the modes. They are normalized. The sum of modal contribution factors over all modes is unity.

Mode - 1	M_1 / M	= 2321.07 / 2475	= 0.9378	= 93.78%
Mode - 2	M_2 / M	= 138.34 / 2475	= 0.05590	= 5.59%
Mode - 3	M_3 / M	= 15.47 / 2475	= 6.3×10^{-3}	= 0.63%
			Σ = 1.00	Σ = 100%

It will be seen that the modal contribution decreases as mode number increases. Therefore, for practical problems number of modes to be used in the analysis should be such that the sum of modal masses of modes considered is at least 90% of the total seismic mass. Modal contributions should be carried out only for modes up to 33 Hz. The effective mass of all modes = 100%.

Modal Participation Factor (P_k) :

Modal participation factor of mode k of vibration is the amount by which mode k contributes to the overall vibration of the structure under horizontal and vertical earthquake ground motion. It is a measure of degree to which n^{th} mode participates in the response.

$$P_k = \frac{\Sigma (W_i \cdot \phi_{ik})}{\Sigma (W_i \, \phi_{ik}^2)} \qquad \text{... ... (13.10.2)}$$

$$P_1 = 1777 / 1360.46 = \mathbf{1.3062}$$

$$P_2 = -414.53 / 1242.11 = \mathbf{-0.3337}$$

$$P_3 = 575.38 / 21394.94 = \mathbf{0.02689}$$

Design lateral force at each Floor in each mode : (clause 7.8.4.5c)

The peak lateral force Q_{ik} at floor i in mode k is given by :

$$Q_{ik} = A_k \, P_k \, \phi_{ik} \, W_i \qquad \text{... ... (13.10.3)}$$

where A_k = Design horizontal acceleration spectrum value (A_h) using the natural period of vibration (T_k) of mode k.

$$A_h = \frac{Z}{2} \times \frac{I}{R} \times \frac{S_a}{g} \qquad \text{(clause 6.4.2)} \qquad (Eq.\ 13.9.2)$$

For medium soil in zone IV $\quad Z = 0.24$ *(Table 2 of code)*

$I = 1.5$ *(Table 6 of code)*, $\quad R = 5.0$ *(Table 7 (ii) of code)*

Mode - 1 $\quad T_1 = 0.755$ sec.

$S_a/g = 1.36 / T = 1.36 / 0.755 = 1.8013$ *(for 0.55 < = T < = 4.0)*

$$\therefore \quad A_h = \frac{0.24}{2} \times \frac{1.5}{5} \times \frac{S_a}{g} = 0.036 \times (1.8013) = 0.06485$$

$$Q_{i1} = A_{h1} \times P_1 \times \phi_{i1} \times W_i = (0.06485 \times 1.3062) \times \phi_{i1} \times W_i$$

$$= (0.08471) \times \phi_{i1} \times W_i$$

Mode - 2

$T_2 = 0.327$ sec. $S_a/g = 2.5$ (for $0.1 \le T \le 0.55$)

$\therefore A_{h2} = 0.036 \times (2.5) = 0.09$

$Q_{i2} = A_{h2} \times P_2 \times \phi_{i2} \times W_i = (0.09 \times -0.3337) \times \phi_{i2} \times W_i$

$= (-0.030) \times \phi_{i2} \times W_i$

Mode - 3

$T_3 = 0.182$ sec.

$S_a/g = 2.5$ (for $0.1 \le T \le 0.55$)

$\therefore A_{h3} = 0.09$

$Q_{i3} = A_{h3} \times P_3 \times \phi_{i3} \times W_i = (0.09 \times 0.02689) \times \phi_{i3} \times W_i$

$= (2.42 \times 10^{-3}) \ \phi_{i3} \times W_i$

$$V_{ik} = \sum_{j=i+1}^{n} Q_{ik} \quad \text{......(13.10.4)}$$

Calculation of Peak Lateral force Q_i and storey shear for Mode 1

Floor Level	W_i (kN)	$A_h P_k$	ϕ_{i1}	Q_{i1}	V_{i1}
Roof	630	0.08471	1.00	53.37	53.37
IInd Floor	839	0.08471	0.728	51.74	105.11
Ist Floor	1006	0.08471	0.533	45.42	150.53

Calculation of Peak Lateral force Q_i and storey shear for Mode 2

Floor Level	W_i (kN)	$A_h P_k$	ϕ_{i2}	Q_{i2}	V_{i2}
Roof	630	-0.03	1.00	-18.90	-18.90
IInd Floor	839	-0.03	-0.45	11.33	- 7.57
Ist Floor	1006	-0.03	-0.663	20.01	12.44

Calculation of Peak Lateral force Q_i and storey shear for Mode 3

Floor Level	W_i (kN)	$A_h P_k$	ϕ_{i3}	Q_{i3}	V_{i3}
Roof	630	0.00242	1.00	1.52	1.52
IInd Floor	839	0.00242	-3.703	-7.52	-6.00
Ist Floor	1006	0.00242	3.034	7.39	1.39

$\omega_1 = 8.32$ rad/sec , $\omega_2 = 19.19$ rad/sec , $\omega_3 = 34.57$ rad/sec.

Lowest frequency (*i.e. fundamental* frequency) = 8.32 rad/sec.

$0.9\ \omega_1 = 7.49$ rad/sec , $1.1\ \omega_1 = 9.15$ rad/sec.

It will be seen that ω_2 and ω_3 differ from ω_{11} by more than 10%. All the modes are well separated.

The modal combinations are obtained by following methods.

Modal Combinations :

(1) ***Absolute Sum Method (ABS)*** (clause 7.8.4.4)

Assuming that the maximum modal responses attend peak at the same instant of time , then the maximum response quantity is equal to the sum of maximum absolute value of the response associated with each mode . The peak response quantity for the closely spaced modes is given by ;

$$\lambda^{*} = \Sigma \lambda_c \qquad \text{... ... (13.10.5)}$$

where λ^* = peak response quantity for closely spaced modes , λ_c = closely spaced modes.

This upper bound method gives very conservative estimate of maximum response , as the time of occurrence of maximum response can be different .

(2) Square Root of Sum of Squares (SRSS) method :

In general simultaneous occurrence of peak response in all the modes can be different . If the natural frequencies are not very closely spaced then the peak response quantity due to all modes is obtained as :

$$\lambda = \sqrt{\sum_{k=1}^{r} (\lambda_k)^2} \qquad \text{... ... (13.10.6)}$$

where , λ_k = absolute value of quantity in mode k

r = Number modes under consideration

The peak response in each mode is squared , the squared modal peak are summed , and the square root of sum provides an estimate of peak total response with well separated natural frequencies .

(3) Complete Quadratic Combination (CQC) Method

The method (CQC) for modal combination is applicable to a wider class of structures because it overcomes the limitations of SRSS rule .

$$\lambda = \sqrt{\sum_{i=1}^{r} \sum_{j=1}^{r} \lambda_i \, \rho_{ij} \, \lambda_j} \qquad \text{... ... (13.10.7)}$$

where,

r = Number of modes being considered,

ρ_{ij} = Cross-modal coefficient,

λ_i = Response quantity in mode i (including sign)

λ_j = Response quantity in mode j (including sign)

$$\rho_{ij} = \frac{8\zeta^2 (1 + \beta_{ij}) \beta_{ij}^{1.5}}{(1 - \beta_{ij}^2)^2 + 4\zeta^2 \beta_{ij} (1 + \beta_{ij})^2}$$

ζ = Modal damping ratio

β_{ij} = Frequency ratio = ω_j / ω_i

ω_i = Circular frequency in i^{th} mode and

ω_j = Circular frequency in j^{th} mode.

The same problem is solved by all the three methods as under.

(1) Absolute Sum Method (*ABS*) :

V(Roof) = |(53.37)| + |(-18.90)| + |(1.52)| = 73.79 *kN*

V(II - floor) = |(105.11)| + |(-7.57)| + |(-6.0)| = 118.68 *kN*

V(I - floor) = |(150.53)| + |(12.44)| + |(1.39)| = 164.36 *kN*

The storey shear and design loads are shown in *Fig. 13.10.7*

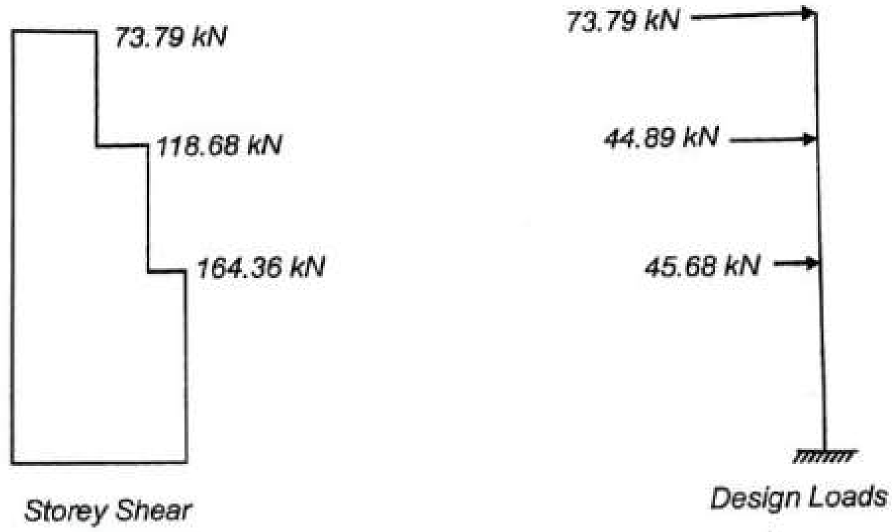


Fig. 13.10.7 Absolute Sum Method (*ABS*)

(2) Square Root of Sum of Squares (*SRSS*) Method :

If the building does not have closely spaced modes, then the peak response quantity (λ) due to all modes considered, shall be obtained as :

$$\lambda = \sqrt{\sum_{k=1}^{r} (\lambda_k)^2} \qquad \text{(Eq. 13.10.6)}$$

Where , r = number of modes under considerations

Peak storey shear forces due to all modes :

$$V\ (Roof) = \sqrt{(53.37)^2 + (-18.90)^2 + (1.52)^2} = 56.64\ kN$$

$$V\ (IInd\ Floor) = \sqrt{(105.11)^2 + (-7.57)^2 + (-6.0)^2} = 105.55\ kN$$

$$V\ (Ist\ Floor) = \sqrt{(150.53)^2 + (12.44)^2 + (1.39)^2} = 151.05\ kN$$

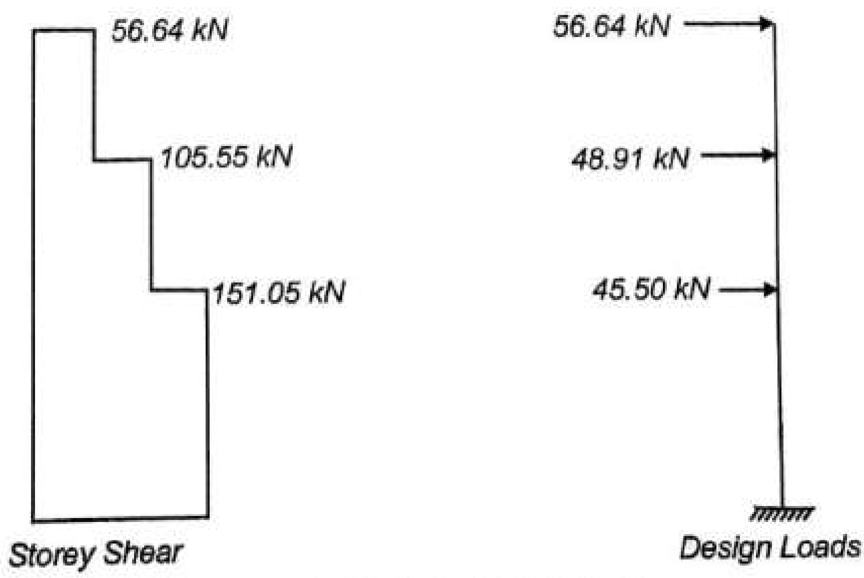


Fig. 13.10.8 SRSS Method

(3) **Complete Quadratic Combination (CQC) :**

$$\lambda = \sum_{i=1}^{r} \sum_{j=1}^{r} \lambda_i \, \rho_{ij} \, \lambda_j \qquad (Eq.\ 13.10.7)$$

$$\rho_{ij} = \frac{8\zeta^2 (1+\beta_{ij})\, \beta_{ij}^{1.5}}{(1-\beta_{ij}^2)^2 + 4\zeta^2 \beta_{ij} (1+\beta_{ij})^2}$$

For 5% damping, $\zeta = 0.05$ $\therefore$ $\rho_{ij} = \dfrac{0.02 \times (1+\beta_{ij})\, \beta_{ij}^{1.5}}{(1-\beta_{ij}^2)^2 + 0.01\, \beta_{ij} (1+\beta_{ij})^2}$

where, ζ_i = modal damping ratio = 5%
ω_i = natural circular frequency in the i[th] mode
ω_j = natural circular frequency in the j[th] mode
β_{ij} = Frequency ratio = ω_j / ω_i

$$\beta_{ij} = \begin{bmatrix} \beta_{11} & \beta_{12} & \beta_{13} \\ \beta_{21} & \beta_{22} & \beta_{23} \\ \beta_{31} & \beta_{32} & \beta_{33} \end{bmatrix} = \begin{bmatrix} \omega_1/\omega_1 & \omega_2/\omega_1 & \omega_3/\omega_1 \\ \omega_1/\omega_2 & \omega_2/\omega_2 & \omega_3/\omega_2 \\ \omega_1/\omega_3 & \omega_2/\omega_3 & \omega_3/\omega_3 \end{bmatrix}$$

where, ω_1 = 8.32 radians/sec , ω_2 = 19.19 radians/sec , ω_3 = 34.57 radians/sec

$$\therefore \beta_{ij} = \begin{bmatrix} 8.32/8.32 & 19.19/8.32 & 34.57/8.32 \\ 8.32/19.19 & 19.19/19.19 & 34.57/19.19 \\ 8.32/34.57 & 19.19/34.57 & 34.57/34.57 \end{bmatrix} = \begin{bmatrix} 1.00 & 2.306 & 4.155 \\ 0.434 & 1.00 & 1.801 \\ 0.241 & 0.555 & 1.00 \end{bmatrix}$$

$$\rho_{11} = \frac{0.02 \times (1+1)\ (1)^{1.5}}{(1-1^2)^2 + 0.01 \times (1) \times (1+1)^2} = 1.0$$

$$\rho_{21} = \frac{0.02 \times (1+0.434) \times (0.434)^{1.5}}{(1-0.434^2)^2 + 0.01 \times (0.434) \times (1+0.434)^2} = 8.2 \times 10^{-3} / 0.6677 = 0.0123$$

$$\rho_{31} = \frac{0.02 \times (1+0.241) \times (0.241)^{1.5}}{(1-0.241^2)^2 + 0.01 \times (0.241) \times (1+0.241)^2} = 2.936 \times 10^{-3} / 0.8909 = 3.3 \times 10^{-3}$$

$$\rho_{12} = \frac{0.02 \times (1+2.306) \times (2.306)^{1.5}}{(1-2.306^2)^2 + 0.01 \times (2.306) \times (1+2.306)^2} = 0.2315 / 18.8940 = 0.0123$$

$$\rho_{22} = \frac{0.02 \times (1+1)\ (1)^{1.5}}{(1-1^2)^2 + 0.01 \times (1) \times (1+1)^2} = 1.0$$

$$\rho_{32} = \frac{0.02 \times (1+0.555) \times (0.555)^{1.5}}{(1-0.555^2)^2 + 0.01 \times (0.555) \times (1+0.555)^2} = 0.01286 / 0.4922 = 0.0261$$

$$\rho_{13} = \frac{0.02 \times (1+4.155) \times (4.155)^{1.5}}{(1-4.155^2)^2 + 0.01 \times (4.155) \times (1+4.155)^2} = 0.8732 / 265.623 = 3.3 \times 10^{-3}$$

$$\rho_{23} = \frac{0.02 \times (1+1.801) \times (1.801)^{1.5}}{(1-1.801^2)^2 + 0.01 \times (1.801) \times (1+1.801)^2} = 0.1354 / 5.175 = 0.0261$$

$$\rho_{33} = \frac{0.02 \times (1+1)\ (1)^{1.5}}{(1-1^2)^2 + 0.01 \times (1) \times (1+1)^2} = 1.0$$

$$\therefore \rho_{ij} = \begin{bmatrix} \rho_{11} & \rho_{12} & \rho_{13} \\ \rho_{21} & \rho_{22} & \rho_{23} \\ \rho_{31} & \rho_{32} & \rho_{33} \end{bmatrix} = \begin{bmatrix} 1.00 & 0.0123 & 3.3 \times 10^{-3} \\ 0.0123 & 1.00 & 0.0261 \\ 3.3 \times 10^{-3} & 0.0261 & 1.00 \end{bmatrix}$$

Here the terms λ_i, λ_j *represent the response of different modes of certain storey level*

$$\begin{Bmatrix} V1 \\ V2 \\ V3 \end{Bmatrix} = \sqrt{\{\lambda_1\ \lambda_2\ \lambda_3\} \begin{bmatrix} \rho_{11} & \rho_{12} & \rho_{13} \\ \rho_2 & \rho_{22} & \rho_{23} \\ \rho_3 & \rho_{32} & \rho_{33} \end{bmatrix} \begin{Bmatrix} \lambda_1 \\ \lambda_2 \\ \lambda_3 \end{Bmatrix}}$$

Using matrix notation , Storey shear for each mode and other storeys are as under :

Mode 1 for roof , Storey Shear = 53.37 *kN*, Mode 2 Storey Shear = -18.90 *kN* , Mode 3 Storey Shear = 1.52 *kN*

$$V\ (Roof) = \sqrt{\{53.37\ \ -18.90\ \ 1.52\} \begin{bmatrix} 1.00 & 0.0123 & 3.3 \times 10^{-3} \\ 0.0123 & 1.00 & 0.0261 \\ 3.3 \times 10^{-3} & 0.0261 & 1.00 \end{bmatrix} \begin{Bmatrix} 53.37 \\ -18.90 \\ 1.52 \end{Bmatrix}} = 56.41\ \text{kN}$$

Storey Shear for II - Floor are as under :

Mode 1 - Storey Shear = 105.11 *kN*, Mode 2 Storey Shear = -7.57 *kN* , Mode 3 Storey Shear = -6.00*kN*

$$V\ (II\text{-}Floor) = \sqrt{\{105.11\ \ -7.57\ \ -6.0\} \begin{bmatrix} 1.00 & 0.0123 & 3.3 \times 10^{-3} \\ 0.0123 & 1.00 & 0.0261 \\ 3.3 \times 10^{-3} & 0.0261 & 1.00 \end{bmatrix} \begin{Bmatrix} 105.11 \\ -7.57 \\ -6.00 \end{Bmatrix}} = 105.45\ \text{kN}$$

Storey Shear for I - Floor are as under :

Mode 1 - Storey Shear = 150.53 *kN*, Mode 2 Storey Shear = 12.44 *kN* , Mode 3 Storey Shear = 1.39 *kN*

$$V\ (I\text{-}Floor) = \sqrt{\{150.53\ \ 12.44\ \ 1.39\} \begin{bmatrix} 1.00 & 0.0123 & 3.3 \times 10^{-3} \\ 0.0123 & 1.00 & 0.0261 \\ 3.3 \times 10^{-3} & 0.0261 & 1.00 \end{bmatrix} \begin{Bmatrix} 150.53 \\ 12.44 \\ 1.39 \end{Bmatrix}} = 151.21\ \text{kN}$$

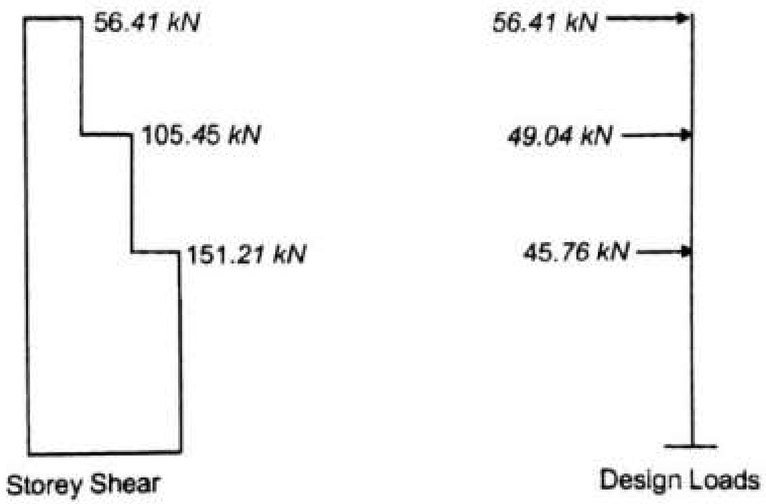


Fig. 13.10.9 CQC Method

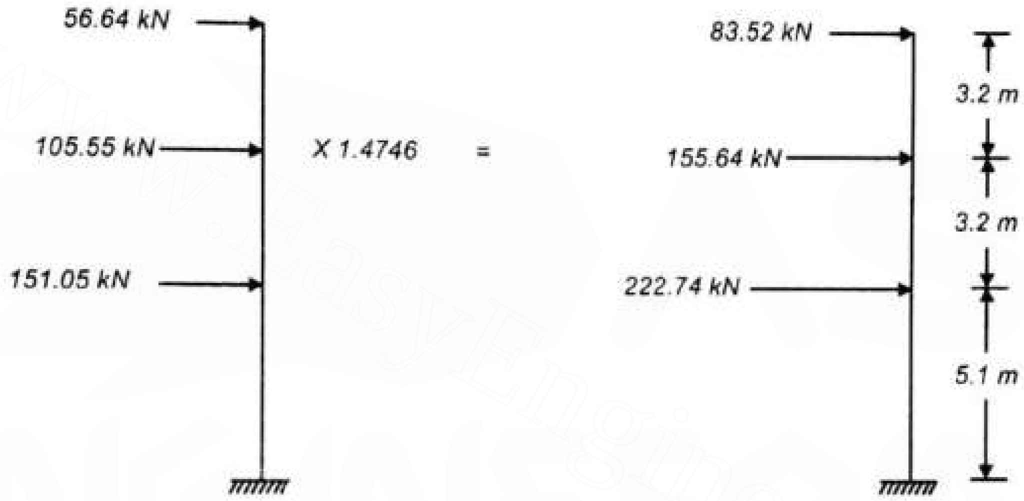
Floor Level	Static	Dynamic		
		ABS	*SRSS*	*CQC*
Roof	110.94	73.79	56.64	56.41
II-Floor	187.90	118.68	105.55	105.45
I -Floor	220.74	164.36	151.05	151.21

Since V_B (=151.05 kN) < $\overline{V}_B = 222.75\ kN$).

The response quantities obtained from SRSS method will be scaled up *i.e.* to be multiplied by $\overline{V}_B / V_B$

∴ Scaling factor = 222.75 / 151.05 = 1.4746

The details are shown in *Fig. 1310.10.*



Storey Shear (SRSS) Method *Scaled Storey Shear at each floor level*

Fig . 13.10.10 Scaled Storey Shear - SRSS Method

Storey Shear for Static and Scaled SRSS dynamic method

Floor Level	Storey Shear *kN*	
	Static (*ABS*)	Dynamic (*SRSS*)
Roof	110.94	83.52
II-Floor	187.90	155.64
I- Floor	222.74	222.74

Concluding Remarks :

Storey shear are significantly affected by change in load distribution. Advantage of dynamic analysis is to reduce storey shear . Seismic analysis carried out by static method gives higher values of storey shear .

13.11 SHEAR WALL

Shear wall is defined as a wall designed to resist lateral forces in its own plane .Shear walls are quite stiff in their own plane and flexible in the perpendicular plane . Therefore , it can transfer the lateral force in its own plane by developing moment and shear resistance . Shear walls increase the stiffness of the building so that the horizontal deflections due to earthquake forces are minimized. The requirements of shear walls have been given in IS: 139320

References :

1. Anil K. Chopra, " Dynamics of Structures Theory and Application to earthquake engineering", Prentice - Hall of India, New Delhi, 2001

2. Joshi, D.S. et al, " Design of Reinforced Concrete Structures for earthquake resistance", Indian Society of Structural Engineers, Dec. 2001

3. Jain, S.K., " Short Courses on Seismic Design of R.C. Buildings", Mumbai, Dec. 1995

4. IS:1893(part - 1) 2016, " Criteria for earthquake resistant design of structures", BIS, New Delhi, 2016

5. IS:13920-2016, " Ductile design and detailing of R.C. structures subjected to seismic forces - code of practice", BIS, New Delhi, 2016

6. Pankaj Agarwal , Manish Shrikhande " Earthquake Resistant Design of Structures ", Prentice-Hall of India , New Delhi , 2007

13.12 DUCTILE DETAILING *(IS:13920)*

13.12.1 Introduction

It has already been mentioned that structures are designed for Design Basis Earthquake *(DBE)*, and maximum considered earthquake is taken care of through ductility provisions. Thus, since the damage controlled limit state has been accepted as the design method the reinforced concrete structure must be detailed properly to have adequate ductility and toughness to resist severe earthquake shocks without collapse. The specifications given in ductile detailing code (IS: 13920) is intended to prevent brittle failure of a structure causing either partial *or* total collapse.

13.12.2 General Specifications.

1. *(a) Grade of Concrete.*

Mininimum grade of concrete shall be M20. Grade of M25 shall be used for buildings > 15 m height in zone III, IV, and V but the grade shall not be less than that required for exposure conditions given in Appendix. C.

(b) Grade of Steel.

Grade of steel tobe used shall be FE415 or less, FE500 and Fe550 al shall confirm to IS:1786. For FE500 and Fe550 produced by thero-mechanical process have elongation > 14.5%. Actual 0.2% proof stress shall not be greater than characteristic proof stress by more than 20%.

2. The flexural members shall have width- to-depth ratio of more than 0.3

$$i.e. \ \frac{b}{D} > 0.3 \quad or \quad D < 3.33\, b$$

3. The width of the beam shall not be less than 200*mm*
   i. e. for minimum width of 200*mm* the depth *D* should be less than 660*mm*

4. The depth *D* of beam shall not be more than 1/4 of clear span.
   i. e. $D >$ clear span /4

5. Width b_w of supporting member + distace on either side os supporting member <= depth of supporting member or 0.75 of width of column.

6. Beams shall have at least 2-12 mm diameter bars each at top ans bottom.

7 Tension steel ratio on any face, at any section, shall not be less than $0.24\sqrt{f_{ck}}/f_y$

$$i.e.\ p_{t.min.} \nless 0.24\sqrt{f_{ck}}/f_y$$

The maximum steel ratio on any face at any section, shall not exceed 0.025

$$i.e.\ p_{t.max} \ngtr 0.025$$

where , $p_t = \dfrac{A_s}{bD}$

Comments :

i) Minimum amount of tension reinforcement is specified in positive and negative moment region to prevent sudden failure.

ii) The maximum steel requirement is intended to avoid steel congestion.

8. The positive steel *at the joint face* must be at least equal to *half* the negative steel provided at that face.

Comments : *(1) This requirement is to cater for the possibility of occurance of positive moment at one end of the beam, due to combination of earthquake induced positive moment and negative moment due to gravity loads on the span as shown in Fig 13.12.1*
(2) It ensures ductile behavior

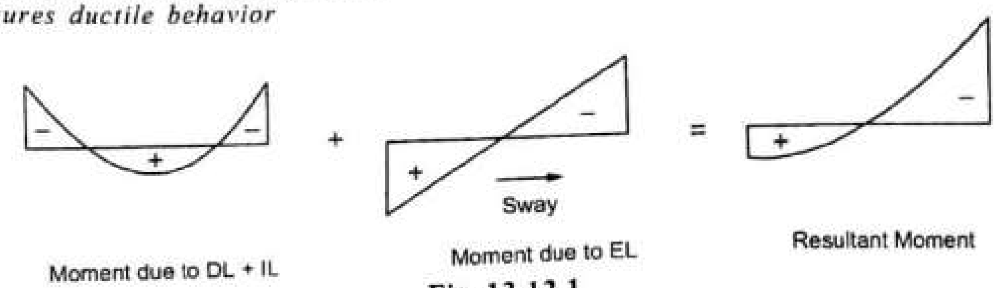
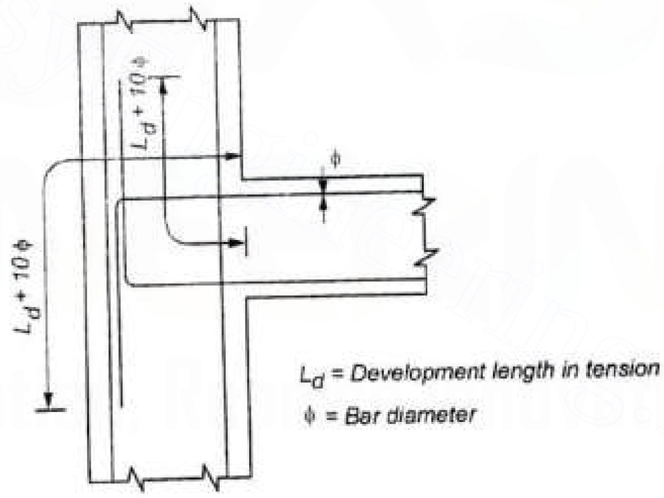


Fig. 13.12.1

9..The steel provided at each of the top and bottom face of the beam *at any section along its length shall* be at least equal to 1/4 of the maximum negative moment steel provided at the face of either joint.

Redistribution of moment shall be carried out only for vertical load moments and NOT for lateral load moments.

10.*In an external joint,* both the top and the bottom bars of the beam shall be provided with anchorage length, beyond the inner face of the column, equal to development length in tension plus 10 times the bar diameter minus the allowance for 90 degree bends *(see. Fig 13.12.2).*



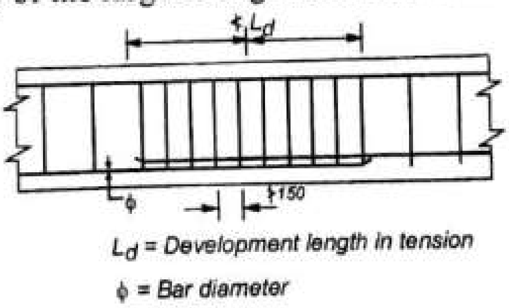
(a) Anchorage of Beam Bars in an External Joint

Fig. 13.12.2

Lap Spices

1. The longitudinal bars shall be spliced, only if hoops are provided over the entire splice length, at a spacing not exceeding 150*mm* (*see Fig. 13.12.3*). The lap length shall not be less than the bar development length L_d of the largest longitudinal reinforcement in tension..

Fig.13.12.3



Lap splices shall **Not** be provided at the following places :

(a). within a joint , (b). within a distance of $2d$ from the face of jointcolumn.

(c). within the quarter length of the member, where flexural yielding may occur under earthquake effects.

2. Not more than 50% of the bars on either top or bottom faces shall be spliced at any one section.

13.12.3 Column and Frame members subjected to Bending and Axial load.

Scope: These requirments apply to frame members which have factored axial stress in excess of $0.1f_{ck}$ under the effect of earthquake forces.

13.12.4 Web Reinforcement

1. Web reinforcement shall consist of vertical hoops.

The basic requirements of transverse reinforcement is :

(a) to resist the shear forces, (b) to confine the concrete,

(c) to give lateral support to the reinforcement in expected yielding regions.

A vertical hoop not less than 8 mm, is a closed vertical stirrup having 135^0 hook with a 6 diameter extension (but >= 65*mm*) at each end that is embedded in the confined core.(*see Fig. 12.12.4a*). In compelling circumstances, it may be made up of two pieces of reinforcement ; a U - stirrup with 135^0 hook and a 8 diameter extension (but >= 65 *mm*) at each end, embedded in the confined core \and a cross-tie (*Fig.12.12.4b*).

A cross-tie is a bar having 135^0 hook with 6 diameter extension (but >= 65*mm*) at each end. The hooks shall engage peripheral longitudinal bars.

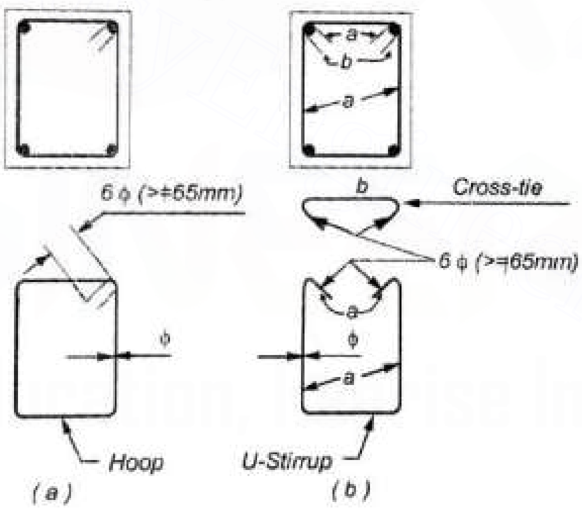


Fig. 13.12.4 Beam Web Reinforcement

2. Only vertical links shall be provided in beams.

The minimum diameter of the bar forming a hoop shall be 8 *mm*.

In addition to this the requirements specified by IS : 456 should be adhered to and they are given as under:.

(a) Spacing of stirrups for design shear reinforcement shall be

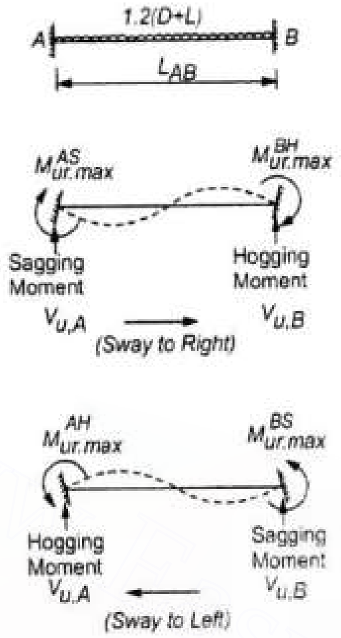
$$s < \frac{0.87\, f_y\, A_{sv}\, d}{V_{usv}} \qquad \text{... ... (Eq. 4.4.5)}$$

(b) Spacing of stirrups for minimum shear reinforcement shall be

$$s < \frac{0.87\, f_y\, A_{sv}}{0.4b} \qquad \text{... ... (Eq. 5.3.11)}$$

(c) Spacing of stirrups shall be < (0.75d *or* 300 *mm*) whichever is less.

3. The shear force to be resisted by vertical hoops shall be maximum of
   (a) ultimate shear force calculated as per analysis, and
   (b) shear force due to formation of plastic hinges at both ends of the beam plus the factored gravity load on the span ; given by *(see Fig. 13.12.5.)*



i) Sway to Right

$$V_A^{D+L} = V_B^{D+L} = \frac{1.2(D+L)}{2}$$

$$V_{uA} = V_A^{D+L} - 1.4\left[\frac{M^{AS}_{ur.max} + M^{BH}_{ur.max}}{L_{AB}}\right]$$

$$V_{uB} = V_B^{D+L} + 1.4\left[\frac{M^{AS}_{ur.max} + M^{BH}_{ur.max}}{L_{AB}}\right]$$

ii) Sway to Left

$$V_{uA} = V_A^{D+L} + 1.4\left[\frac{M^{AH}_{ur.max} + M^{BS}_{ur.max}}{L_{AB}}\right]$$

$$V_{uB} = V_B^{D+L} - 1.4\left[\frac{M^{AH}_{ur.max} + M^{BS}_{ur.max}}{L_{AB}}\right]$$

Fig. 13.12.5 Design Shear Force for Beam

where, $M^{AS}_{ur.\ max}$ = Ultimate Sagging moment of resistance of a balanced section at end *A*

$M^{BH}_{ur.\ max}$ = Ultimate Hogging moment of resistance of a balanced section at end *B*

$M^{BS}_{ur.\ max}$ = Ultimate Sagging moment of resistance of a balanced section at end *B*

$M^{AH}_{ur.\ max}$ = Ultimate Hogging moment of resistance of a balanced section at end *A*

V_{uA}, V_{uB} = Ultimate shear force at end *A* and *B*

V_A^{D-L}, V_B^{D-L} = Shear force at end *A* and *B* due to vertical loads with a partial safety factor of 1.2

L_{AB} = Clear span of the beam

It is advisable NOT to provide much more quantity of reinforcement than required in beams because it may cause brittle failure. On the contrary provide slightly less steel to ensure ductile flexural failure.

4. The contribution of bent up bars and inclined hoops in R.C. sction to shear resistance of the section shall not be considered.

During a severe earthquake, the direction of the shear force may reverse. Bent up bars and inclined stirrups are effective in resisting shear in one direction only. Therefore, their contribution to shear resistance of the section shall not be considered.

5. The spacing of the stirrups (*or* hoops) over a length of 2*d* at either end of the beam shall not exceed,
   (a) *d*/4 and,
   (b) 8 times the diameter (ϕ) of the smallest longitudinal bar,
   (c) 100 *mm*.
   This restriction is required to prevent the occurrence of flexural yielding in this region at beam ends under the effect of earthquake forces.
6. In the remain portion (*i.e. L*–2 x 2*d*) spacing of the hoops shall not exceed *d*/2 .
7. The first hoop shall be at a distance not exceeding 50 *mm* from the face of the joint.

The special confining reinforcement required for beam reinforcement is shown in *Fig. 13.12.6*

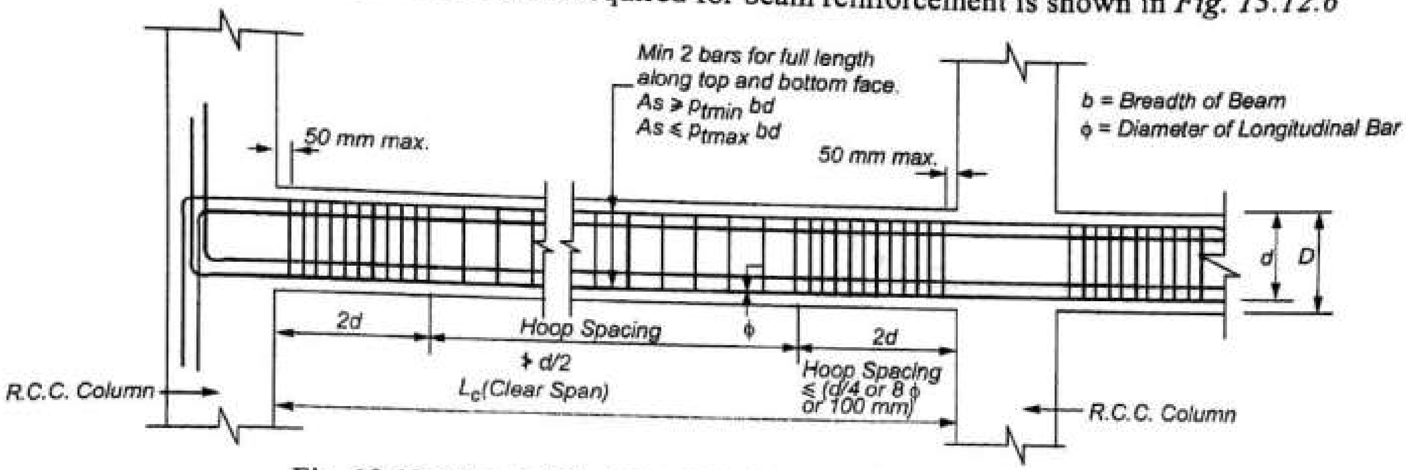


Fig. 13.12.6 Special Confining Reinforcement Required for Beam

13.12.5 Column and inclined Members

Column size

1. The minimum dimension of the member shall not be less than $20d_b$ *mm*.
   where, d_b *is the diameter of the largest* reinforceing bar in the beam.

2. The ratio of smaller dimension to larger dimension of the cross-section of the column shall not be less than 0.45 *i.e.*

$$i.e. \quad \frac{b}{D} \not< 0.45 \quad or \quad b \geq 0.45\,D$$

It is preferable to have large size of column to achieve formation of hinges at the joint in the beam portion so as to make the structure capable of resisting earthquake forces effectively. This leads to the view point of strong column weak beam design concept.

13.12.6 Longitudinal Reinforcement in Column.

(1) (a) Lap splices shall be provided only in the central half of clear height of column.

(b) Not more than 50% of the bars shall be spliced at one section.

(c) Hoop shall be provided over the entire splice length at spacing not exceeding 150 *mm*.

(d) Lap length ≮ development length of largest longitudinal bar in tension.

(e) For bar diameter > 32 mm lap splice should not be provided.

(f) Circular columns shall have minimum 6 bars.

During earthquake, spalling of concrete cover is likely to occur at the ends of the column where large rotations occur. Further, the effect of stress reversal in the central portion of the column are much less. Therefore, splice shall be provided in the centre half portion of the column.

(2) Any area of column that extends more than 100 *mm* beyond the confined core due to architectural requirements, shall be detailed in the following manner.

(a) In case the contribution of this area to strength has been considered, then it will have the minimum longitudinal and transverse reinforcement.

(b) If this area has been treated as non - structural, the minimum reinforcement requirement shall be governed by IS: 456-2000.

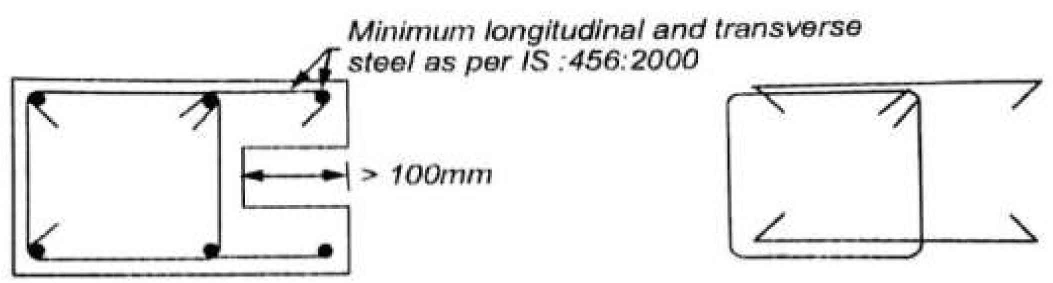


Fig. 13.12.7 Reinforcement Requirement for Column with More than 100 *mm* Projection Beyond Core

13.12.7 Transverse Reinforcement

(1) The transverse reinforcement for circular columns shall consist of spiral *or* circular hoops.

2) A closed link is a closed stirrup, having 135° hook ends with 6 diameter extension (but ≮ 65 *mm*) at each end, that is embedded in the confined core *(see Fig. 13.12.8a)*

In rectangular columns, rectangular hoops may be used.

a) Mimnimum diameter of transverse reinforcement shall be 8 mm.
For longitudinal bar of diameter more than 32 mm, 10 mm diameter of hoop shall be used.

b) The parallel legs of the rectangular hoops shall be spaced not more than 300 *mm* centre to centre. If the length of any side of the rectangular hoop exceeds 300 *mm*, a cross-tie shall be provided *(Fig. 13.12.8b)*
Alternatively, a pair of overlapping hoop may be provided within the column *(see Fig. 13.12.8c)*

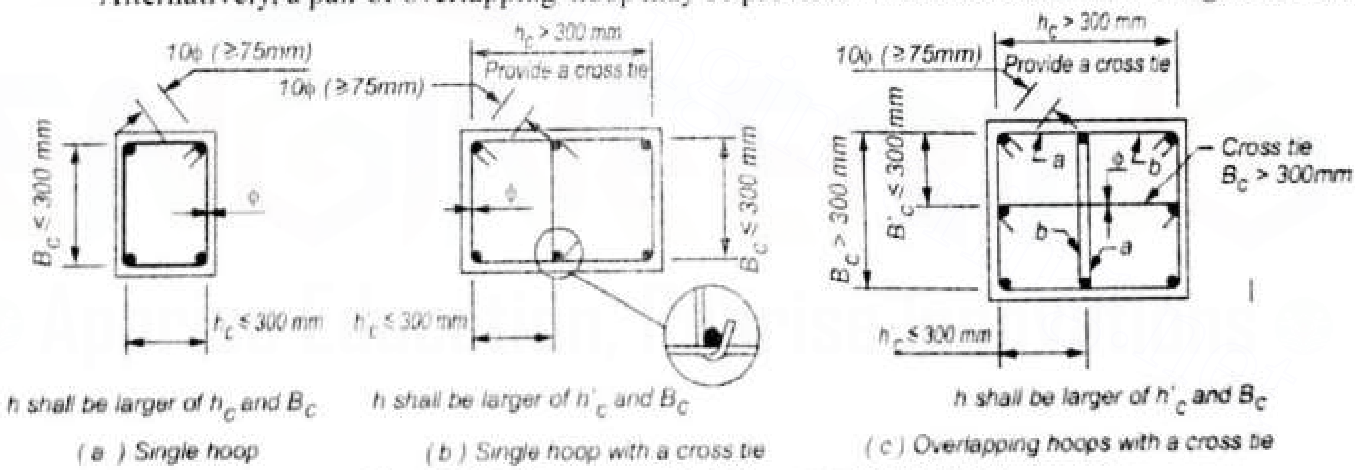


Fig. 13.12.8 Transverse Reinforcement in Column

2. The maximum spacing of hoops shall not exceed half the least lateral dimension of the column, except where special confining reinforcement is provided.
3. The design shear force for column shall be maximum of :
   a) calculated factored shear force as per analysis, and
   b) a factored shear force given by:

For sway to right

$$V_u = 1.4\ \frac{M_u^{As} + M_u^{Bh}}{h_{st}}$$

For sway to left

$$V_u = 1.4\ \frac{M_u^{Ah} + M_u^{Bs}}{h_{st}}$$

where, M_u^{As}, M_u^{Ah}, M_u^{Bs} and M_u^{Bh} design sagging and hogging moment of resistance of beams framing in to the column on oopposite dfaces A and B respectively with one hogging moment and the other sagging moment.(Fig. 13.12.9) and h_{st} is the storey height. The beam moment capacity to be calculated as per IS:456

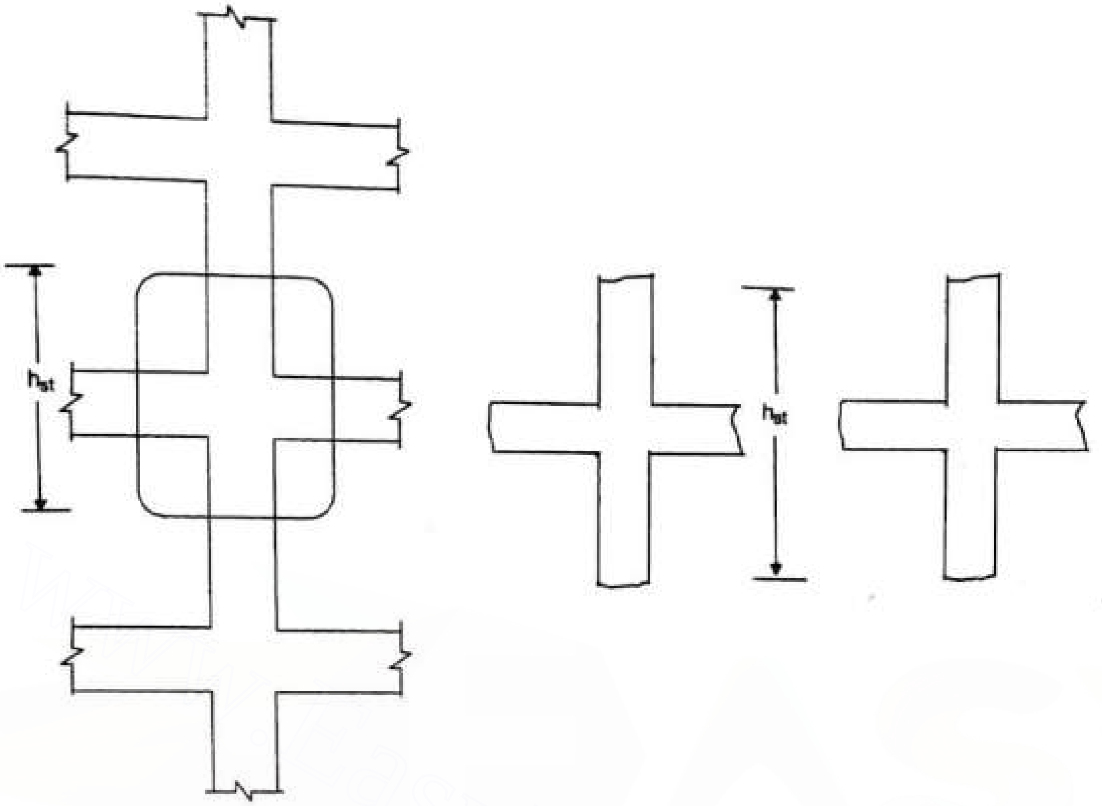


Fig. 13.12.9 Calculation of Design Shear Force for Column

13.12.8 Special Confining Reinforcement

This requirement shall be met with, unless a larger amount of transverse reinforcement is required from shear strength considerations.

Flexural yielding is likely in beams during strong earthquake shaking and the column when the shaking intensity is excees the espected intensity of earthquake shaking the special confining reinforcement shall be provided.

Special confining reinforcement shall consist of closely spaced hoops *or* spirals serves the following purposes :

(a) to confine the concrete there by increasing the ultimate concrete strain,

(b) increase ductility to concrete cross section,

(c) to provide lateral restraint against buckling to compression reinforcement.

(1) Special confining reinforcement shall be provided from the face of the joint towards mid-span of beam and mid-heigtht of columns, on either sideof joint for the length L_o given by:

(a) larger lateral dimension (*i.e.D*) of the section, where yielding occurs,

(b) 1/6 of the clear span of the member, or

(c) 450 *mm*

At spacing not more than,

(a) 1/4 of minimum diamension of thebeam or column,

(b) 6 times minimum diameter of longitudinal bars, and

(c) 100mm link.

And area A_{sh} of cross-section of bar forming links or spirals of at least:

a) A_{sh} in circular links or spirals:

$$A_{sh} = \text{Maximum of } \quad 0.09\, s_v\, D_k \times \frac{f_{ck}}{f_y}\left[\frac{A_g}{A_k} - 1\right] \quad \text{or} \quad 0.024\, s_v D_k \frac{f_{ck}}{f_y}$$

where, s_v = pitch of spiral or spacing of links

D_k = dimeter of core of cicular column measured to outside of spiral link,

A_g = gross area of column section,

$$A_{sh} = \text{Maximum of } \quad 0.18\, s_v\, h \times \frac{f_{ck}}{f_y}\left[\frac{A_g}{A_k} - 1\right] \quad \text{or} \quad 0.05\, s_v h \frac{f_{ck}}{f_y}$$

where, h = longer diamension of rectangular link measured to its outer face not greater than 300mm

A_k = area of confined concrete core in rectangular link measured to its outer dimension.

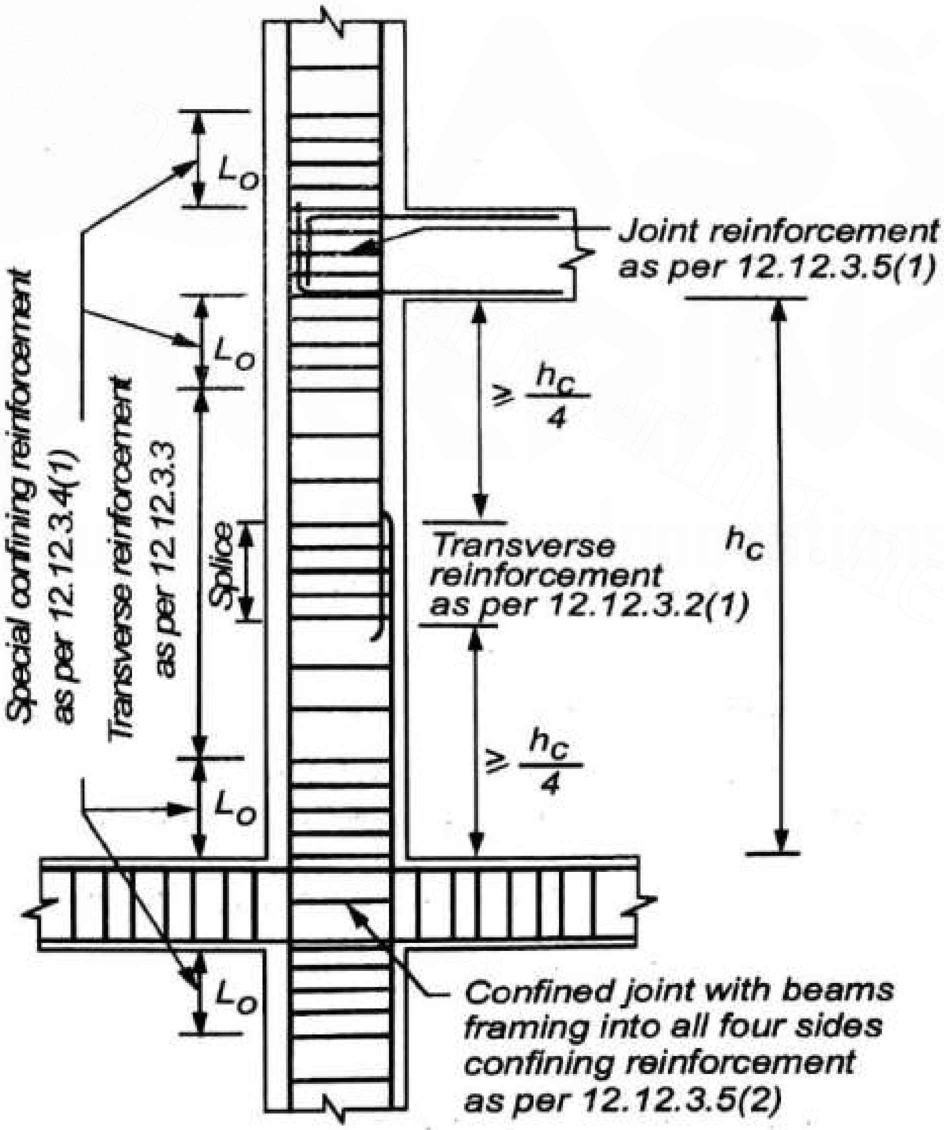


Fig. 13.12.10 Column and Joint Detailing

The dimension h of the link could be reduced by introducing cross-ties . In such cases, Ak shall be measured as overall core area, regadless of link arrangement. Hooks of cross-ties shall engaged pripherial longitudinal bars.

When a column terminates into a footing *or* mat, special confining reinforcement shall extend at least 300 *mm* into the footing *or* mat *(see Fig. 13.12.11)*

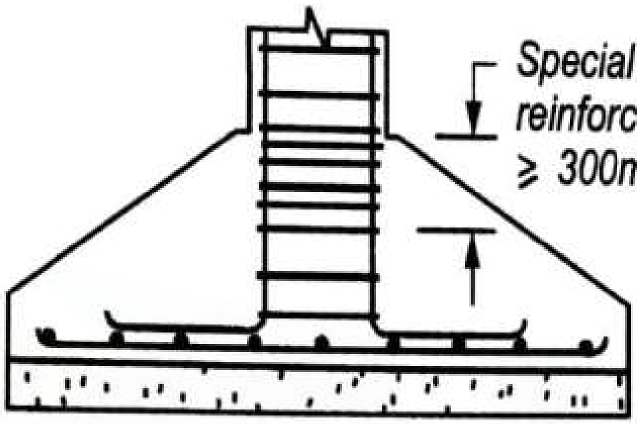


Fig. 13.12.11 Provision of Special Confining Reinforcement in Footings

During severe earthquake, plastic hinge is likely to occur at the bottom of the column that terminates into a footing *or* mat, resulting in large cyclic inelastic deformations. Hence, confining reinforcement needs the provision.

When the calculated point of contraflexure, under the effect of gravity and earthquake effects, is not within the middle half of the member clear height, special confining reinforcement shall be provided over the full height of the column.

Special confining reinforcement shall be provided over the full height of the column which has significant variation in stiffness along its height. This variation in stiffness due to abrupt changes in cross-section size, or unintended restrajnt to the column provided by stir-slab, mazzanine floor, plinth or lintelbeam framing into the columns, RC wall or masonry wall adjoinig column and extending only for partial column height.

Columns supporting reactions from discontinued stiff member, such as walls, shall be provided with special confining reinforcement over their full height as shown in *Fig. 13.12.12*
This reinforcement shall also be placed above the discontinuity for at least the development length of the largest longitudinal bar in the column.
Where the column is supported on wall, this reinforcement shall be provided over the full height of the column; it shall also be provided below the discontinuity for the same development length.

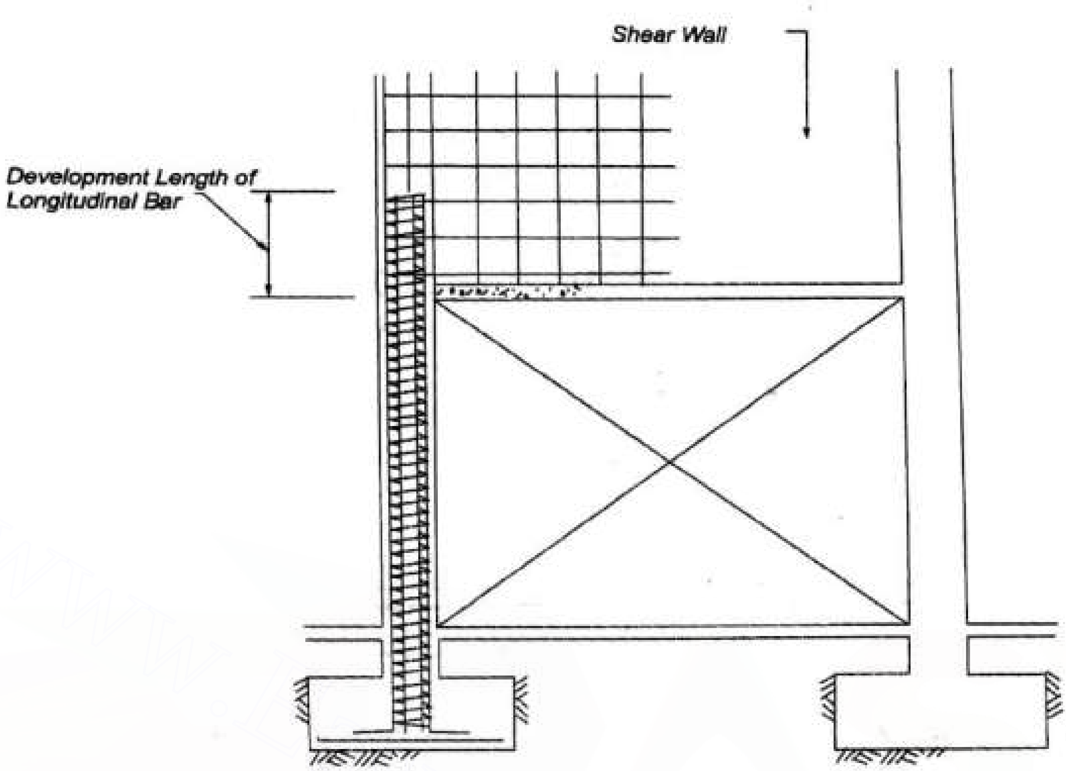


Fig. Columns with Variable Stiffness

APPENDIXES

APPENDIX - A LOAD DATA

Table A-1 : Dead Load : - [as per IS : 875 (Part - I) 1987]

The dead load is the load produced by static weight of structural members, walls, partitions, floors, roofs, and finishes, including all other permanent construction which act on the structure throughout their life span.

Dead Load Allowances for Partitions.

Partitions shown on plans	*Actual weight*
Loads due to Light partitions	UDL per square meter shall be ≮ 33 $^1/_3$ % weight per metre run of finished partitions subject to a minimum of 1 kN/m^2 provided total weight of partition walls $/m^2$ of wall area ≯ 1.5 kN/m^2 and total weight per metre length < 4 *kN*.

Unit Weights of Building Materials

Material	*Particulars*	*Unit weight* kN/m^3
Bricks	- Broken - Fine - course - Common burnt clay bricks - Refractory bricks - Engineering bricks	14.2 9.9 15.7 to 18.85 17.25 to 19.6 21.2
Cement Cement concrete	- Ordinary portland - Rapid hardening - Plain with sand and gravel or natural crushed stone aggreagte	14.1 12.55 22.0 to 23.5
Masonry	- Common burnt clay brick - stone	18.85 22.0 to 26.5
Mortar	- Cement - Lime	20.4 15.7 to 18.05
Plaster	- Cement - Lime	20.4 17.25
Prestressed Concrete		23.5
Reinforced Concrete	- with 2 % steel - with 5 % steel	23.25 to 24.80 24.80 to 26.50
Sand	- Dry clean - River - Wet	15.1 to 15.70 18.05 17.25 to 19.60
Aggregate Course	- Dry - Wet	15.7 to 18.35 18.85 to 21.95
Soils	- Dry - Moist	13.85 to 18.05 15.70 to 19.60
Steel	-	76.5

Table A - 2 Imposed Floor Load [as per IS : 875 (Part - 2) 1987]

Imposed load is assumed to be produced by the intended use or occupancy of a building, including the weight of movable partitions, load due to impact and vibration but excluding wind, seismic, snow and other loads due to temperature changes, creep, shrinkage, differential settlement etc.

Sr. No. *Occupancy Classification*	*Uniformly Distributed Load (udl) kN/m²*	*Concentrated Load kN*
(1) RESIDENTIAL BUILDINGS		
(a) Dwelling houses :		
(1) All rooms and kitchens	2.0	1.8
(2) Toilet and bath rooms	2.0	-
(3) Corridors, passages, staircases including fire escapes and store rooms	3.0	4.5
(4) Balconies	3.0	1.5 per metre run concentrated at the outer edge
(b) Dwelling units planned and executed in accordance with IS : 8888 - 1979 only :		
(1) Habitable rooms, kitchens, toilet and bathrooms	1.5	1.4
(2) Corridors, passages and staircases including fire escapes	1.5	1.4
(3) Balconies	3.0	1.5 per metre run concentrated at the outer edge
(c) Hotels, hostels, boarding houses,lodging houses, dormitories, residential clubs :		
(1) Living rooms, bed rooms and dormitories	2.0	1.8
(2) Kitchens and laundries	3.0	4.5
(3) Billiards room and public lounges	3.0	2.7
(4) Store rooms	5.0	4.5
(5) Dining rooms, cafeterias and restaurants	4.0	2.7
(6) Office rooms	2.5	2.7
(7) Rooms for indoor games	3.0	1.8
(8) Baths and toilets	2.0	-
(9) Corriodors,passages,staircases etc.including fire escapes, lobbies - as per the floor serviced (excluding stores and the like) but not less than	3.0	4.5
(10) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 per metre run concentrated at the outer edge
(d) Boiler rooms and plant rooms - to be calculated but not less than	5.0	6.7
(e) Garages :		
(1) Garage floors (including parking area and repair workshops) for passenger cars and vehicles not exceeding 2.5 tonnes gross weight, including access ways and ramps - to be calculated but not less than	2.5	9.0

Sr. No. Occupancy Classification	Uniformly Distributed Load (udl) kN/m²	Concentrated Load kN
Residential Buildings Continued		
(2) Garage floors for vehicles not exceeding 4.0 tonnes gross weight (including access ways and ramps) - to be calculated but not less than	5.0	9.0
(2) EDUCATIONAL BUILDINGS		
(a) Class rooms and lecture rooms (not used for assembly purposes)	3.0	2.7
(b) Dining rooms, cafeterias and restaurants	3.0^+	2.7
(c) Offices, lounges and staff rooms	2.5	2.7
(d) Dormitories	2.0	2.7
(e) Projection rooms	5.0	-
(f) Kitchens	3.0	4.5
(g) Toilets and bathrooms	2.0	-
(h) Store rooms	5.0	4.5
(j) Libraries and archivers :		
(1) Stack room / stack area	6.0 kN/m^2 for a minimum height of 2.2 m + 2.0 kN/m^2 per metre height beyond 2.2 m	4.5
(2) Reading rooms (without separate storage)	4.0	4.5
(3) Reading rooms (with separate storage)	3.0	4.5
(k) Boiler rooms and plant rooms -to be calculated but not less than	4.0	4.5
(l) Corridors, passages, lobbies, staircases including fire escapes - as per the floor serviced (without accounting for storage and projection rooms) but not less than	4.0	4.5
(m) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 per metre run concentrated at the outer edge
(3) INSTITUTIONAL BUILDINGS		
(a) Bed rooms, wards, dressing rooms, dormitories and lounges	2.0	1.8
(b) Kitchens, laundries and laboratories	3.0	4.5
(c) Dining rooms, cafeterias and restaurants	3.0^+	2.7
(d) Toilets and bathrooms	2.0	-
(e) X-ray rooms, operating rooms, general storage areas - to be calculated but not less than	3.0	4.5
(f) Office rooms and OPD rooms	2.5	2.7

Sr. No. Occupancy Classification	Uniformly Distributed Load (udl) kN/m²	Concentrated Load kN
Institutional Buildings Continued ...		
(g) Corridors, passages, lobbies and staircases including fire escapes - as per the the floor services but not less than	4.0	4.5
(h) Boiler rooms and plant rooms - to be calculated but not less than	5.0	4.5
(j) Balconies	Same as the rooms to which they give access but with a minimum of 4.0	1.5 per meter run concentrated at the outer edge
(4) ASSEMBLY BUILDINGS		
(a) Assembly areas :		
(1) with fixed seats #	-	4.0
(2) without fixed seats	5.0	3.6
(b) Restaurants (subject to assembly) museums and art galleries and gymnasia	4.0	4.5
(c) Projection rooms	5.0	-
(d) Stages	5.0	4.5
(e) Office rooms, kitchens and laundries	3.0	4.5
(f) Dressing rooms	2.0	1.8
(g) Lounges and billiards rooms	2.0	2.7
(h) Toilets and bathrooms	2.0	-
(i) Corridors passages, staircases including fire escapes	4.0	4.5
(j) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 per metre run concentrated at the outer edge
(k) Boiler rooms and plant rooms including weight of machinery	7.5	4.5
(l) Corridors, passages subject to loads greater than from crowds, such as wheeled vehicles, trolleys and the like. Corridors, staircases and passages in grandstands	5.0	4.5
(5) BUSINESS AND OFFICE BUILDINGS		
(a) Rooms for general use with separate storage	2.5	2.7
(b) Rooms without separate storage	4.0	4.5
(c) Banking halls	3.0	2.7
(d) Business computing machine rooms (with fixed computers or similar equipment)	3.5	4.5
(e) Records / files store rooms and storage space	5.0	4.5

Sr. No. Occupancy Classification	Uniformly Distributed Load (udl) kN/m²	Concentrated Load kN
Business And Office Buildings Continued...		
(f) Vaults and strong room - to be calculated but not less than	5.0	4.5
(g) Cafeterias and dining rooms	3.0^{+}	2.7
(h) Kitchens	3.0	2.7
(i) Corridors, passages, lobbies and staircases including fire escapes - as per the floor serviced (excluding stores) but not less than	4.0	4.5
(j) Bath and toilet rooms	2.0	-
(k) Balconies	Same as rooms to which they give access but with a minimum of 4.0	1.5 per metre run concentrated at the outer edge
(l) Stationary Stores	4.0 for each metre of storage height	9.0
(m) Boiler rooms and plant rooms - to be calculated but not less than	5.0	6.7
(n) Libraries	Same as (ii) Educational Building Sr.No.(j)	
(6) MERCANTILE BUILDINGS		
(a) Retail shops	4.0	3.6
(b) Wholesale shop-to be calculated but not less than	6.0	4.5
(c) Office rooms	2.5	2.7
(d) Dining rooms, restaurants and cafeterias	3.0^{+}	2.7
(e) Toilets 2.0	-	
(f) Kitchens and laundries	3.0	4.5
(g) Boiler rooms and plant rooms - to be calculated but not less than	5.0	6.7
(h) Corridors, passages, staircases including fire escapes and lobbies	4.0	4.5
(i) Corridors, passages, staircases subject to loads greater than from crowds, such as wheeled vehicles, trolleys and the like.	5.0	4.5
(j) Balconies	same as (iii) Institutional Buildings Sr.No.(j)	
(7) INDUSTRIAL BUILDINGS		
(a) Work areas without machinery / equipment	2.5	4.5
(b) Work areas with machinery / equipment @		
(1) Light duty } To be calculated but not less than	5.0	4.5
(2) Medium duty } To be calculated but not less than	7.0	4.5
(3) Heavy duty } To be calculated but not less than	10.0	4.5

Sr. No.	Occupancy Classification	Uniformly Distributed Load (udl) kN/m^2	Concentrated Load kN
	Industrial Buildings Continued...		
	(c) Boiler rooms and plant rooms - to be calculated but not less than	5.0	6.7
	(d) Cafeterias and dining rooms	3.0+	2.7
	(e) Corridors, passages and staircases including fire escapes	4.0	4.5
	(f) Corridors, passages, staircases subject to machine loads,wheeled vehicles - to be calculated but not less than	5.0	4.5
	(g) Kitchens	3.0	4.5
	(h) Toilets and bathrooms	2.0	-
(8)	***STORAGE BUILDINGS *****		
	(a) Storage rooms (other than cold storage) warehouse - to be calculated based on the with a minimum of bulk density of materials stored but not less than	2.4 kN/m^2 per each metre of storage height 7.5 kN/m^2	7.0
	(b) Cold storage - to be calculated but not less than meter	5.0 kN/m^2 per each metre of storage height with a minimum of 15 kN/m^2	9.0
	(c) Corridors, passage and staircases including fire eacapes - as per the floor serviced but not less than	4.0	4.5
	(d) Corridors, passage subject to loads greater than from crowds, such as wheeled vehicles, trolleys and the like.	5.0	4.5
	(e) Boiler rooms and plant rooms	7.5	4.5

* Guide for requirements of low income housing.

\+ Where unrestricted assembly of persons is anticipated, the value of UDL should be increased to 4.0 kN/m^2

\# With fixed seats' implies that the removal of the seating and the use of the space for other purpose is improbable. The maximum likely load in this case is, therefore closely controlled.

@ The loading in industrial buildings (workshops and factories) varies considerably and so three loadings under the terms " light ", " medium " and " heavy " are introduced in order to allow for more economical designs. It is, however, important particularly in the case of heavy weight loads, to assess the actual loads to ensure that they are not in excess of 10 kN/m^2 ; in case where they are in excess, the design shall be based on the actual loadings.

** For various machanical handing equipment which are used to transport goods, as in warehouse, store rooms, etc. the actual load coming from the use of such equipment shall be ascertained and design should carter to such loads.

Note :

(1) The concentrated loads specified may be assumed to act over an area of 0.3 m x 0.3 m.

(2) The concentrated loads need not be considered where the floors are capable of effective lateral distribution of this load.

Table A 3 -Load Data for Residential Building

1. Values of Load due to Floor/Roof Finish (FF), Imposed Load (LL) and Total Depth of the Slab (D) assumed for different types of Slabs for Computing its Self Weight.

(a) Roof	: FF = 1.5 kN/m^2 and	LL = 1.5 kN/m^2	, D = 120 mm
(b) Floor	: FF = 1.0 kN/m^2 and	LL = 2.0 kN/m^2	, D = 120 mm
(c) Bath & w.c.	: FF = 2.5 kN/m^2 and	LL = 2.0 kN/m^2	, D = 100 mm
(d) Loft	: FF = 0.75 kN/m^2 and	LL = 2.0 kN/m^2	, D = 100 mm
(e) Balconies -			
Cantilever	: FF = 1.0 kN/m^2 and	LL = 3.0 kN/m^2	, D = 150 mm
Simply supported	: FF = 1.0 kN/m^2 and	LL = 3.0 kN/m^2	, D = 100 mm
(f) Stair	: FF = 1.0 kN/m^2 and	LL = 3.0 kN/m^2	, D = 140 mm

2. Load on Slab

(a) Roof : Ultimate load = w_u = (Self+LL+FF)x1.5 = (25x.12+1.5+1.5)x1.5 = 9 kN/m^2

(b) Floor : Working load = w_u = (Self+LL+FF)x1.5 = (25x.12+1.0+2.0)x1.5 = 9 kN/m^2

(c) Bath and W.C.

Ultimate load = w_u = (Self+LL+FF)x1.5 = (25x.10+2.0+2.5)x1.5 = 10.5 kN/m^2

(d) Loft : Ultimate load = w_u = (Self+LL+FF)x1.5 = (25x.10+2.0+.75)x1.5 = 8 kN/m^2

(e) Balcony (i) Cantilever :

Ultimate load = w_u = (Self+LL+FF)x1.5 = (25x.15+3.0+1.)x1.5 = 12 kN/m^2

(ii) Simply supported

Ultimate Load= w_u = (25 x 0.1+1+3)x1.)x1.5 = 10 kN/m^2

3. Loads due to walls Per Metre Hight Per Metre length :

(a) Brick masonry: (i) 200 mm (8") thick , 225 mm with plaster.

Ultimate load = w_u = (20x.0.225)x1.5 = 6.75 $kN/m/m$

(ii) 100 mm (4") thick , 125 mm with plaster.

Ultimate load = w_u = (20x.0.125)x1.5 = 3.75 $kN/m/m$

Unit weight of bricks massonry assumed = 20 kN/m^3

(b) Solid Concrete Block 150mm thick, 175mm with plaster

Ultimate load = w_u = (25x.0.175)x1.5 = 6.75 $kN/m/m$

4. Load due to R.C.C Parapet wall 80 mm thick :

Ultimate load = w_u = (25x.0.08)x1.5 = 3.00 $kN/m/m$

5. Load due to R.C.C Grill 80 mm thick with 50% openings :

Ultimate load = w_u = (25x.0.08/2)x1.5 = 1.50 $kN/m/m$

6. Factored Self weight of Beam in kN/m

Total Depth of beam in mm	300	380	450	530	600	680	750
Flanged beam 200 mm wide rib	1.5	2.1	2.6	3.7	3.8	4.4	4.9
Rectangular beam 230 mm wide	2.6	3.3	3.8	4.6	5.2	5.9	6.5

Depth of rib = $(D-D_f)$ = (D–100) , for assumed minimum slab depth of 100 mm

APPENDIX - B BEARING CAPACITY OF SOIL

Table B-1 : Safe Bearing Capacity (S.B.C) of Different Soils

Non - Cohesive Soils		*# Cohesive Soils*	
Type of Soil	S.B.C *kN/m²*	Type of Soil	S.B.C *kN/m²*
1. Gravel, Sand and Gravel compact offering high resistance to penetration when excavated by tools.	450	1. Softshale, hard or stiff clay in deep bed, dry	450
2. Coarse sand compact and dry*	450	2. Medium clay readily indented with a thumb nail	250
3. Medium sand compact and dry	250	3. Moist clay and sand clay mixture which can be indented with strong thumb pressure	150
4. Loose gravel or sand gravel mixture; Loose coarse to medium sand, dry	250	4. Black cotton soil or other shrinkable or expansive clay in dry condition (50% saturation)	150
5. Fine sand,silt (dry lumps)easily pulverised by the fingures	150	5. Soft clay indented with moderated thumb pressure	100
6. Fine sand, loose and dry	100	6. Very soft clay which can be penetrated several inches with the thumb.	50

Notes :

1) Increase or decrease the allowable bearing values as follows :
   (a) The allowable bearing values may be increased by an amount equal to the weight of the mataerial removed from above the bearing level, that is, the base of the foundation.
   (b) For not - cohesive soils, the allowable bearing value shall be reduced by 50% if the water table is above or near the bearing surface of the soil at a distance at least equal to the width of the foundations, no such reduction shall apply. For intermediate depths of the water table, proportional reduction of the allowable bearing value may be made.
2) Compactness or looseness of non - cohesive materials may be determined by driving a wooden picket of dimensions 50 x 50 x 700mm with a sharp point. The picket shall be pushed vertically into the soil by the full weight of person weighing at least 70 kg. If the penetration of the picket exceeds 200mm, the loose state shall be assumed to exist.

* 3) Dry means that the ground water level is at a depth not less than the width foundation below the base of the foundation.

\# 4) Cohesive soils are susceptible to long term consolidation settlement.

5) The bearing capacities of peat, fills and made - up ground shall be determined after investigation. No generalised values for safe bearing pressures can be given for these types of soils. In such areas, adequate site investigation (See IS : 1892 - Code of Practice for Site Investigations for Foundations) shall be carried out expert advice shall be sought.

APPENDIX - C EXPOSURE CONDITIONS

Table C-1 : Exposure Conditions, Nominal Concrete Cover and Minimum Grade of Concrete

see connected clauses and Tables T3,T5,T16

Environment	*Exposure Conditions*	*Minimum Concrete grade For RCC work*	*Nominal Cover*
Mild	Concrete surfaces protected against weather or aggressive conditions, except those situated in coastal area.	M 20	20 mm
Moderate	Concrete surfaces sheltered from severe rain or freezing whilst wet. Concrete exposed to rain and condensation. Concrete continuously under water. Concrete in contact or buried under non-aggressive soil/ground water. Concrete surfaces sheltered from saturated salt air in coastal area.	M 25	30 mm
Severe	Concrete surfaces exposed to severe rain, alternate wetting and drying or occasional freezing whilst wet or severe condensation. Concrete completely immersed in sea water. Concrete exposed to coastal environment.	M 30	45 mm
Very Severe	Concrete surfaces exposed to sea water spray, corrosive fumes, or severe freezing conditions whilst wet. Concrete in contact with or buried under aggressive subsoil / ground water.	M 35	50 mm
Extreme	Surface of members in tidal zone. Members in direct contact with liquid/solid aggressive chemicals.	M 40	75 mm

Notes : *1. Nominal cover is the design depth of concrete cover measured from concrete surface to all steel reinforcement including links. It shall not be less than the diameter of the bar.*

2. For main reinforcement upto 12 mm diameter bar for mild exposure the nominal cover may be reduced by 5 mm.

3. (a) For a longitudinal reinforcing bar in a column nominal cover shall not be less than 40 mm nor diameter of such bar.

(b) For columns of minimum dimension of 200 mm. or under whose reinforcing bars do not exceed 12 mm a nominal cover of 25 mm. may be used.

4. For footing minimum cover shall be 50 mm.

5. Unless specified otherwise, actual concrete cover should not deviate from the required nominal cover by + 10 mm to 0 mm.

6. For exposure condition 'severe' and 'very severe', reduction of specified cover by 5 mm. may be made, where concrete grade is M35and above.

Comments :

(1) For mild exposure for main steel of 12 mm diameter the reduced cover of 15 mm (= 20 mm - 5 mm) for beam of any size is considered to be inadequate by the authors.

(2) Increased cover in flexural members particularly for R.C. slabs is likely to lead increased crack width affecting durability by permitting ingress of moisture and chemical attack.

Appendix - D - 1 PRACTICAL SIZES OF MEMBERS

(a) Practical Dimensions

Member	Dimension	Values in mm
Slab	Thickness	**100**, 110, **120,125**,130,**140,150**,160,180,200
Beam	Width	150, **200,230.250,300**,380,400,450
	Depth	150,200,**230,250,300,380,400,450,500,530,600** **840,900**,1000
Column	Width/Diam.	150,200,230250,300,380,400,450
	Depth	150,200,**230,250,300,380,400,450,500,530, 600, 680,750 ,800,840, 900**,1000

Note: Bold values are more common

(b) Recomended Practical Sizes in mm for Beam and Column

Sr.No.	Size	Sr. No.	Size	Sr. No.	Size	Sr. No.	Size	Sr. No.	Size
1	200 x 200	9	230 x 230	17	250 x 250	25	300 x 300	33	300 x 400
2	200 x 250	10	230 x 300	18	250 x 300	26	300 x 380	34	300 x 500
3	200 x 300	11	230 x 380	19	250 x 400	27	300 x 450	35	300 x 800
4	200 x 400	12	230 x 450	20	250 x 450	28	300 x 530	36	300 x 950
5	200 x 450	13	230 x 530	21	250 x 500	29	300 x 600	37	400 x 400
6	200 x 500	14	230 x 600	22	250 x 600	30	300 x 680	38	450 x 450
7	200 x600	15	230 x 680	23	250 x 750	31	300 x 750	39	500 x 500
8	200 x 800	16	230 x 750	24	250 x 800	32	300 x 840	40	600 x 600

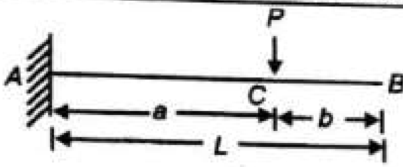
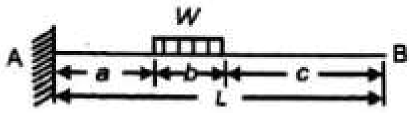
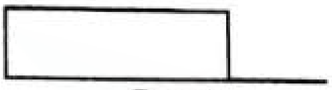
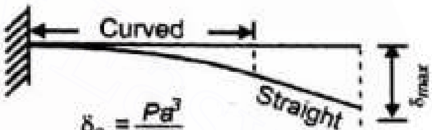
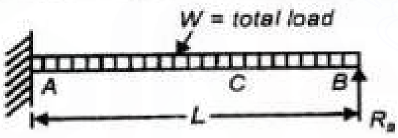
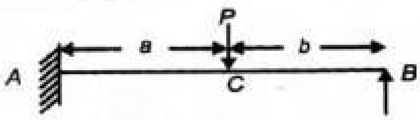
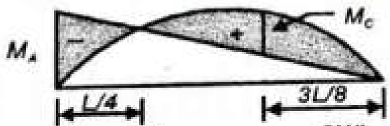
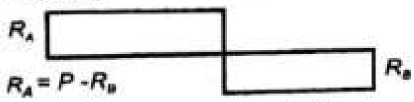
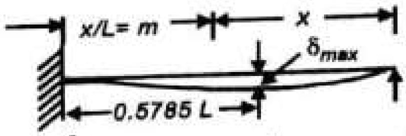
Notes: (1) Sr. No. 1 to 8, 17 to 24 and 33 to 36 form new section series with a module of 50
(2) Sr,No, 9 to 16, and 25 to 32 form old section series with module of 3 inches
These are still very common old modules and still used in practice.
They correspod to 9,12,15,18,21,24,27,30,33 and 36 inches
(3) Sr. No. 37 to 40 are for square/ circular columns.

(c) Recommended Main Steel Percentage

	Fe 250	Fe415/500
Slabs and singly reinforced sections	0.3 to 0.7	0.2 to 0.4
Doubly reinforced section	0.7 to 2.0	0.3 to 1.5
Columns	0.8 to 2.0	0.8 to 2.0

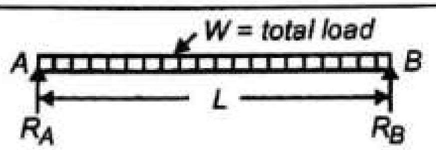
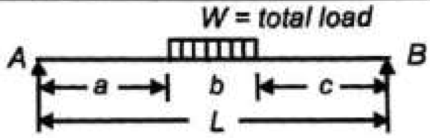
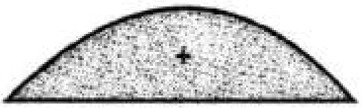
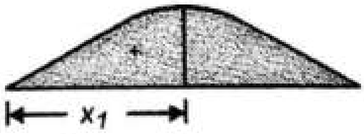
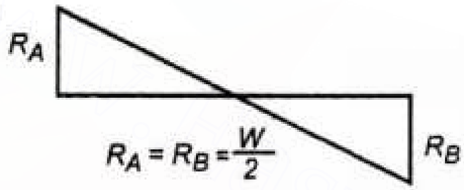
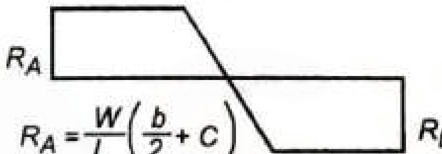
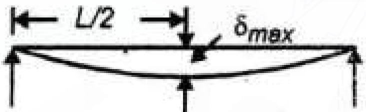
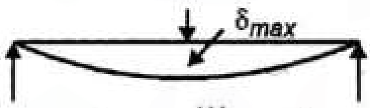
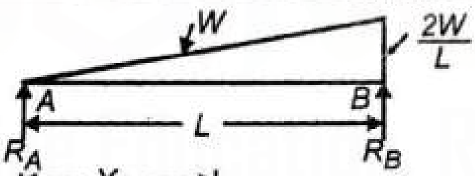
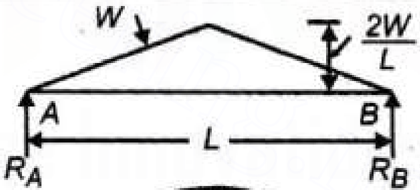
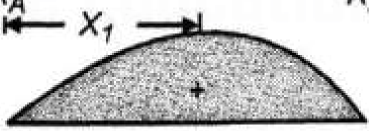
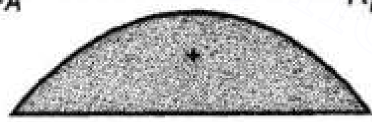
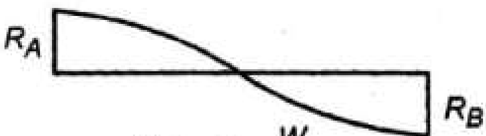
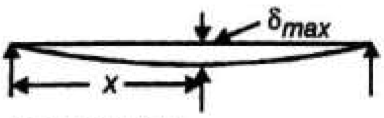
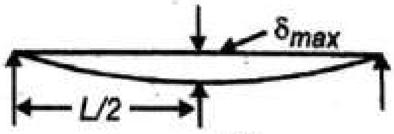
APPENDIX - D - 2 Formulae for calculating B- M , S. F and Deflection of Beams

Cantilever beam and Propped Cantilever

	For point load at cantilever end take b = o	For UDL for whole span take a = c = o
Loading		
Bending Moment	 $M_x = Px$ $M_{max} = Pa$	 $M_{max} = W(a + b/2)$
Shear Force	 $R_A = P$	 $R_A = W$
Deflection	 $\delta_C = \frac{Pa^3}{3EI}$ $\delta_{max} = \frac{Pa^3}{3EI}\left(1 + \frac{3b}{2a}\right)$	 $\delta_{max} = \frac{W}{24EI}[8a^3 + 18a^2b + 12ab^2 + 3b^3 + 12a^2c + 12abc + 4b^2c]$
Loading Loading		
Bending Moment	 $M_A = \frac{-WL}{8}$, $M_C = \frac{9WL}{128}$	 $M_A = \frac{-Pb(L^2 - b^2)}{2L^2}$, $M_C = \frac{Pb}{2}\left(2 - \frac{3b}{L} + \frac{b^3}{L^3}\right)$ $(M_A)_{max} = -0.193\ PL$ when $b = 0.577L$ $(M_C)_{max} = 0.174\ PL$ when $b = 0.366\ L$
Shear Force Shear Force Deflection	 $R_A = \frac{5W}{8}$ $R_B = \frac{3W}{8}$	 $R_A = P - R_B$ $R_B = \frac{Pa^2}{2L^3}(b + 2L)$
Deflection	 $\delta = \frac{WL^3}{48EI}(m - 3m^3 + 2m^4)$, $\delta_{max} = \frac{WL^3}{185EI}$	 $\delta_C = \frac{Pa^3b^2}{12EIL^3}(4L - a)$

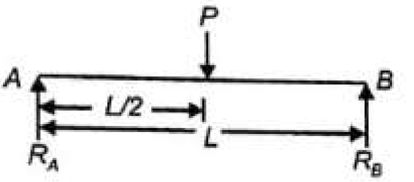
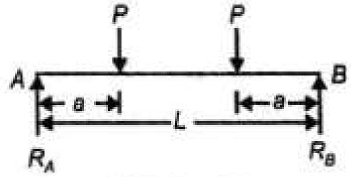
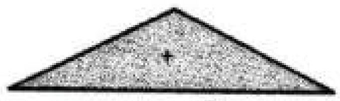
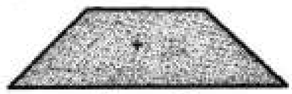
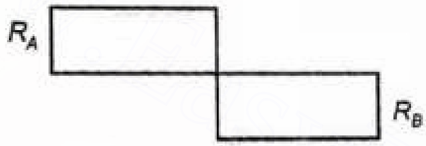
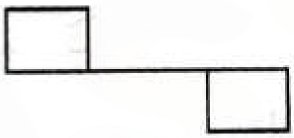
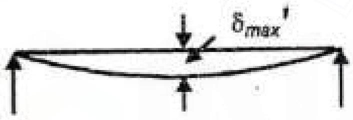
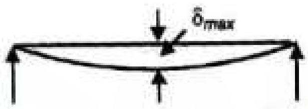
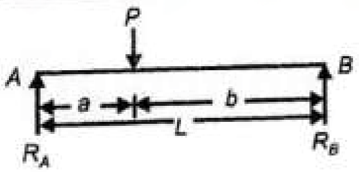
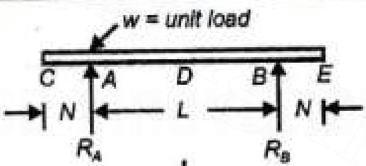
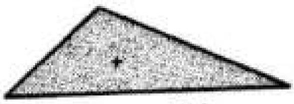
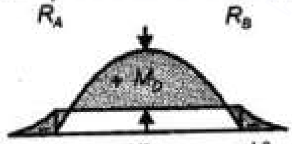
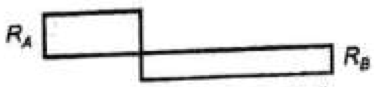
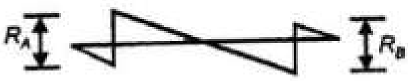
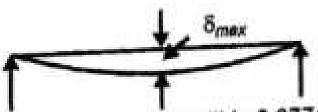
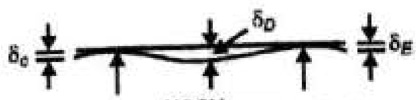
APPENDIX - D - 3 Formulae for calculating B-M, S. F and Deflection of Beams

Simply Supported Beams

Loading		
Bending Moment	 $M_x = \frac{Wx}{2}\left(1 - \frac{x}{L}\right)$, $M_{max} = \frac{WL}{8}$	 $M_{max} = \frac{W}{b}\left(\frac{x_1^2 - a^2}{2}\right)$ when $x_1 = a + \frac{R_A \times b}{W}$
Shear Force	 $R_A = R_B = \frac{W}{2}$	 $R_A = \frac{W}{L}\left(\frac{b}{2} + c\right)$ $R_B = \frac{W}{L}\left(\frac{b}{2} + a\right)$
Deflection	 $\delta_{max} = \frac{5}{384} \cdot \frac{WL^3}{EI}$	 when $a = c$, $\delta_{max} = \frac{W}{384EI}(8L^3 - 4Lb^2 + b^3)$
Loading		
Bending Moment	 $M_x = \frac{Wx}{3}\left(1 - \frac{x^2}{L^2}\right)$ $M_{max} = 0.128\ WL$ when $x_1 = 0.5774L$	 $M_x = Wx\left(\frac{1}{2} - \frac{2x^2}{3L^2}\right)$ $M_{max} = WL/6$
Shear Force	 $R_A = W/3$ $R_B = 2\ W/3$	 $R_A = R_B = \frac{W}{2}$
Deflection	 $\delta_{max} = \frac{0.01304WL^3}{EI}$, when $x = 0.5193L$	 $\delta_{max} = \frac{WL^3}{60EI}$

APPENDIX - D - 4 Formulae for calculating B-M, S.F and Deflection of Beams

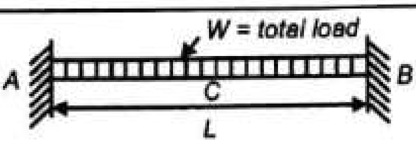
Simply Supported Beams (Continued)

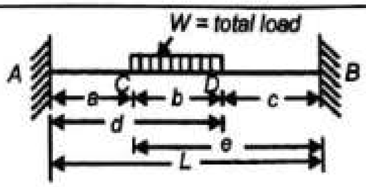
Loading		
Bending Moment	 $M_{max} = \frac{PL}{4}$	 $M_{max} = Pa$
Shear Force	 $R_A = R_B = \frac{P}{2}$	 $R_A = R_B = P$
Deflection	 $\delta_{max} = \frac{PL^3}{48EI}$	 $\delta_{max} = \frac{PL^3}{6EI}\left[\frac{3a}{4L} - \left(\frac{a}{L}\right)^3\right]$
Loading		
Bending Moment	 $M_{max} = \frac{Pab}{L}$	 $M_A = M_B = -\frac{wN^2}{2}$, $M_D = \frac{wL^2}{8} + M_A$
Shear Force	 $R_A = Pb/L$ $R_A = Pa/L$	 $R_A = R_B = w\left(N - \frac{L}{2}\right)$
Deflection	 δ_{max} always occurs within 0.0774 L of the centre of the beam. When b > a $\delta_{centre} = \frac{PL^3}{48EI}\left[\frac{3a}{L} - 4\left(\frac{a}{L}\right)^3\right]$ This value is always within 2.5 % of the maximum value	 $\delta_c = \delta_E = \frac{WL^3N}{24EI}(6n^2 + 3n^3 - 1)$ $\delta_D = \frac{WL^4}{384EI}(5 - 24n^2)$ where $n = \frac{N}{L}$

APPENDIX - D - 5 Formulae for calculating B-M, S.F and Deflection of Beams

Fixed Beams

Loading

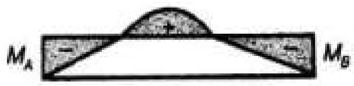




Bending Moment

$M_A = M_B = -\dfrac{WL}{12}$

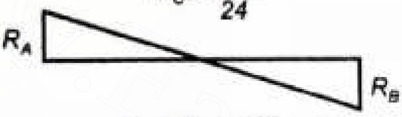
$M_C = \dfrac{WL}{24}$



$M_A = \dfrac{-W}{12L^2 b}\left[e^3(4L-3e) - c^3(4L-3C)\right]$

$M_B = \dfrac{-W}{12L^2 b}\left[d^3(4L-3d) - a^3(4L-3a)\right]$

Shear Force

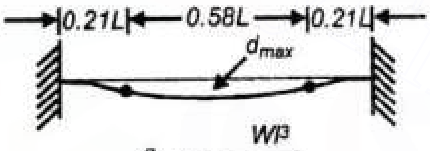


$R_A = R_B = W/2$

When r is simple support reaction

$R_A = r_A + \dfrac{M_A - M_B}{L}$, $R_B = r_B + \dfrac{M_B - M_A}{L}$

Deflection

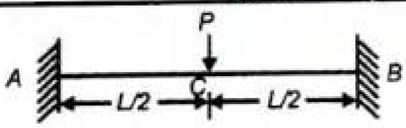


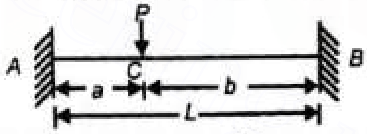
$\delta_{max} = \dfrac{WL^3}{384EI}$

When a = c

$\delta_{max} = \dfrac{-W}{384EI}\left[L^3 + 2L^2 a + 4La^2 - 8a^3\right]$

Loading





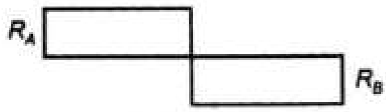
Bending Moment

$-M_A = -M_B = M_C = PL/8$

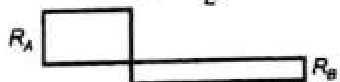
$M_A = -\dfrac{Pab^2}{L^2}$, $M_B = -\dfrac{Pba^2}{L^2}$

$M_C = \dfrac{2Pa^2 b^2}{L^3}$

Shear Force



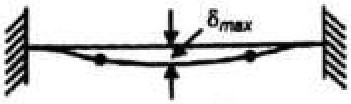
$R_A = R_B = P/2$



$R_A = P\left(\dfrac{b}{L}\right)^2\left(1 + 2\dfrac{a}{L}\right)$

$R_B = P\left(\dfrac{a}{L}\right)^2\left(1 + 2\dfrac{b}{L}\right)$

Deflection



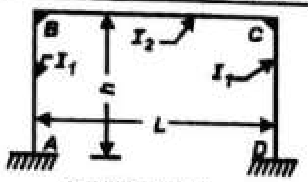
$\delta_{max} = \dfrac{PL^3}{192EI}$

$\delta_C = \dfrac{Pa^3 b^3}{EIL^3}$

$\delta_{max} = \dfrac{2Pa^2 b^3}{3EI(3L-2a)^2}$ when $x = \dfrac{L^2}{3L-2a}$

APPENDIX - D - 6 Formulae for Portal Frames

Formulae for Fixed Based Rectangular Portal Frames

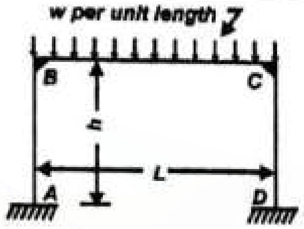


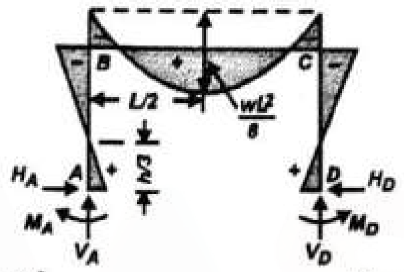
FRAME DATA

Coefficients :

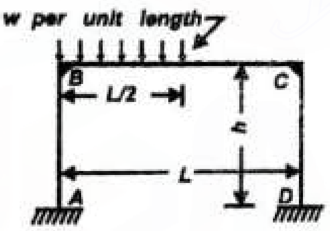
$$k = \frac{I_2}{I_1} \cdot \frac{h}{L}$$

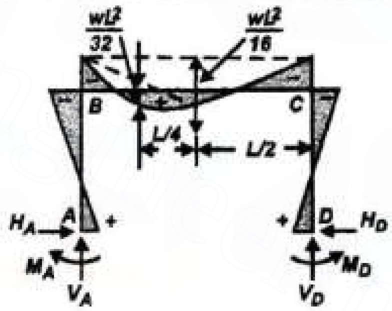
$$N_1 = k + 2 \qquad N_2 = 6k + 1$$





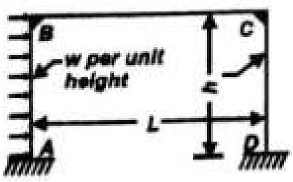
$$M_A = M_D = \frac{wL^2}{12N_1}, \quad M_B = M_C = -\frac{wL^2}{6N_1} = -2M_A, \quad M_{max} = \frac{wL^2}{8} + M_B, \quad V_A = V_D = \frac{wL}{2}, \quad H_A = H_D = \frac{3M_A}{h}$$

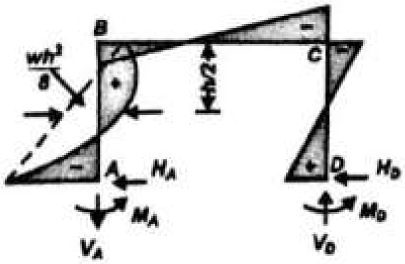




$$M_A = \frac{wL^2}{8}\left[\frac{1}{3N_1} - \frac{1}{8N_2}\right], \quad M_B = -\frac{wL^2}{8}\left[\frac{2}{3N_1} + \frac{1}{8N_2}\right], \quad M_D = \frac{wL^2}{8}\left[\frac{1}{3N_1} + \frac{1}{8N_2}\right], \quad M_C = -\frac{wL^2}{8}\left[\frac{2}{3N_1} - \frac{1}{8N_2}\right]$$

$$V_D = \frac{wL}{8}\left[1 - \frac{1}{4N_2}\right], \quad V_A = \frac{wL}{2} - V_D, \quad H_A = H_D = \frac{wL^2}{8hN_1}$$



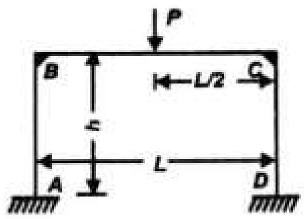


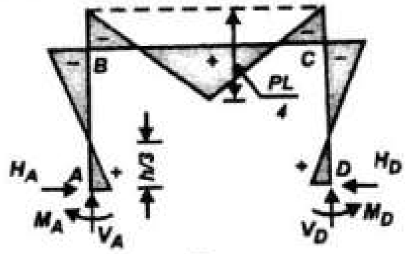
$$M_A = \frac{wh^2}{4}\left[-\frac{k+3}{6N_1} - \frac{4k+1}{N_2}\right], \quad M_B = \frac{wh^2}{4}\left[\frac{-k}{6N_1} + \frac{2k}{N_2}\right], \quad M_D = \frac{wh^2}{4}\left[-\frac{k+3}{6N_1} + \frac{4k+1}{N_2}\right], \quad M_C = \frac{wh^2}{4}\left[-\frac{k}{6N_1} + \frac{2k}{N_2}\right]$$

$$H_D = \frac{wh(2k+3)}{8N_1}, \quad H_A = -(wh - H_D), \quad V_A = -V_D = -\frac{wh^2k}{LN_2}$$

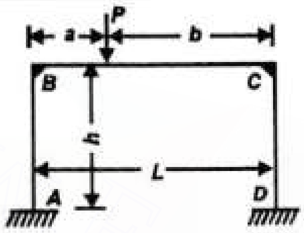
APPENDIX - D - 7 Formulae for Portal Frames

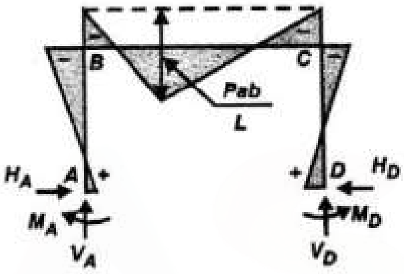
Formulae forFixed Based Rectangular Portal Frames (continued)





$$M_A = M_D = +\frac{PL}{8N_1}, \quad M_B = M_C = -2M_A, \quad V_A = V_D = \frac{P}{2}, \quad H_A = H_D = \frac{3M_A}{h}$$

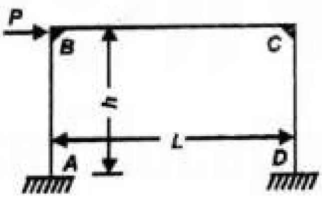


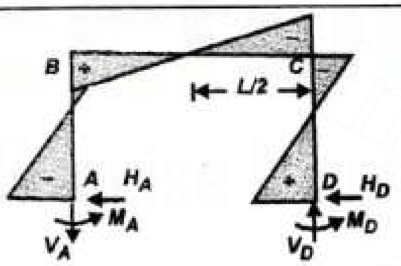


Constants : $a_1 = a/L \qquad b_1 = b/L$

$$M_A = +\frac{Pab}{L}\left[\frac{1}{2N_1} - \frac{b_1 - a_1}{2N_2}\right], \quad M_B = -\frac{Pab}{L}\left[\frac{1}{N_1} + \frac{b_1 - a_1}{2N_2}\right], \quad M_D = +\frac{Pab}{L}\left[\frac{1}{2N_1} + \frac{b_1 - a_1}{2N_2}\right]$$

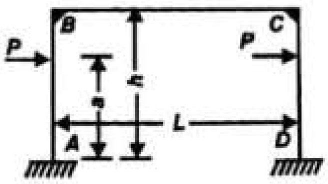
$$M_C = -\frac{Pab}{L}\left[\frac{1}{N_1} - \frac{b_1 - a_1}{2N_2}\right], \quad V_A = Pb_1\left[1 + \frac{a_1(b_1 - a_1)}{N_2}\right], \quad V_D = P - V_A, \quad H_A = H_D = \frac{3Pab}{2LhN_1}$$

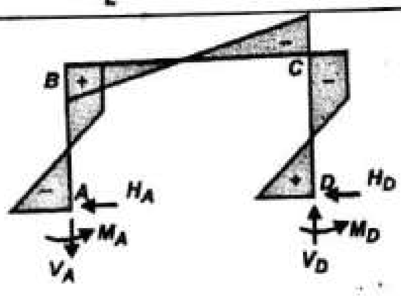




$$M_A = -\frac{Ph}{2} \times \frac{3k+1}{N_2} \qquad M_B = +\frac{Ph}{2} \times \frac{3k}{N_2}, \quad M_D = +\frac{Ph}{2} \times \frac{3k+1}{N_2}, \quad M_C = -\frac{Ph}{2} \times \frac{3k}{N_2}$$

$$H_A = -H_D = -\frac{P}{2}, \quad V_A = -V_D = -\frac{2M_B}{L}$$



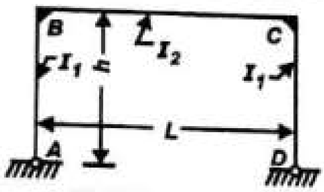


Constants : $a_1 = \dfrac{a}{h}, \quad X_1 = \dfrac{3Pa\,a_1 k}{N_2}$

$$M_A = -Pa + X_1, \quad M_B = X_1, \quad M_D = +Pa - X_1, \quad M_C = -X_1, \quad V_A = -V_D = -\frac{2X_1}{L}, \quad H_A = -H_D = -P$$

APPENDIX - D -8 Formulae for Portal Frames

Formulae for Hinged Based Rectangular Portal Frames

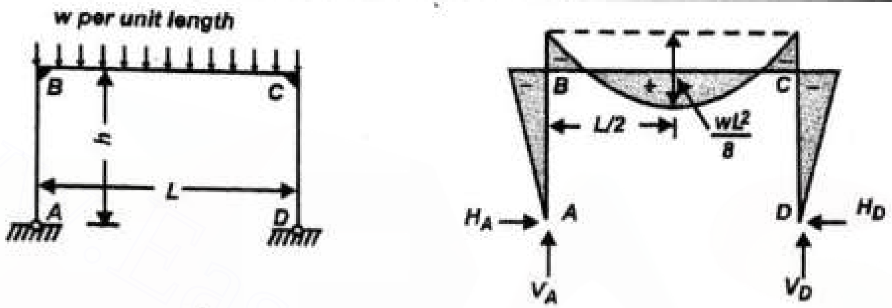


FRAME DATA

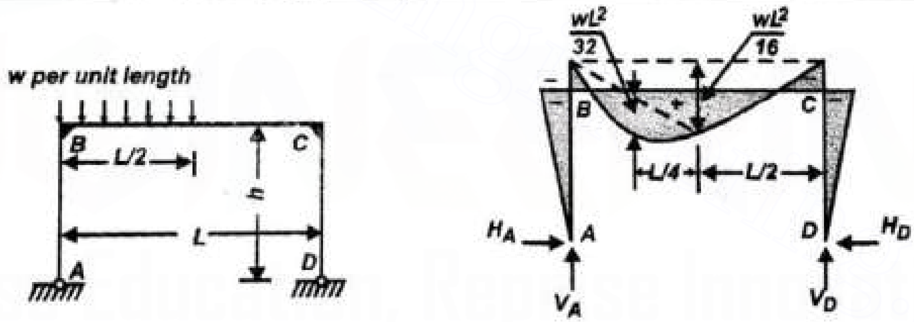
Coefficients :

$$k = \frac{I_2}{I_1} \cdot \frac{h}{L}$$

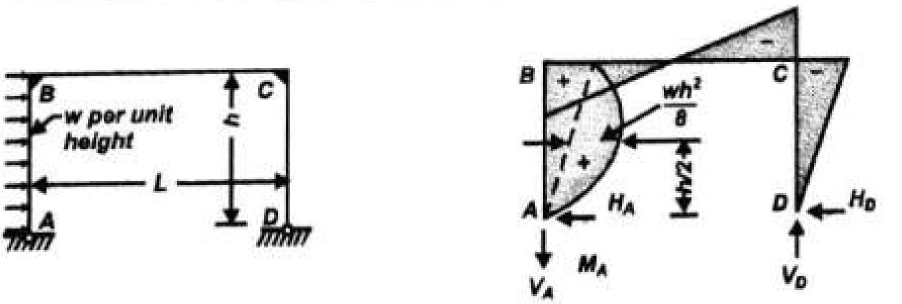
$$N = 2k + 3$$



$$M_B = M_C = -\frac{wL^2}{4N} \quad , \quad M_{max} = \frac{wL^2}{8} + M_B \quad , \quad V_A = V_D = \frac{wL}{2} \quad , \quad H_A = H_D = -\frac{M_B}{h}$$



$$M_B = M_C = -\frac{wL^2}{8N} \quad , \quad V_A = \frac{3wL}{8} \quad , \quad V_D = \frac{wL}{8} \quad , \quad H_A = H_D = -\frac{M_B}{h}$$

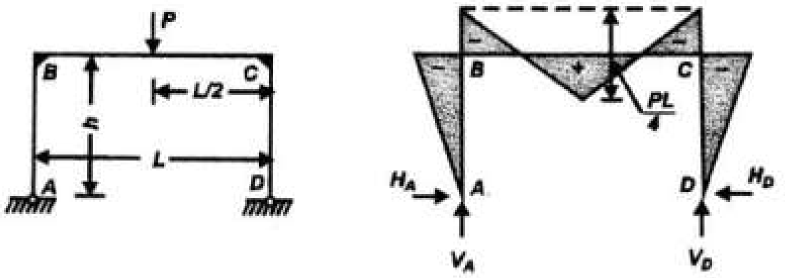


$$M_B = \frac{wh^2}{4}\left[-\frac{k}{2N} + 1\right] \quad , \quad H_D = -\frac{M_C}{h} \quad , \quad M_C = \frac{wh^2}{4}\left[-\frac{k}{2N} - 1\right] \quad , \quad H_A = -(wh - H_D)$$

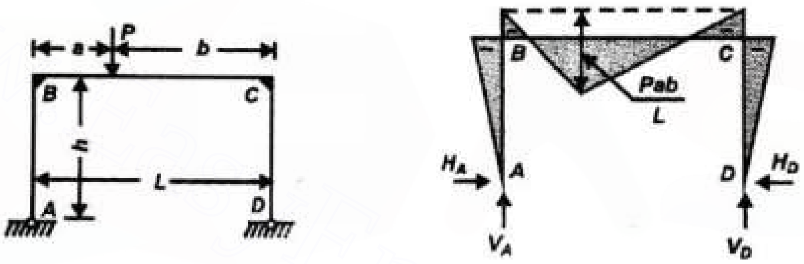
$$V_A = -V_D = -\frac{wh^2}{2L}$$

APPENDIX - D - 9 Formulae for Portal Frames

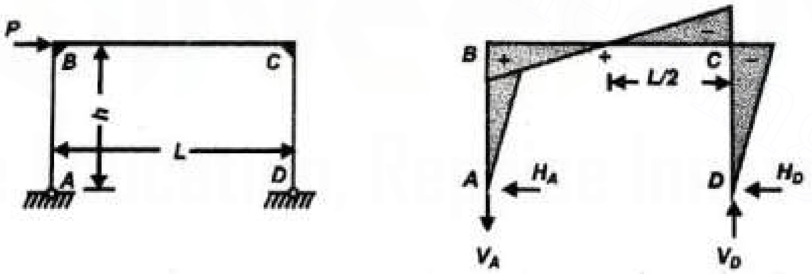
Formulae for Hinged Based Rectangular Portal Frames (Continued)



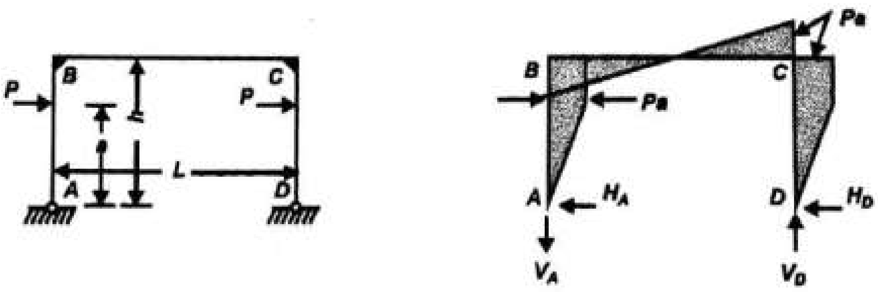
$M_B = M_C = -\frac{3PL}{8N}$, $V_A = V_D = \frac{PL}{2}$, $H_A = H_D = -\frac{M_B}{h}$



$M_B = M_C = -\frac{Pab}{L} \times \frac{3}{2N}$, $V_A = \frac{Pb}{L}$, $V_D = \frac{Pa}{L}$, $H_A = H_D = -\frac{M_B}{h}$



$M_B = -M_C = +\frac{Ph}{2}$, $V_A = -V_D = \frac{-Ph}{L}$, $H_A = H_D = -\frac{P}{2}$

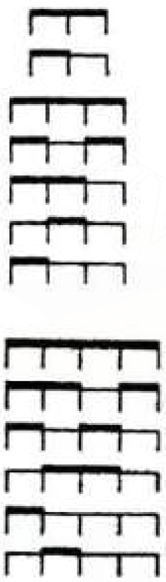
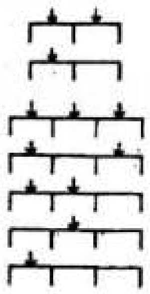


$M_B = -M_C = Pa$, $H_A = H_D = P$, $V_A = -V_D = -\frac{2Pa}{L}$

APPENDIX -D -10 Coefficients for Continuous Beam

Table D-10. Continuous Beam with Equal Spans Loaded by Uniformly Distributed Load, Maximum Bending Moment Coefficients without Redistribution of moments.

(a) B.M. Coefficients for Different Loadings

Loaded Spans	*No of span*	Serial number of span from left end						
		1 *mid-span*	*Sup-port*	2 *mid-span*	*Sup-port*	3 *mid-span*	*sup-port*	4 *mid-span*
	(i)	***Uniformaly Distributed Load w on Loaded Span.***						
	2	0.070	-0.125	0.070	0	-	-	-
	2	0.096	-0.063	-0.031	0	-	-	-
	3	0.080	-0.100	0.025	-0.100	0.080	0	-
	3	0.101	-0.050	-0.050	-0.050	0.101	0	-
	3	0.073	-0.117	0.054	-0.033	-0.017	0	-
	3	-0.025	-0.050	0.075	-0.050	-0.025	0	-
	3	0.094	-0.067	-0.025	0.017	0.009	0	-
	4	0.077	-0.107	0.036	-0.071	0.036	-0.107	0.077
	4	0.072	-0.121	0.061	-0.018	-0.038	-0.058	0.098
	4	0.100	-0.054	-0.045	-0.036	0.081	-0.054	-0.027
	4	-0.018	-0.036	0.056	-0.107	0.056	-0.036	-0.018
	4	0.094	-0.067	-0.025	0.018	0.007	-0.004	-0.002
	4	-0.025	-0.049	0.074	-0.054	-0.020	0.014	0.007
	(ii)	***Central point Load W on Loaded Span***						
	2	0.156	-0.188	0.156	0	-	-	-
	2	0.203	-0.094	-0.047	0	-	-	-
	3	0.175	-0.150	0.100	-0.150	0.175	0	-
	3	0.213	-0.075	-0.075	-0.075	0.213	0	-
	3	0.163	-0.175	0.138	-0.050	-0.025	0	-
	3	-0.038	-0.075	0.175	-0.075	-0.038	0	-
	3	0.200	-0.100	-0.038	0.025	0.013	0	-

Note :- Bending Moment = *B.M. Coef x $W_u L$* *Shear Force* = *S.F. Coef x W_u*

Where, W_u = *Total Ultimate Load on beam* and L = *Span*

Table D-11 Effect of End Moment on Intermediate Spans of a Continuous Beam.

No. of Spans.	*Support Moment Coefficents.*					
	Serial No. of Support next to end support					
	End Support	1	2	3	4	5
2	1.0	-0.250	0	---	---	---
3	1.0	-0.267	+0.067	0	---	---
4	1.0	-0.268	+0.071	-0.018	0	---
5	1.0	-0.268	+0.072	-0.019	-0.005	0

Table D-12 **Bending moment and Shear force coefficients for Continuous Beam / Slab with three or more Equal Spans. (as per IS:456-2000)**

(a) Bending Moment Coefficients :

		End Support		Penultimate support		Interior supports →		
DL	α_d	0	+1/12	– 1/10	+1/16	– 1/12	+1/16	-1/12
LL	α_L	0	+1/10	– 1/9	+1/12	– 1/9	+1/12	-1/9

(b) Shear Coefficients :

DL	0.4	0.6	0.55	0.5	0.5	0.5
LL	0.45	0.6	0.6	0.6	0.6	0.6

Notes :

(1) DL = Dead load , LL = Live load or imposed load not fixed.

(2) For obtaining the bending moment, the BM coefficients shall be multiplied by the total design load and span

(3) For obtaining the shear force, the shear force coefficients shall be multiplied by the total design load.

(4) These coefficients are applicable for three or more spans which do not differ by more than 15% of the longest span. In other cases exact analysis should be made.

(5) At supports where two unequal spans meet or where the spans are not equally loaded, the average of the two values for the negative moment at the support may by taken for design.

(6) When coefficients given in the above Table are used for calculation of bending moment redistribution of moments shall not be permitted.

Table D-13 **Bending moment and Shear force coefficients for Continuous Beam / Slab with three or more Equal Spans - Conventional Practice *(Approximate Method)***

(a) Bending Moment Coefficients :

	End Support		*Penultimate support*		*Interior supports* →		
Bending Moment Coef.:	*0*	*+1/10*	*- 1/10*	*+1/12*	*- 1/12*	*+1/12*	*- 1/12*
Shear force Coeff. :	| 0.45		0.60 | 0.55		0.50 | 0.50		0.50 | 0.50

Notes:

(1) For obtaining the bending moment, the BM coefficients shall be multiplied by the total design load and span

(2) For obtaining the shear force, the shear force coefficients shall be multiplied by the total design load.

(3) Total design load for Limits State Method = $W_u L = 1.5\ WL$

(4) These coefficients are applicable for three or more spans which do not differ by more than 15% of the longest span. In other cases exact analysis should be made.

(5) When bending moments given in the above Table are used for calculation of bending moment redistribution of moments shall not be permitted.

Table D-14 **Bending moment Coefficients for Rectangular Two-way Slab - Simply Supported on Four Sides**

L_y/L_x	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	2.5	3.0
α_x	0.062	0.074	0.084	0.093	0.099	0.104	0.113	0.118	0.122	0.124
α_y	0.062	0.061	0.059	0.055	0.051	0.046	0.037	0.029	0.020	0.014

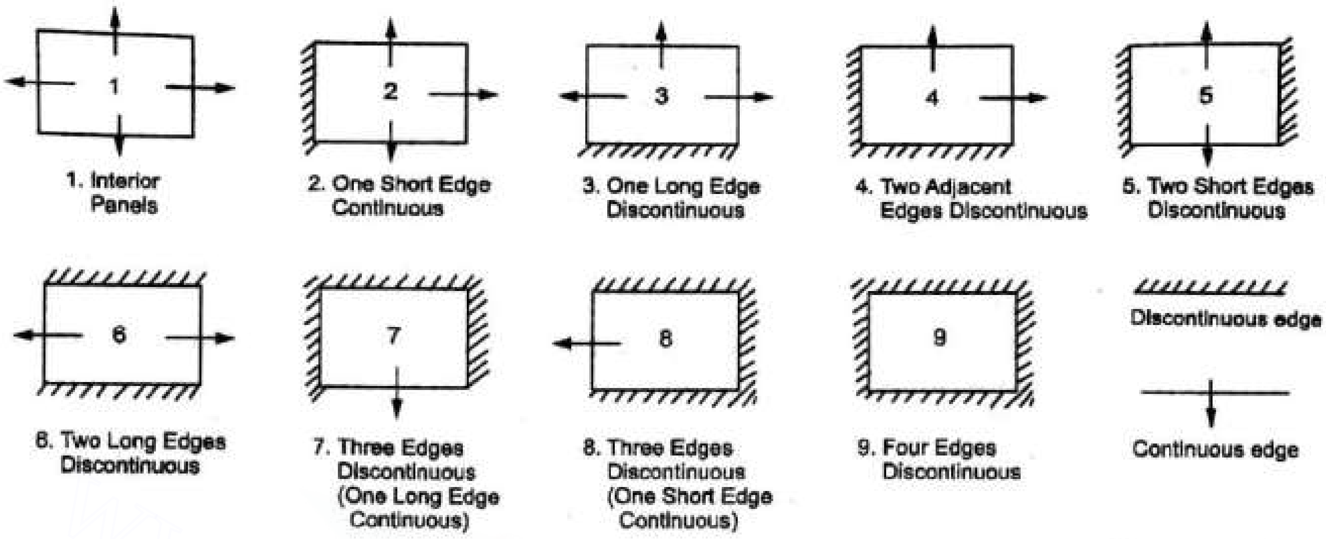
***Note** : At least 50% of the tension reinforcement provided at mid - span should extend to the supports. The remaining 50% should extend to within $0.1L_x$ or $0.1\ L_y$ from the support, as appropriate.*

Table D-15 **Bending Moment Coefficient for Rectangular Slabs Supported on Four Sides with Provision for Torsion at Corners.**

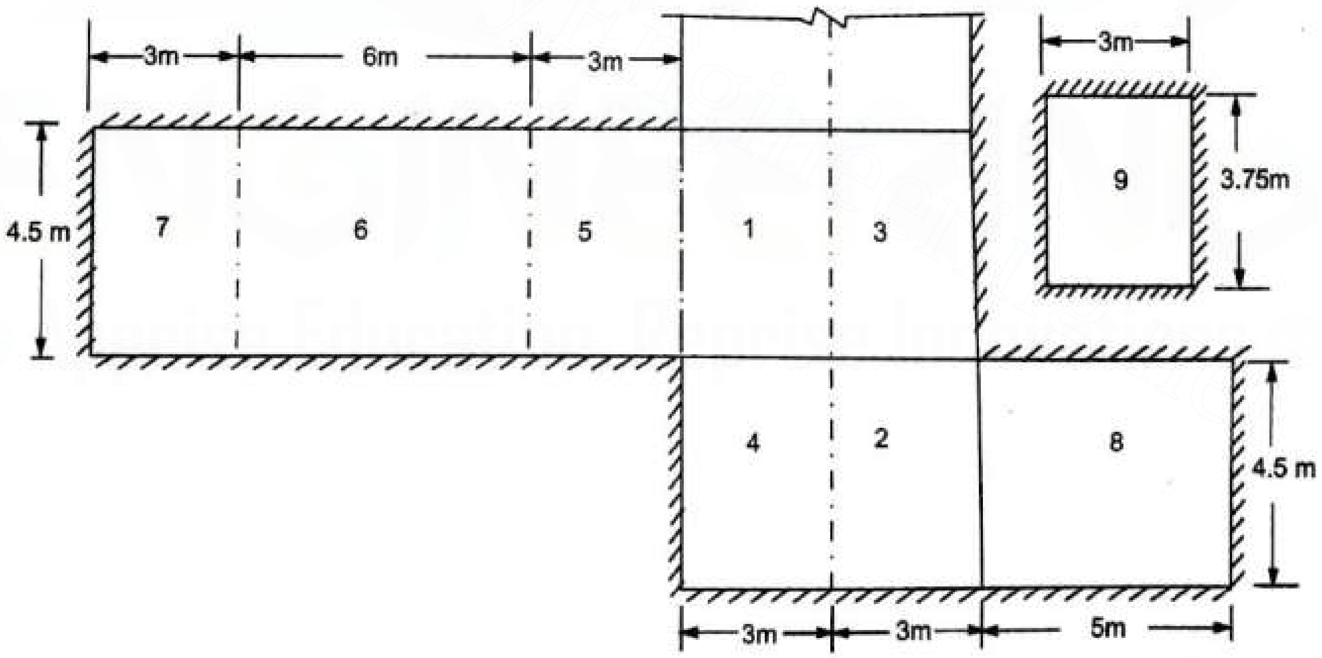
Case No.	*Types of Panel and Moments Considered*	*Short Span Moments Coefficients α_x Values of L_y/L_x*								*Long Span Coef. α_y for all Values of L_y/L_x*
		1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
1	*Interior Panels* -ve moment at continuous edge +ve moment at mid - span	0.032 0.024	0.037 0.028	0.043 0.032	0.047 0.036	0.051 0.039	0.053 0.041	0.060 0.045	0.065 0.049	0.032 0.024
2	*One short Edge Discontinuous* -ve moment at continuous edge +ve moment at mid - span	0.037 0.028	0.043 0.032	0.048 0.036	0.051 0.039	0.055 0.041	0.057 0.044	0.064 0.048	0.068 0.052	0.037 0.028
3	*One Long Edge Discontinuous* -ve moment at continuous edge +ve moment at mid - span	0.037 0.028	0.044 0.033	0.052 0.039	0.057 0.044	0.063 0.047	0.067 0.051	0.077 0.059	0.085 0.065	0.037 0.028
4	*Two Adjacent Edges Discontinuous* -ve moment at continuous edge +ve moment at mid - span	0.047 0.035	0.053 0.040	0.060 0.045	0.065 0.049	0.071 0.053	0.075 0.056	0.084 0.063	0.091 0.069	0.047 0.035
5	*Two Short Edges Discontinuous* -ve moment at continuous edge +ve moment at mid - span	0.045 0.035	0.049 0.037	0.052 0.040	0.056 0.043	0.059 0.044	0.060 0.045	0.065 0.049	0.069 0.052	– 0.035
6	*Two Long Edges Discontinuous* -ve moment at continuous edge +ve moment at mid - span	– 0.035	– 0.043	– 0.051	– 0.057	– 0.063	– 0.068	– 0.080	– 0.088	0.045 0.035
7	*Three Edges Discontinuous (One Long Edge Continuous)* -ve moment at continuous edge +ve moment at mid - span	0.057 0.043	0.064 0.048	0.071 0.053	0.076 0.057	0.080 0.060	0.084 0.064	0.091 0.069	0.097 0.073	– 0.043
8	*Three Edges Discontinuous (One Short Edge Continuous)* -ve moment at continuous edge +ve moment at mid - span	– 0.043	– 0.051	– 0.059	– 0.065	– 0.071	– 0.076	– 0.087	– 0.096	0.057 0.043
9	*Four Edges Discontinuous* +ve moment at mid - span	0.056	0.064	0.072	0.079	0.085	0.089	0.100	0.107	0.056

***Note :-** Ultimate Moment per unit width is given by :*
$M_x = \alpha_x w_u L_x^2$, $M_y = \alpha_y w_u L_x^2$, L_x = *Shorter span* , L_y = *Longer span*
Various support conditions given above are illustrated in Fig. D-1

The various types of support conditions given in *Table D-7* are illustrated diagrammatically below.



(a) Different Types of Support Conditions for Rectangular Two - way Slabs



(b) Practical Illustration of the Support Conditions

Fig. D-1 The various types of support conditions given in *Table D-7* are illustrated diagrammatically above.

APPENDIX - E TABLES FOR SLAB

Table E-1 **Maximum Span for Given Depth of slabs satisfying Requirements of Deflection -One way Slabs having Spans upto 10 *m* and two way Slabs of shorter span, L_x more than 3.5 *m* but less than 10 *m* and loading class greater than 3 *kN/m*² AND Grade of Steel Fe 415**

(a) Simply Supported Slab (Values of Span in m)

Steel p_t %	Effective Depth of Slab 'd' in mm							
	80	90	100	110	120	130	140	150
0.12	3.10	3.48	3.87	4.26	4.65	5.03	5.42	5.81
0.15	2.94	3.31	3.68	4.05	4.42	4.78	5.15	5.52
0.20	2.69	3.02	3.36	3.70	4.03	4.37	4.70	5.04
0.30	2.38	2.68	2.98	3.28	3.58	3.87	4.17	4.47
0.40	2.13	2.39	2.66	2.93	3.19	3.46	3.72	3.99
0.45	2.04	2.30	2.55	2.81	3.06	3.32	3.57	3.83
0.50	1.95	2.20	2.44	2.68	2.93	3.17	3.42	3.66
0.60	1.87	2.11	2.34	2.57	2.81	3.04	3.28	3.51
0.70	1.76	1.98	2.20	2.42	2.64	2.86	3.08	3.30
0.72	1.75	1.97	2.18	2.40	2.62	2.84	3.06	3.28

(b) Continuous Slab (Values of Span in m)

Steel p_t %	Effective Depth of Slab 'd' in mm							
	80	90	100	110	120	130	140	150
0.12	4.03	4.53	5.03	5.54	6.04	6.54	7.05	7.55
0.15	3.83	4.31	4.78	5.26	5.74	6.22	6.70	7.18
0.20	3.49	3.93	4.37	4.80	5.24	5.68	6.12	6.55
0.30	3.10	3.49	3.87	4.26	4.65	5.04	5.42	5.81
0.40	2.77	3.11	3.46	3.80	4.15	4.50	4.84	5.19
0.45	2.65	2.98	3.32	3.65	3.98	4.31	4.64	4.97
0.50	2.54	2.85	3.17	3.49	3.81	4.12	4.44	4.76
0.60	2.43	2.74	3.04	3.35	3.65	3.95	4.26	4.56
0.70	2.29	2.57	2.86	3.15	3.43	3.72	4.00	4.29
0.72	2.27	2.56	2.84	3.12	3.41	3.69	3.97	4.26

(c) Cantilever Slab (Values of Span in m)

Steel p_t %	Effective Depth of Slab 'd' in mm							
	80	90	100	110	120	130	140	150
0.12	1.08	1.22	1.36	1.49	1.63	1.76	1.90	2.03
0.15	1.03	1.16	1.29	1.42	1.55	1.67	1.80	1.93
0.20	0.94	1.06	1.18	1.29	1.41	1.53	1.65	1.76
0.30	0.83	0.94	1.04	1.15	1.25	1.36	1.46	1.56
0.40	0.74	0.84	0.93	1.02	1.12	1.21	1.30	1.40
0.45	0.71	0.80	0.89	0.98	1.07	1.16	1.25	1.34
0.50	0.68	0.77	0.85	0.94	1.02	1.11	1.20	1.28
0.60	0.66	0.74	0.82	0.90	0.98	1.06	1.15	1.23
0.70	0.62	0.69	0.77	0.85	0.92	1.00	1.08	1.15
0.72	0.61	0.69	0.76	0.84	0.92	0.99	1.07	1.15

Notes : *(1) These Tables have been prepared using Fig. 4.71,(or fig. 4 of code) in which the curve for f_s = 240 N/mm² has been used for f_y = 415 N/mm² (For details see Ref. 6.1 , 6.2 given in Chap. 6)*

(2) Required Total depth = d + Nominal Cover + Dia. of main steel/2

(3) Nominal cover to be obtained from Table-C-1

Table E-2A **Ultimate Moment of Resistance (in kN.m) of Slabs of Different Total Depths for given Diameter-Spacing Combinations of Bars - Steel Fe 415 Mild Environment- Concrete M20, *Nominal cover 15 mm*.**

M 20, Fe 415
Cover = 15 mm

Steel			*Total depth D of slab*						
Dia *mm*	Spacing *mm*	Area mm^2	100 *mm*	110 *mm*	120 *mm*	130 *mm*	140 *mm*	150 *mm*	160 *mm*
8	100	502.7	12.80	14.61	16.43	18.24	20.06	21.87	23.68
8	110	457.0	11.79	13.44	15.09	16.74	18.39	20.04	21.69
8	120	418.9	10.93	12.44	13.95	15.46	16.98	18.49	20.00
8	125	402.1	10.54	11.99	13.45	14.90	16.35	17.80	19.25
8	130	386.7	10.18	11.58	12.97	14.37	15.76	17.16	18.55
8	140	359.0	9.53	10.83	12.12	13.42	14.71	16.01	17.30
8	150	335.1	8.95	10.16	11.37	12.58	13.79	15.00	16.21
8	160	314.2	8.44	958	10.71	11.85	12.98	14.11	15.25
8	170	295.7	7.99	9.06	10.12	11.19	12.26	13.32	14.39
8	175	287.2	7.78	8.81	9.85	10.89	11.92	12.96	14.00
8	180	279.3	7.58	8.59	9.59	10.60	11.61	12.62	13.63
8	190	264.6	7.21	8.16	9.12	10.07	11.03	11.98	12.94
8	200	251.3	6.87	7.78	8.69	9.59	10.50	11.41	12.32
8	210	239.4	6.57	7.43	8.30	9.16	10.02	10.89	11.75
8	220	228.5	6.29	7.11	7.94	8.76	9.59	10.41	11.23
8	225	223.4	6.16	6.96	7.77	8.57	9.38	10.09	10.99
8	230	218.5	6.03	6.82	7.61	8.40	9.19	9.97	10.76
8	240	209.4	5.79	6.55	7.31	8.06	8.82	9.57	10.33
8	250	201.1	5.57	6.30	7.03	7.75	8.48	9.20	9.93
8	260	193.3	5.37	6.07	6.77	7.46	8.16	8.86	9.56
8	270	186.2	5.18	5.85	6.53	7.20	7.87	8.54	9.21
8	275	182.8	5.09	5.75	6.41	7.07	7.73	8.39	9.05
8	280	179.5	5.01	5.65	6.30	6.95	7.60	8.25	8.89
8	290	173.3	4.84	5.47	6.09	6.72	7.34	7.97	8.59
8	300	167.6	4.69	5.29	5.90	6.50	7.11	7.71	0.00
Effective depth *d mm*			81	91	101	111	121	131	141
10	100	785.4	0.00	20.39	23.72	26.56	29.39	32.23	35.06
10	110	714.0	16.80	19.37	21.95	24.53	27.10	29.68	32.42
10	120	654.5	15.69	18.05	20.41	22.77	25.13	27.50	29.86
10	125	628.3	15.18	17.45	19.72	21.99	24.25	26.52	28.79
10	130	604.2	14.71	16.89	19.07	21.25	23.43	25.61	27.79
10	140	561.0	13.84	15.86	17.89	19.91	21.94	23.96	25.99
10	150	523.6	13.06	14.95	16.84	18.73	20.62	22.51	24.40
10	160	490.9	12.37	14.14	15.91	17.58	19.45	21.22	23.00
10	170	462.0	11.74	13.41	15.07	16.74	18.41	20.08	21.74
10	175	448.8	11.45	13.07	14.69	16.31	17.93	19.55	21.17

Table E-2A *Ultimate Moment of Resistance (in kN.m) of Slab Continued...*

M 20 , Fe 415
Cover 15 mm

Steel			Total depth D of slab						
Dia mm	Spacing mm	Area mm^2	100 mm	110 mm	120 mm	130 mm	140 mm	150 mm	160 mm
10	180	436.3	11.17	12.75	14.32	15.89	17.47	19.04	20.62
10	190	413.4	10.65	12.15	13.64	15.13	16.62	18.11	19.60
10	200	392.7	10.18	11.60	13.02	14.43	15.85	17.27	18.69
10	210	374.0	9.75	11.10	12.45	13.80	15.15	16.50	17.85
10	220	357.0	9.35	10.64	11.93	13.22	14.51	15.79	17.08
10	225	349.1	9.16	10.42	11.68	12.94	14.20	15.46	16.72
10	230	341.5	8.99	10.22	11.45	12.68	13.91	15.15	16.38
10	240	327.2	8.65	9.83	11.01	12.19	13.37	14.55	15.73
10	250	314.2	8.33	9.46	10.60	11.73	12.87	14.00	15.13
10	260	302.1	8.04	9.13	10.22	11.31	12.40	13.49	14.58
10	270	290.9	7.76	8.81	9.86	10.91	11.96	13.01	14.06
10	275	285.6	7.63	8.66	9.70	10.73	11.76	12.79	13.82
10	280	280.5	7.51	8.52	9.53	10.55	11.56	12.57	13.58
10	290	270.8	7.27	8.25	9.22	10.20	11.18	12.16	13.13
10	300	261.8	7.04	7.99	8.93	9.88	10.82	11.77	12.71
Effective depth d mm			80	90	100	110	120	130	140
12	100	1131.0	0.00	0.00	0.00	0.00	38.99	43.07	47.15
12	110	1028.2	0.00	0.00	0.00	32.24	36.95	39.95	43.66
12	120	942.5	0.00	0.00	27.02	30.42	33.82	37.22	40.62
12	125	904.8	0.00	0.00	26.19	29.46	32.72	35.99	39.25
12	130	870.0	0.00	0.00	25.41	28.55	31.69	34.83	37.97
12	140	807.8	0.00	21.06	23.97	26.89	29.80	32.72	35.64
12	150	754.0	17.24	19.96	22.68	25.40	28.12	30.84	33.56
12	160	706.9	16.41	18.96	21.51	24.06	26.61	29.16	31.72
12	170	665.3	15.65	18.05	20.45	22.85	25.26	27.66	30.06
12	175	646.3	15.30	17.63	19.96	22.29	24.63	26.96	29.29
12	180	628.3	14.96	17.22	19.49	21.76	24.03	26.29	28.56
12	190	595.2	14.32	16.46	18.61	20.76	22.91	25.06	27.21
12	200	565.5	13.73	15.77	17.81	19.85	21.89	23.93	25.97
12	210	538.6	13.18	15.13	17.07	19.01	21.96	22.90	24.84
12	220	51431	12.68	14.53	16.39	18.24	20.10	21.95	23.81
12	225	502.7	12.44	14.25	16.07	17.88	19.67	21.51	23.32
12	230	491.7	12.21	13.98	15.76	17.53	19.31	21.08	22.85
12	240	471.2	11.77	13.47	15.17	16.87	18.57	20.27	21.97
12	250	452.4	11.36	13.00	14.63	16.26	17.89	19.53	21.16
12	260	435.0	1098	12.55	14.12	15.69	17.26	18.83	20.40
12	270	418.9	10.63	12.14	13.65	15.16	16.67	18.19	19.70
12	275	411.3	10.46	11.94	13.43	14.91	16.39	17.88	19.36
12	280	403.9	10.29	11.75	13.21	14.67	16.12	17.58	19.04

Table E-2A *Ultimate Moment of Resistance (in kN.m) of Slab Continued...*

M 20, Fe 415
Cover 15 mm

Steel Dia mm	Spacing mm	Area mm^2	Total depth D 100 mm	110 mm	120 mm	130 mm	140 mm	150 mm	160 mm
12	290	390.0	9.98	11.37	12.79	14.20	15.61	17.02	18.42
12	300	377.0	9.68	11.04	12.40	13.76	15.13	16.49	17.85
Effective depth d mm			79	89	99	109	119	129	139
16	100	2010.6	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16	110	1827.8	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16	120	1675.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16	125	1608.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16	130	1546.5	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16	140	1436.2	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16	150	1340.4	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16	160	1256.6	0.00	0.00	0.00	0.00	0.00	0.00	50.30
16	170	1182.7	0.00	0.00	0.00	0.00	0.00	43.73	48.00
16	175	1148.9	0.00	0.00	0.00	0.00	0.00	42.77	46.92
16	180	1117.0	0.00	0.00	0.00	0.00	37.82	41.85	45.88
16	190	1058.2	0.00	0.00	0.00	0.00	36.29	40.11	43.93
16	200	1005.3	0.00	0.00	0.00	31.25	34.88	38.51	42.13
16	210	957.4	0.00	0.00	0.00	30.11	33.56	37.02	40.47
16	220	913.9	0.00	0.00	25.74	29.03	32.33	35.63	38.93
16	225	893.6	0.00	0.00	25.30	28.53	31.75	34.97	38.20
16	230	874.2	0.00	0.00	24.88	28.03	31.19	34.34	37.50
16	240	837.8	0.00	0.00	24.07	27.09	30.12	33.14	36.16
16	250	804.2	0.00	20.41	23.31	26.21	29.11	32.02	34.92
16	260	773.3	0.00	19.80	22.59	25.38	28.83	30.96	33.75
16	270	744.7	0.00	19.23	21.91	24.60	27.29	29.98	32.66
16	275	731.1	16.31	18.95	21.59	24.23	26.87	29.51	32.14
16	280	718.1	16.09	18.68	21.27	23.87	26.46	29.05	31.64
16	290	693.3	15.67	18.17	20.67	23.17	25.67	28.18	30.68
16	300	670.2	15.26	17.68	20.10	22.52	24.93	27.35	29.77
Effective depth d mm			77	87	97	107	117	127	137

Note : *Values below the zig-zag lines execeed the permissible spacing of 3d and therefore may be considered for extra steel at support.*

Table E-2B **Ultimate Moment of Resistance (in kN.m) of Slabs of Different Total Depths for given Diameter-Spacing Combinations of Bars - Steel Fe 415**
Mild Environment - Concrete M20
***Nominal cover 20mm* to meet o.5 hour of fire resistance.**

M 20, Fe 415
Cover = 20 mm

Steel			*Total depth D of Slab*						
Dia *mm*	Spacing *mm*	Area mm^2	100 *mm*	110 *mm*	120 *mm*	130 *mm*	140 *mm*	150 *mm*	160 *mm*
#8	100	502.7	11.89	13.71	15.52	17.34	19.15	20.96	22.78
#8	110	457.0	10.97	12.62	14.27	15.92	17.57	19.21	20.86
#8	120	418.9	10.17	11.69	13.20	14.71	16.22	17.73	19.24
#8	125	402.1	9.82	11.27	12.72	14.17	15.62	17.07	18.52
#8	130	386.7	9.49	10.88	12.28	13.67	15.07	16.46	17.86
#8	140	359.0	8.88	10.18	11.47	12.77	14.06	15.36	16.66
#8	150	335.1	8.35	9.56	10.77	11.98	13.19	14.40	15.61
#8	160	314.2	7.88	9.01	10.14	11.28	12.41	13.55	14.68
#8	170	295.7	7.45	8.52	9.59	10.66	11.72	12.79	13.86
#8	175	287.2	7.26	8.30	9.33	10.37	11.41	12.44	13.48
#8	180	279.3	7.07	8.08	9.09	10.10	11.11	12.11	13.12
#8	190	264.6	6.73	7.69	8.64	9.60	10.55	11.51	12.46
#8	200	251.3	6.42	7.33	8.23	9.14	10.05	10.95	11.86
#8	210	239.4	6.14	7.00	7.86	8.73	9.59	10.45	11.32
#8	220	228.5	5.88	6.70	7.52	8.35	9.17	10.00	10.82
#8	225	223.4	5.75	6.56	7.37	8.17	8.98	9.78	10.59
#8	230	218.5	5.64	6.42	7.21	8.00	8.79	9.58	10.37
#8	240	209.4	5.42	6.17	6.93	7.68	8.44	9.19	9.95
#8	250	201.1	5.21	5.94	6.66	7.39	8.11	8.84	9.57
#8	260	193.3	5.02	5.72	6.42	7.12	7.81	8.51	9.21
#8	270	186.2	4.85	5.52	6.19	6.86	7.53	8.21	8.88
#8	275	182.8	4.76	5.42	6.08	6.74	7.40	8.06	8.72
#8	280	179.5	4.68	5.33	5.98	6.63	7.27	7.92	8.57
#8	290	173.3	4.53	5.15	5.78	6.41	7.03	7.66	8.28
#8	300	167.6	4.39	4.99	5.59	6.20	6.80	7.41	8.01
Effective depth d mm			76	86	96	106	116	126	136
#10	100	785.4	16.64	19.47	22.31	25.14	27.98	30.81	33.64
#10	110	714.0	15.51	18.08	20.66	23.24	25.81	28.39	30.97
#10	120	654.5	14.51	16.87	19.23	21.59	23.95	26.32	28.68
#10	125	628.3	14.05	16.32	18.58	20.85	23.12	25.39	27.65
#10	130	604.2	13.62	15.80	17.98	20.16	22.34	24.52	26.70
#10	140	561.0	12.83	14.85	16.88	18.90	20.92	22.95	24.97
#10	150	523.6	12.12	14.01	15.90	17.79	19.68	21.57	23.46
#10	160	490.9	11.48	13.25	15.02	16.80	18.57	20.34	22.11
#10	170	492.0	10.91	12.57	14.24	15.91	17.57	19.24	20.91
#10	175	448.8	10.64	12.26	13.88	15.50	17.12	18.74	20.36
#10	180	436.3	10.38	11.96	13.53	15.11	16.68	18.26	19.83
#10	190	413.4	9.91	11.40	12.89	14.38	15.88	17.37	18.86
#10	200	392.7	9.47	10.89	12.31	13.73	15.14	16.56	17.98
#10	210	374.0	9.07	10.42	11.77	13.12	14.47	15.82	17.17
#10	220	357.0	8.71	10.00	11.28	12.57	13.86	15.15	16.44
#10	225	349.1	8.54	9.79	11.05	12.31	13.57	14.83	16.09
#10	230	341.5	8.37	9.60	10.83	12.07	13.30	14.53	15.76
#10	240	327.2	8.06	9.24	10.42	11.60	12.78	13.96	15.14
#10	250	314.2	7.76	8.90	10.03	11.16	12.30	13.43	14.57
#10	260	302.1	7.49	8.58	9.67	10.76	11.85	12.94	14.03
#10	270	290.9	7.24	8.29	9.34	10.39	11.44	12.49	13.54
#10	270	285.6	7.12	8.15	9.18	10.21	11.24	12.27	13.30
#10	275	280.5	7.00	8.01	9.03	10.04	11.05	12.06	13.08
#10	280	270.8	6.78	7.76	8.74	9.71	10.69	11.67	12.64

Table E-2B *Ultimate Moment of Resistance (in kN.m) of Slabs Continued . . .*

M 20, Fe 415
Cover = 20 mm

Steel			Total depth D of Slab						
Dia mm	Spacing mm	Area mm^2	100 mm	110 mm	120 mm	130 mm	140 mm	150 mm	160 mm
#10	300	261.8	6.57	07.52	08.46	09.41	10.35	11.30	12.24
Effective depth d mm			75	85	95	105	115	125	135
#12	100	1131.0	–	–	–	–	36.95	41.03	45.11
#12	110	1028.2	–	–	–	–	34.38	38.09	41.80
#12	120	942.5	–	–	–	28.72	32.12	35.52	38.92
#12	125	904.5	–	–	–	27.83	31.09	34.36	37.62
#12	130	870.0	–	–	23.84	26.98	30.12	33.26	36.40
#12	140	807.8	–	–	22.52	25.43	28.35	31.26	34.18
#12	150	754.0	–	18.60	21.32	24.04	26.76	29.48	32.20
#12	160	706.9	15.13	17.69	20.24	22.79	25.34	27.89	30.44
#12	170	665.3	14.45	16.85	19.25	21.65	24.05	26.46	28.86
#12	175	646.3	14.13	16.46	18.80	21.13	23.46	25.79	28.12
#12	180	628.3	13.82	16.09	18.36	20.62	22.89	25.16	27.43
#12	190	595.2	13.24	15.39	17.54	19.69	21.83	23.98	26.13
#12	200	565.5	12.71	14.75	16.79	18.83	20.87	22.91	24.95
#12	210	538.6	12.21	14.15	16.10	18.04	19.98	21.93	23.87
#12	220	514.1	11.75	13.60	15.46	17.31	19.17	21.02	22.88
#12	225	502.7	11.53	13.35	15.16	16.97	18.79	20.60	22.41
#12	230	491.7	11.32	13.10	14.87	16.64	18.42	20.19	21.97
#12	240	471.2	10.92	12.62	14.32	16.02	17.72	19.42	21.12
#12	250	452.4	10.55	12.18	13.81	15.45	17.08	18.71	20.34
#12	260	435.0	10.20	11.77	13.34	14.91	16.48	18.05	19.62
#12	270	418.9	9.87	11.38	12.90	14.41	15.92	17.43	18.94
#12	275	411.3	9.72	11.20	12.68	14.17	15.65	17.14	18.62
#12	280	403.9	9.56	11.02	12.48	13.94	15.40	16.85	18.31
#12	290	390.0	9.28	10.68	12.09	13.50	14.91	16.31	17.72
#12	300	377.0	9.00	10.36	11.72	13.08	14.44	15.81	17.17
Effective depth d mm			74	84	94	104	114	124	134
#16	100	2010.6	–	–	–	–	–	–	–
#16	110	1827.8	–	–	–	–	–	–	–
#16	120	1675.5	–	–	–	–	–	–	–
#16	125	1608.5	–	–	–	–	–	–	–
#16	130	1546.6	–	–	–	–	–	–	–
#16	140	1436.2	–	–	–	–	–	–	–
#16	150	1340.4	–	–	–	–	–	–	–
#16	160	1256.6	–	–	–	–	–	–	48.03
#16	170	1182.7	–	–	–	–	–	–	45.86
#16	175	1148.9	–	–	–	–	–	–	44.84
#16	180	1117.0	–	–	–	–	–	39.83	43.87
#16	190	1058.2	–	–	–	–	34.39	38.20	42.02
#16	200	1005.3	–	–	–	–	33.06	36.69	40.32
#16	210	957.4	–	–	–	28.38	31.83	35.29	38.74
#16	220	913.9	–	–	–	27.39	30.68	33.98	37.28
#16	225	893.6	–	–	–	26.91	30.14	33.36	36.59
#16	230	874.2	–	–	23.30	26.46	29.61	32.76	35.92
#16	240	837.8	–	–	22.56	25.58	28.60	31.63	34.65
#16	250	804.2	–	18.96	21.86	24.76	27.66	30.56	33.47
#16	260	773.3	–	18.41	21.20	23.99	26.78	29.57	32.36
#16	270	744.7	–	17.88	20.57	23.26	25.95	28.63	31.32
#16	275	731.1	–	17.63	20.27	22.91	25.55	28.19	30.82
#16	280	718.1	–	17.39	19.98	22.57	25.16	27.75	30.34
#16	290	693.3	–	16.92	19.42	21.92	24.42	26.92	29.43
#16	300	670.2	14.05	16.47	18.89	21.31	23.72	26.14	28.56
Effective depth d mm			72	82	92	102	112	122	132

Note : *Values below the zig-zag lines exceed the permissible spacing of 3d and therefore may be considered for extra steel at support.*

APPENDEX - F TABLES FOR BEAM

Table F-1 **Ultimate Moment of Resistance (M_{ur}) in kN.m of Rectangular Beam 230mm wide for Different Depths and for given Number - Diameter combination of Bars Mild Environment – Under - Reinforced Design.**

Values of M_{ur} in kN.m

M 20, Fe 415, b = 230 mm

N1-# N2-#	A_{st}	Cover	RO	Depth of the beam in mm								
				300	350	380	400	450	500	530	550	600
2-#12+ 0-# 0	226.2	32	1	20.21	24.29	26.74	28.37	32.45	36.54	–	–	–
3-#10+ 0-# 0	235.6	31	1	21.07	25.32	27.87	29.57	33.82	38.07	40.62	–	–
3-#12+ 0-# 0	339.3	32	1	29.07	35.19	38.86	41.31	47.43	53.55	57.23	59.68	65.80
2-#16+ 0-# 0	402.1	34	1	33.34	40.59	44.94	47.85	55.10	62.36	66.71	69.61	76.87
3-#12+ 1-#10	417.8	32	1	34.73	42.27	46.79	49.80	57.34	64.88	69.41	72.42	79.96
4-#12+ 0-# 0	452.4	32	1	37.09	45.25	50.15	53.41	61.58	69.74	74.64	77.90	86.07
2-#16+ 1-#10	480.7	34	1	38.62	47.29	52.49	55.96	64.64	73.31	78.51	81.98	90.65
2-#16+ 1-#12	515.2	34	1	40.81	50.11	55.69	59.41	68.70	78.00	83.58	87.30	96.59
2-#16+ 2-#10	559.2	34	1	43.50	53.59	59.64	63.68	73.77	83.86	89.91	93.95	104.04
5-#12+ 0-# 0	565.5	32	1	44.28	54.48	60.60	64.69	74.89	85.09	91.21	95.30	105.50
3-#16+ 0-# 0	603.2	34	1	–	56.94	63.47	67.82	78.71	89.59	96.12	100.47	111.36
2-#16+ 2-#12	628.3	34	1	–	58.80	65.60	70.13	81.47	92.81	99.61	104.15	115.48
2-#20+ 0-# 0	628.3	38	1	–	57.89	64.69	69.23	80.56	91.90	98.70	103.24	114.58
2-#20+ 1-#12	741.4	38	1	–	–	73.61	78.96	92.34	105.71	113.74	119.09	132.47
4-#16+ 0-# 0	804.2	34	1	–	–	–	85.17	99.68	114.19	122.90	128.70	143.21
2-#20+ 2-#12	854.5	38	1	–	–	–	–	103.27	118.69	127.94	134.11	149.53
3-#20+ 0-# 0	942.5	38	1	–	–	–	–	–	128.21	138.42	145.22	162.22
2-#25+ 0-# 0	981.7	41	1	–	–	–	–	–	131.41	142.04	149.13	166.84
3-#20+ 1-#12	1055.6	38	1	–	–	–	–	–	–	151.14	158.76	177.80
3-#20+ 2-#12	1168.7	38	1	–	–	–	–	–	–	–	–	192.55
2-#25+ 2-#12	1207.9	41	1	–	–	–	–	–	–	–	–	196.39

Notes : (1) Dashes indicate that the particular Number-Diameter combination of bars is not admissible for the condition of either Minimum steel or Maximum Steel.
(2) Diameter of stirrup assumed is 6mm for bar diameter < = 16mm. For Bar diameter > 16mm diameter of stirrup assumed is 8 mm
(3) For Mild Environment Nominal cover is 20 mm (See Table C-1)

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Table F-2A **Ultimate Moment of Resistance (M_{ur}) in *kN.m* of Singly reinforced and Doubly reinforced Rectangular Beam - 150 *mm* wide Mild Environment - Concrete M20, Steel Fe 415, (for Nominal Cover See *Table C-1*)**

M_{ur} Singly and Doubly Rein.Sect M 20 , Fe 415
b = 150 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{st1} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	$V_{usv.min}$ kN	V_{uc} kN	V_{uc1} kN	$V_{ur.min}$ kN	V_{urmin1} kN
150	300	2 10 + 0 0	A	157	157	–	1	14.01	16.14	17.45	17.45	33.59	33.59
150	300	2 12 + 0 0	n	226	226	–	1	19.32	16.08	20.15	20.15	36.23	36.23
150	300	2 10 + 1 12	c	270	157	–	1	22.48	16.08	21.59	17.41	37.67	33.49
150	300	2 12 + 1 10	h	305	226	–	1	24.83	16.08	22.60	20.15	38.68	36.23
150	300	2 10 + 2 10	o	314	157	–	2	24.14	15.39	22.24	16.96	37.63	32.35
150	300	2 12 + 1 12	r	339	226	–	1	27.06	16.08	23.53	20.15	39.61	36.23
150	300	2 10 + 1 16	Bars	358	157	–	1	27.96	15.96	23.89	17.33	39.85	33.29
150	300	2 10 + 2 12	2 8 + 0 0	383	157	101	2	30.21	15.27	23.80	16.89	39.07	32.16
150	300	2 10 + 2 12	2 10 + 0 0	383	157	157	2	31.04	15.27	23.80	16.89	39.07	32.16
150	300	2 10 + 2 12	2 12 + 0 0	383	157	226	2	31.65	15.27	23.80	16.89	39.07	32.16
150	300	2 10 + 2 12	2 10 + 1 12	383	157	270	2	31.79	15.27	23.80	16.89	39.07	32.16
150	300	2 10 + 2 12	2 10 + 1 16	383	157	358	2	31.78	15.27	23.80	16.89	39.07	32.16
150	300	2 16 + 0 0	2 8 + 0 0	402	402	101	1	33.05	15.96	24.91	24.91	40.87	40.87
150	300	2 16 + 0 0	2 10 + 0 0	402	402	157	1	33.99	15.96	24.91	24.91	40.87	40.87
150	300	2 16 + 0 0	2 12 + 0 0	402	402	226	1	34.71	15.96	24.91	24.91	40.87	40.87
150	300	2 16 + 0 0	2 10 + 1 12	402	402	270	1	35.02	15.96	24.91	24.91	40.87	40.87
150	300	2 16 + 0 0	2 10 + 1 16	402	402	358	1	34.78	15.96	24.91	24.91	40.87	40.87
150	300	2 12 + 1 16	2 8 + 0 0	427	226	101	1	34.67	15.96	25.45	20.06	41.41	36.02
150	300	2 12 + 1 16	2 10 + 0 0	427	226	157	1	35.75	15.96	25.45	20.06	41.41	36.02
150	300	2 12 + 1 16	2 12 + 0 0	427	226	226	1	36.63	15.96	25.45	20.06	41.41	36.02
150	300	2 12 + 1 16	2 10 + 1 12	427	226	270	1	37.01	15.96	25.45	20.06	41.41	36.02
150	300	2 12 + 1 16	2 10 + 0 0	427	226	358	1	36.43	15.96	25.45	20.06	41.41	36.02
150	300	2 12 + 1 12	2 8 + 0 0	427	226	101	2	34.35	15.27	25.24	19.53	41.51	34.80
150	300	2 12 + 2 12	2 10 + 0 0	452	226	157	2	35.57	15.27	25.24	19.53	41.51	34.80
150	300	2 12 + 2 12	2 12 + 0 0	452	226	226	2	36.60	15.27	25.24	19.53	41.51	34.80
150	300	2 12 + 2 12	2 10 + 1 12	452	226	270	2	37.09	15.27	25.24	19.53	41.51	34.80
150	300	3 12 + 1 12	2 8 + 0 0	452	339	101	2	34.35	15.27	25.24	22.77	40.51	38.04
150	300	3 12 + 1 12	2 10 + 0 0	452	339	157	2	35.57	15.27	25.24	22.77	40.51	38.04
150	300	3 12 + 1 12	2 12 + 0 0	452	339	226	2	36.60	15.27	25.24	22.77	40.51	38.04
150	300	3 12 + 1 12	2 10 + 1 0	452	339	270	2	37.09	15.27	25.24	22.77	40.51	38.04
150	300	2 16 + 1 12	2 8 + 0 0	515	402	101	2	36.85	15.00	26.08	23.93	41.08	38.93
150	300	2 16 + 1 12	2 10 + 0 0	515	402	157	2	38.44	15.00	26.08	23.93	41.08	38.93
150	300	2 16 + 1 12	2 12 + 0 0	515	402	226	2	39.89	15.00	26.08	23.93	41.08	38.93
150	300	2 16 + 1 12	2 10 + 1 12	515	402	270	2	40.62	15.00	26.08	23.93	41.08	38.93
150	300	2 16 + 1 12	2 10 + 1 16	515	402	358	2	41.32	15.00	26.08	23.93	41.08	38.93
150	300	3 12 + 1 16	2 8 + 0 0	540	339	101	2	38.04	15.00	26.50	22.51	41.50	37.51
150	300	3 12 + 1 16	2 10 + 0 0	540	339	157	2	39.76	15.00	26.50	22.51	41.50	37.51
150	300	3 12 + 1 16	2 12 + 0 0	540	339	226	2	41.39	15.00	26.50	22.51	41.50	37.51
150	300	3 12 + 1 16	2 10 + 1 12	540	339	270	2	42.23	15.00	26.50	22.51	41.50	37.51
150	300	3 12 + 1 16	2 10 + 1 16	540	339	358	2	43.11	15.00	26.50	22.51	41.50	37.51
150	300	2 16 + 2 12	2 8 + 0 0	628	402	101	2	41.69	15.00	27.84	23.93	42.84	38.93

Table F-2A M_{ur} **of Singly Reinforced and Doubly Reinforced Beam - 150 mm wide Continued ...**

M_{ur} Singly and Doubly Rein.Sect
M 20 , Fe 415
b = 150 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{st1} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	$V_{usv.min}$ kN	V_{uc} kN	V_{uc1} kN	$V_{ur.min}$ kN	V_{urmin1} kN
150	300	2 16 + 2 12	2 10 + 0 0	628	402	157	2	43.89	15.00	27.84	23.93	42.84	38.93
150	300	2 16 + 2 12	2 12 + 0 0	628	402	226	2	46.13	15.00	27.84	23.93	42.84	38.93
150	300	2 16 + 2 12	2 10 + 1 12	628	402	270	2	47.37	15.00	27.84	23.93	42.84	38.93
150	300	2 16 + 2 12	2 10 + 1 16	628	402	358	2	48.97	15.00	27.84	23.93	42.84	38.93
150	300	2 20 + 0 0	2 8 + 0 0	628	628	101	1	44.41	15.72	28.74	28.74	44.46	44.46
150	300	2 20 + 0 0	2 10 + 0 0	628	628	157	1	46.61	15.72	28.74	28.74	44.46	44.46
150	300	2 20 + 0 0	2 12 + 0 0	628	628	226	1	48.86	15.72	28.74	28.74	44.46	44.46
150	300	2 20 + 0 0	2 10 + 1 12	628	628	270	1	50.09	15.72	28.74	28.74	44.46	44.46
150	300	2 20 + 0 0	2 10 + 1 16	628	628	358	1	51.68	15.72	28.74	28.74	44.46	44.46
150	380	2 10 + 0 0	A	157	157	–	1	18.55	20.94	20.32	20.32	41.26	41.26
150	380	2 12 + 0 0	n	226	226	–	1	25.85	20.88	23.58	23.58	44.46	44.46
150	380	2 10 + 1 12	c	270	157	–	1	30.28	20.88	25.32	20.28	46.20	41.16
150	380	2 12 + 1 10	h	305	226	–	1	33.62	20.88	26.55	23.58	47.43	44.46
150	380	2 10 + 2 10	o	314	157	–	2	33.21	20.19	26.32	19.89	46.51	40.08
150	380	2 12 + 1 12	r	339	226	–	1	36.85	20.88	27.68	23.58	48.56	44.46
150	380	2 10 + 1 16		358	157	–	1	38.30	20.76	28.16	20.22	48.92	40.98
150	380	2 10 + 2 12	B	383	157	–	2	38.92	20.07	28.29	19.82	48.36	39.89
150	380	2 16 + 0 0	a	402	402	–	1	42.12	20.76	29.42	29.42	50.18	50.18
150	380	2 12 + 1 16	r	427	226	–	1	44.22	20.76	30.09	23.50	50.85	44.26
150	380	2 12 + 2 12	s	452	226	–	2	44.37	20.07	30.09	23.03	50.16	43.10
150	380	3 12 + 1 12		452	339	–	2	44.37	20.07	30.09	27.01	50.16	47.08
150	380	2 16 + 1 12	2 8 + 0 0	515	402	101	2	51.73	19.80	31.26	28.56	51.06	48.36
150	380	2 16 + 1 12	2 10 + 0 0	515	402	157	2	53.31	19.80	31.26	28.56	51.06	48.36
150	380	2 16 + 1 12	2 12 + 0 0	515	402	226	2	54.76	19.80	31.26	28.56	51.06	48.36
150	380	2 16 + 1 12	2 10 + 1 12	515	402	270	2	55.49	19.80	31.26	28.56	51.06	48.36
150	380	2 16 + 1 12	2 10 + 1 16	515	402	358	2	56.20	19.80	31.26	28.56	51.06	48.36
150	380	3 12 + 1 16	2 8 + 0 0	540	339	101	2	53.64	19.80	31.79	26.79	51.59	46.59
150	380	3 12 + 1 16	2 10 + 0 0	540	339	157	2	55.36	19.80	31.79	26.79	51.59	46.59
150	380	3 12 + 1 16	2 12 + 0 0	540	339	226	2	56.99	19.80	31.79	26.79	51.59	46.59
150	380	3 12 + 1 16	2 10 + 1 12	540	339	270	2	57.82	19.80	31.79	26.79	51.59	46.59
150	380	3 12 + 1 16	2 10 + 1 16	540	339	358	2	58.72	19.80	31.79	26.79	51.59	46.59
150	380	2 16 + 2 12	2 8 + 0 0	628	402	101	2	59.83	19.80	33.51	28.56	53.31	48.36
150	380	2 16 + 2 12	2 10 + 0 0	628	402	157	2	62.03	19.80	33.51	28.56	53.31	48.36
150	380	2 16 + 2 12	2 12 + 0 0	628	402	226	2	64.27	19.80	33.51	28.56	53.31	48.36
150	380	2 16 + 2 12	2 10 + 1 12	628	402	270	2	65.51	19.80	33.51	28.56	53.31	48.36
150	380	2 16 + 2 12	2 10 + 1 16	628	402	358	2	67.11	19.80	33.51	28.56	53.31	48.36
150	380	2 20 + 0 0	2 8 + 0 0	628	628	101	1	62.55	20.52	34.30	34.30	54.82	54.82
150	380	2 20 + 0 0	2 10 + 0 0	628	628	157	1	64.75	20.52	34.30	34.30	54.82	54.82
150	380	2 20 + 0 0	2 12 + 0 0	628	628	226	1	66.99	20.52	34.50	34.30	54.82	54.82
150	380	2 20 + 0 0	2 10 + 1 12	628	628	270	1	68.23	20.52	34.30	34.30	54.82	54.82
150	380	2 20 + 0 0	2 10 + 1 16	628	628	358	1	69.83	20.52	34.30	34.30	54.82	54.82

Table F-2A M_{ur} *of Singly Reinforced and Doubly Reinforced Beam - 150 mm wide Continued ...*

M_{ur} Singly and Doubly Rein.Sect
M 20 , Fe 415
b = 150 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{st1} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	$V_{us.min}$ kN	V_{uc} kN	V_{uc1} kN	$V_{ur.min}$ kN	V_{urmin1} kN
150	380	2 20 + 1 12	2 8 + 0 0	741	628	101	2	64.50	19.32	34.85	32.97	54.17	52.29
150	380	2 20 + 1 12	2 10 + 0 0	741	628	157	2	67.32	19.32	34.85	32.97	54.17	52.29
150	380	2 20 + 1 12	2 12 + 0 0	741	628	226	2	70.32	19.32	34.85	32.97	54.17	52.29
150	380	2 20 + 1 12	2 10 + 1 12	741	628	270	2	72.04	19.32	34.85	32.97	54.17	52.29
150	380	2 20 + 1 12	2 10 + 1 16	741	628	358	2	74.66	19.32	34.85	32.97	54.17	52.29
150	380	2 20 + 1 16	2 8 + 0 0	829	628	101	2	68.67	19.32	36.15	32.97	55.47	52.29
150	380	2 20 + 1 16	2 10 + 0 0	829	628	157	2	71.97	19.32	36.15	32.97	55.47	52.29
150	380	2 20 + 1 16	2 12 + 0 0	829	628	226	2	75.56	19.32	36.15	32.97	55.47	52.29
150	380	2 20 + 1 16	2 10 + 1 12	829	628	270	2	77.64	19.32	36.15	32.97	55.47	52.29
150	380	2 20 + 1 16	2 10 + 1 16	829	628	358	2	81.02	19.32	36.15	32.97	55.47	52.29
150	380	2 20 + 2 12	2 8 + 0 0	855	628	101	2	69.72	19.32	36.49	32.97	55.81	52.29
150	380	2 20 + 2 12	2 10 + 0 0	855	628	157	2	73.15	19.32	36.49	32.97	55.81	52.29
150	380	2 20 + 2 12	2 12 + 0 0	855	628	226	2	76.91	19.32	36.49	32.97	55.81	52.29
150	380	2 20 + 2 12	2 10 + 1 12	855	628	270	2	79.10	19.32	36.49	32.97	55.81	52.29
150	380	2 20 + 2 12	2 10 + 1 16	855	628	358	2	82.69	19.32	36.49	32.97	55.81	52.29
150	380	2 25 + 0 0	2 8 + 0 0	982	982	101	1	80.25	20.37	39.53	39.53	59.90	59.90
150	380	2 25 + 0 0	2 10 + 0 0	982	982	157	1	84.38	20.37	39.53	39.53	59.90	59.90
150	380	2 25 + 0 0	2 12 + 0 0	982	982	226	1	88.99	20.37	39.53	39.53	59.90	59.90
150	380	2 25 + 0 0	2 10 + 1 12	982	982	270	1	91.72	20.37	39.53	39.53	59.90	59.90
150	380	2 25 + 0 0	2 10 + 1 16	982	982	358	1	96.39	20.37	39.53	39.53	59.90	59.90
150	400	2 10 + 0 0	Anchor Bars	157	157	–	1	19.68	22.14	20.98	20.98	43.12	43.12
150	400	2 12 + 0 0		226	226	–	1	27.48	22.08	24.38	24.38	46.46	46.46
150	400	2 10 + 1 12		270	157	–	1	32.23	22.08	26.19	20.95	48.27	43.03
150	400	2 12 + 1 10		305	226	–	1	35.82	22.08	27.47	24.38	49.55	46.46
150	400	2 10 + 2 10		314	157	–	2	35.48	21.39	27.27	20.57	48.66	41.96
150	400	2 12 + 1 12		339	226	–	1	39.30	22.08	28.65	24.38	50.73	46.46
150	400	2 10 + 1 16		358	157	–	1	40.89	21.96	29.15	20.88	51.11	42.84
150	400	2 10 + 2 12		383	157	–	2	41.68	21.27	29.33	20.50	50.60	41.77
150	400	2 16 + 0 0		402	402	–	1	45.02	21.96	30.47	30.47	52.43	52.43
150	400	2 12 + 1 16		427	226	–	1	47.30	21.96	31.17	24.30	53.13	46.46
150	400	2 12 + 2 12		452	226	–	2	47.64	21.27	31.21	23.84	52.48	45.11
150	400	3 12 + 1 12		452	339	–	2	47.64	21.27	31.21	28.00	52.48	49.27
150	400	2 16 + 1 12	2 8 + 0 0	515	402	101	2	55.45	21.00	32.46	29.63	53.46	50.63
150	400	2 16 + 1 12	2 10 + 0 0	515	402	157	2	57.03	21.00	32.46	29.63	53.46	50.63
150	400	2 16 + 1 12	2 12 + 0 0	515	402	226	2	58.48	21.00	32.46	29.63	53.46	50.63
150	400	2 16 + 1 12	2 10 + 1 12	515	402	270	2	59.21	21.00	32.46	29.63	53.46	50.63
150	400	2 16 + 1 12	2 10 + 1 16	515	402	358	2	59.92	21.00	32.46	29.63	53.46	50.63
150	400	3 12 + 1 16	2 8 + 0 0	540	339	101	2	57.54	21.00	33.02	27.78	54.02	48.78
150	400	3 12 + 1 16	2 10 + 0 0	540	339	157	2	59.26	21.00	33.02	27.78	54.02	48.78
150	400	3 12 + 1 16	2 12 + 0 0	540	339	226	2	60.89	21.00	33.02	27.78	54.02	48.78
150	400	3 12 + 1 16	2 10 + 1 12	540	339	270	2	61.72	21.00	33.02	27.78	54.02	48.78

Table F-2A M_{ur} of Singly Reinforced and Doubly Reinforced Beam - 150 mm wide Continued ...

M_{ur} Singly and Doubly Rein.Sect
M 20 , Fe 415
b = 150 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{st1} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	$V_{us,min}$ kN	V_{uc} kN	V_{ucl} kN	$V_{ur,min}$ kN	V_{urminl} kN
150	400	3 12 + 1 16	2 10 + 1 16	540	339	358	2	62.62	21.00	33.02	27.78	54.02	48.78
150	400	2 16 + 2 12	2 8 + 0 0	628	402	101	2	64.36	21.00	34.83	29.63	55.83	50.63
150	400	2 16 + 2 12	2 10 + 0 0	628	402	157	2	66.56	21.00	34.83	29.63	55.83	50.63
150	400	2 16 + 2 12	2 12 + 0 0	628	402	226	2	68.81	21.00	34.83	29.63	55.83	50.63
150	400	2 16 + 2 12	2 10 + 1 12	628	402	270	2	70.04	21.00	34.83	29.63	55.83	50.63
150	400	2 16 + 2 12	2 10 + 1 16	628	402	358	2	71.64	21.00	34.83	29.63	55.83	50.63
150	400	2 20 + 0 0	2 8 + 0 0	628	628	101	1	67.08	21.72	35.60	35.60	57.32	57.32
150	400	2 20 + 0 0	2 10 + 0 0	628	628	157	1	69.28	21.72	35.60	35.60	57.32	57.32
150	400	2 20 + 0 0	2 12 + 0 0	628	628	226	1	71.53	21.72	35.60	35.60	57.32	57.32
150	400	2 20 + 0 0	2 10 + 1 12	628	628	270	1	72.76	21.72	35.60	35.60	57.32	57.32
150	400	2 20 + 0 0	2 10 + 1 16	628	628	358	1	74.37	21.72	35.60	30.60	57.32	57.32
150	400	2 20 + 1 12	2 8 + 0 0	741	628	101	2	69.86	20.52	36.29	34.30	56.81	54.82
150	400	2 20 + 1 12	2 10 + 0 0	741	628	157	2	72.67	20.52	36.29	34.30	56.81	54.82
150	400	2 20 + 1 12	2 12 + 0 0	741	628	226	2	75.67	20.52	36.29	34.30	56.81	54.82
150	400	2 20 + 1 12	2 10 + 1 12	741	628	270	2	77.39	20.52	36.29	34.30	56.81	54.82
150	400	2 20 + 1 12	2 10 + 1 16	741	628	358	2	80.01	20.52	36.29	34.30	56.81	54.82
150	400	2 20 + 1 16	2 8 + 0 0	829	628	101	2	74.66	20.52	37.65	34.30	58.17	54.82
150	400	2 20 + 1 16	2 10 + 0 0	829	628	157	2	77.95	20.52	37.65	34.30	58.17	54.82
150	400	2 20 + 1 16	2 12 + 0 0	829	628	226	2	83.08	20.52	37.65	34.30	58.17	54.82
150	400	2 20 + 1 16	2 10 + 0 12	829	628	270	2	83.63	20.52	37.65	34.30	58.17	54.82
150	400	2 20 + 1 16	2 10 + 1 16	829	628	358	2	87.00	20.52	37.65	34.30	58.17	54.82
150	400	2 20 + 2 12	2 8 + 0 0	855	628	101	2	75.88	20.52	38.02	34.30	58.54	54.82
150	400	2 20 + 2 12	2 10 + 0 0	855	628	157	2	79.32	20.52	38.02	34.30	58.54	54.82
150	400	2 20 + 2 12	2 12 + 0 0	855	628	226	2	83.08	20.52	38.02	34.30	58.54	54.82
150	400	2 20 + 2 12	2 10 + 1 12	855	628	270	2	85.27	20.52	38.02	34.30	58.54	54.82
150	400	2 20 + 2 12	2 10 + 1 16	855	628	358	2	88.86	20.52	38.02	34.30	58.54	54.82
150	400	2 20 + 3 12	2 8 + 0 0	968	628	101	2	80.63	20.52	39.55	34.30	60.07	54.82
150	400	2 20 + 3 12	2 10 + 0 0	968	628	157	2	84.69	20.52	39.55	34.30	60.07	54.82
150	400	2 20 + 3 12	2 12 + 0 0	968	628	226	2	89.20	20.52	39.55	34.30	60.07	54.82
150	400	2 20 + 3 12	2 10 + 1 12	968	628	270	2	91.87	20.52	39.55	34.30	60.07	54.82
150	400	2 20 + 3 12	2 10 + 1 16	968	628	358	2	96.42	20.52	39.55	34.30	60.07	54.82
150	400	2 25 + 0 0	2 8 + 0 0	982	982	101	1	87.34	21.57	41.11	41.11	62.68	62.68
150	400	2 25 + 0 0	2 10 + 0 0	982	982	157	1	91.47	21.57	41.11	41.11	62.68	62.68
150	400	2 25 + 0 0	2 12 + 0 0	982	982	226	1	96.08	21.57	41.11	41.11	62.68	62.68
150	400	2 25 + 0 0	2 10 + 1 12	982	982	270	1	98.81	21.57	41.11	41.11	62.68	62.68
150	400	2 25 + 0 0	2 10 + 1 16	982	982	358	1	103.48	21.57	41.11	41.11	62.68	62.68
150	450	2 10 + 0 0	Anchor	157	157	–	1	22.52	25.14	22.57	22.57	47.71	47.71
150	450	2 12 + 0 0		226	226	–	1	31.56	25.08	26.28	26.28	51.36	51.36
150	450	2 10 + 1 12	Bars	270	157	–	1	37.10	25.08	28.26	22.54	53.34	47.62
150	450	2 12 + 1 10		305	226	–	1	41.32	25.08	29.66	26.28	54.74	51.36
150	450	2 10 + 2 10		314	157	–	2	41.15	24.39	29.53	22.18	53.92	46.57

@Seismicisolation

Table F-2A M_{ur} *of Singly Reinforced and Doubly Reinforced Beam - 150 mm wide Continued ...*

M_{ur} Singly and Doubly Rein.Sect M 20, Fe 415 b = 150 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{stl} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	$V_{usv.min}$ kN	V_{uc} kN	V_{ucl} kN	$V_{ur.min}$ kN	V_{urminl} kN
150	450	2 12 + 1 12	Anchor Bars	339	226	–	1	45.42	25.08	30.96	26.28	56.04	51.36
150	450	2 10 + 1 16		358	157	–	1	47.35	24.96	31.53	22.48	56.49	47.44
150	450	2 10 + 2 12		383	157	–	2	48.60	24.27	31.82	22.12	56.09	46.39
150	450	2 16 + 0 0		402	402	–	1	52.28	24.96	32.98	32.98	57.94	57.94
150	450	2 12 + 1 16		427	226	–	1	55.01	24.96	33.76	26.20	58.72	51.16
150	450	2 12 + 2 12		452	226	–	2	55.80	24.27	33.90	25.78	58.17	50.05
150	450	3 12 + 1 12		452	339	–	2	55.80	24.27	33.90	30.35	58.17	54.62
150	450	2 16 + 1 12		515	402	–	2	61.09	24.00	35.34	32.19	59.34	56.19
150	450	3 12 + 1 16		540	339	–	2	63.39	24.00	35.96	30.14	59.96	54.14
150	450	2 16 + 2 12	2 8 + 0 0	628	402	101	2	75.70	24.00	37.97	32.19	61.97	56.19
150	450	2 16 + 2 12	2 10 + 0 0	628	402	157	2	77.90	24.00	37.97	32.19	61.97	56.19
150	450	2 16 + 2 12	2 12 + 0 0	628	402	226	2	80.14	24.00	37.97	32.19	61.97	56.19
150	450	2 16 + 2 12	2 10 + 1 12	628	402	270	2	81.38	24.00	37.97	32.19	61.97	56.19
150	450	2 16 + 2 12	2 10 + 1 16	628	402	358	2	82.96	24.00	37.97	32.19	61.97	56.19
150	450	2 20 + 0 0	2 8 + 0 0	628	628	101	1	78.42	24.72	38.70	38.70	63.42	63.42
150	450	2 20 + 0 0	2 10 + 0 0	628	628	157	1	80.62	24.72	38.70	38.70	63.42	63.42
150	450	2 20 + 0 0	2 12 + 0 0	628	628	226	1	82.86	24.72	38.70	38.70	63.42	63.42
150	450	2 20 + 0 0	2 10 + 1 12	628	628	270	1	84.10	24.72	38.70	38.70	63.42	63.42
150	450	2 20 + 0 0	2 10 + 1 16	628	628	358	1	85.68	24.72	38.70	38.70	63.42	63.42
150	450	2 20 + 1 12	2 8 + 0 0	741	628	101	2	83.23	23.52	39.71	37.48	63.23	61.00
150	450	2 20 + 1 12	2 10 + 0 0	741	628	157	2	86.05	23.52	39.71	37.48	63.23	61.00
150	450	2 20 + 1 12	2 12 + 0 0	741	628	226	2	89.05	23.52	39.71	37.48	63.23	61.00
150	450	2 20 + 1 12	2 10 + 1 12	741	628	270	2	90.70	23.52	39.71	37.48	63.23	61.00
150	450	2 20 + 1 12	2 10 + 1 16	741	628	358	2	93.39	23.52	39.71	37.48	63.23	61.00
150	450	2 20 + 1 16	2 8 + 0 0	829	628	101	2	89.62	23.52	41.25	37.48	64.77	61.00
150	450	2 20 + 1 16	2 10 + 0 0	829	628	157	2	92.92	23.52	41.25	37.48	64.77	61.00
150	450	2 20 + 1 16	2 12 + 0 0	829	628	226	2	96.51	23.52	41.25	37.48	64.77	61.00
150	450	2 20 + 1 16	2 10 + 1 12	829	628	270	2	98.59	23.52	41.25	37.48	64.77	61.00
150	450	2 20 + 1 16	2 10 + 1 16	829	628	358	2	101.96	23.52	41.25	37.48	64.77	61.00
150	450	2 20 + 2 12	2 8 + 0 0	855	628	101	2	91.30	23.52	41.25	37.48	65.19	61.00
150	450	2 20 + 2 12	2 10 + 0 0	855	628	157	2	94.74	23.52	41.25	37.48	65.19	61.00
150	450	2 20 + 2 12	2 12 + 0 0	855	628	226	2	98.49	23.52	41.25	37.48	65.19	61.00
150	450	2 20 + 2 12	2 10 + 1 12	855	628	270	2	100.69	23.52	41.25	37.48	65.19	61.00
150	450	2 20 + 2 12	2 10 + 1 16	855	628	358	2	104.27	23.52	41.25	37.48	65.19	61.00
150	450	2 20 + 3 12	2 8 + 0 0	968	628	101	2	98.09	23.52	43.40	37.48	66.92	61.00
150	450	2 20 + 3 12	2 10 + 0 0	968	628	157	2	102.15	23.52	43.40	37.48	66.92	61.00
150	450	2 20 + 3 12	2 12 + 0 0	968	628	226	2	106.66	23.52	43.40	37.48	66.92	61.00
150	450	2 20 + 3 12	2 10 + 1 12	968	628	270	2	109.33	23.52	43.40	37.48	66.92	61.00
150	450	2 20 + 3 12	2 10 + 1 16	968	628	358	2	113.88	23.52	43.40	37.48	66.92	61.00
150	450	2 25 + 0 0	2 8 + 0 0	982	982	101	1	105.05	24.57	44.91	44.91	69.48	69.48
150	450	2 25 + 0 0	2 10 + 0 0	982	982	157	1	109.18	24.57	44.91	44.91	69.48	69.48
150	450	2 25 + 0 0	2 12 + 0 0	982	982	226	1	113.79	24.57	44.91	44.91	69.48	69.48

Table F-2.4 ***M_{ur} of Singly Reinforced and Doubly Reinforced Beam - 150 mm wide Continued ...***

M_{ur} Singly and Doubly Rein.Sect M 20 , Fe 415 b = 150 mm

b mm	D mm	N1–D1–N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{st1} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	$V_{usv.min}$ kN	V_{uc} kN	V_{ucl} kN	$V_{ur.min}$ kN	V_{urmin1} kN
150	450	2 25 + 0 0	2 10 + 1 12	982	982	270	1	116.52	24.57	44.91	44.91	69.48	69.48
150	450	2 25 + 0 0	2 10 + 1 16	982	982	358	1	121.19	24.57	44.91	44.91	69.48	69.48
150	450	2 20 + 2 16	2 8 + 0 0	1030	628	101	2	101.31	23.52	44.29	37.48	67.81	61.00
150	450	2 20 + 2 16	2 10 + 0 0	1030	628	157	2	105.71	23.52	44.29	37.48	67.81	61.00
150	450	2 20 + 2 16	2 12 + 0 0	1030	628	226	2	110.64	23.52	44.29	37.48	67.81	61.00
150	450	2 20 + 2 16	2 10 + 1 12	1030	628	270	2	113.58	23.52	44.29	37.48	67.81	61.00
150	450	2 20 + 2 16	2 10 + 1 16	1030	628	358	2	118.66	23.52	44.29	37.48	67.81	61.00
150	500	2 10 + 0 0	Anchor Bars	157	157	–	1	25.35	28.14	24.07	24.07	52.21	52.21
150	500	2 12 + 0 0		226	226	–	1	35.64	28.08	28.07	28.07	56.15	56.15
150	500	2 10 + 1 12		270	157	–	1	41.98	28.08	30.21	24.04	58.29	52.12
150	500	2 12 + 1 10		305	226	–	1	46.82	28.08	31.73	28.07	59.81	56.15
150	500	2 10 + 2 10		314	157	–	2	46.82	27.39	31.65	23.71	59.04	51.10
150	500	2 12 + 1 12		339	226	–	1	51.54	28.08	33.14	28.07	61.22	56.15
150	500	2 10 + 1 16		358	157	–	1	53.81	27.96	33.77	23.98	61.73	51.94
150	500	2 10 + 2 12		383	157	–	2	55.51	27.27	34.17	23.65	61.44	50.92
150	500	2 16 + 0 0		402	402	–	1	59.53	27.96	35.35	35.35	63.31	63.31
150	500	2 12 + 1 16		427	226	–	1	62.72	27.96	36.20	28.00	64.16	55.96
150	500	2 12 + 2 12		452	226	–	2	63.96	27.27	36.44	27.60	63.71	54.87
150	500	3 12 + 1 12		452	339	–	2	63.96	27.27	36.44	32.56	63.71	59.83
150	500	2 16 + 1 12		515	402	–	2	70.39	27.00	38.05	34.61	65.05	61.61
150	500	3 12 + 1 16		540	339	–	2	73.14	27.00	38.73	32.37	65.73	59.37
150	500	2 16 + 2 12		628	402	–	2	82.29	27.00	40.95	34.61	67.95	61.61
150	500	2 20 + 0 0		628	628	–	1	85.01	27.72	41.64	41.64	69.36	69.36
150	500	2 20 + 1 12	2 8 + 0 0	741	628	101	2	96.61	26.52	42.95	40.48	69.47	67.00
150	500	2 20 + 1 12	2 10 + 0 0	741	628	157	2	99.43	26.52	42.95	40.48	69.47	67.00
150	500	2 20 + 1 12	2 12 + 0 0	741	628	226	2	102.43	26.52	42.95	40.48	69.47	67.00
150	500	2 20 + 1 12	2 10 + 1 12	741	628	270	2	104.14	26.52	42.95	40.48	69.47	67.00
150	500	2 20 + 1 12	2 10 + 1 16	741	628	358	2	106.77	26.52	42.95	40.48	69.47	67.00
150	500	2 20 + 1 16	2 8 + 0 0	829	628	101	2	104.59	26.52	44.66	40.48	71.18	67.00
150	500	2 20 + 1 16	2 10 + 0 0	829	628	157	2	107.88	26.52	44.66	40.48	71.18	67.00
150	500	2 20 + 1 16	2 12 + 0 0	829	628	226	2	111.47	26.52	44.66	40.48	71.18	67.00
150	500	2 20 + 1 16	2 10 + 1 12	829	628	270	2	113.56	26.52	44.66	40.48	71.18	67.00
150	500	2 20 + 1 16	2 10 + 1 16	829	628	358	2	116.93	26.52	44.66	40.48	71.18	67.00
150	500	2 20 + 2 12	2 8 + 0 0	855	628	101	2	106.72	26.52	45.12	40.48	71.64	67.00
150	500	2 20 + 2 12	2 10 + 0 0	855	628	157	2	110.16	26.52	45.12	40.48	71.64	67.00
150	500	2 20 + 2 12	2 12 + 0 0	855	628	226	2	113.91	26.52	45.12	40.48	71.64	67.00
150	500	2 20 + 2 12	2 10 + 1 12	855	628	270	2	116.10	26.52	45.12	40.48	71.64	67.00
150	500	2 20 + 2 12	2 10 + 1 16	855	628	358	2	119.69	26.52	45.12	40.48	71.64	67.00
150	500	2 20 + 3 12	2 8 + 0 0	968	628	101	2	115.55	26.52	47.05	40.48	73.57	67.00
150	500	2 20 + 3 12	2 10 + 0 0	968	628	157	2	119.61	26.52	47.05	40.48	73.57	67.00
150	500	2 20 + 3 12	2 12 + 0 0	968	628	226	2	124.12	26.52	47.05	40.48	73.57	67.00

Table F-2A M_{ur} of Singly Reinforced and Doubly Reinforced Beam - 150 mm wide Continued ...

M_{ur} Singly and Doubly Rein.Sect M 20 , Fe 415 b = 150 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{st1} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	$V_{usv,min}$ kN	V_{uc} kN	V_{uc1} kN	$V_{ur,min}$ kN	V_{urmin1} kN
150	500	2 20 + 3 12	2 10 + 1 12	968	628	270	2	126.79	26.52	47.05	40.48	73.57	67.00
150	500	2 20 + 3 12	2 10 + 1 16	968	628	358	2	131.34	26.52	47.05	40.48	73.57	67.00
150	500	2 25 + 0 0	2 8 + 0 0	982	982	101	1	122.77	27.57	48.52	48.52	76.09	76.09
150	500	2 25 + 0 0	2 10 + 0 0	982	982	157	1	126.90	27.57	48.52	48.52	76.09	76.09
150	500	2 25 + 0 0	2 12 + 0 0	982	982	226	1	131.50	27.57	48.52	48.52	76.09	76.09
150	500	2 25 + 0 0	2 10 + 1 12	982	982	270	1	134.24	27.57	48.52	48.52	76.09	76.09
150	500	2 25 + 0 0	2 10 + 1 16	982	982	358	1	138.90	27.57	48.52	48.52	76.09	76.09
150	500	2 20 + 0 16	2 8 + 0 0	1030	628	101	2	119.91	26.52	48.04	40.48	74.56	67.00
150	500	2 20 + 0 16	2 10 + 0 0	1030	628	157	2	124.30	26.52	48.04	40.48	74.56	67.00
150	500	2 20 + 0 16	2 12 + 0 0	1030	628	226	2	129.24	26.52	48.04	40.48	74.56	67.00
150	500	2 20 + 0 16	2 10 + 1 12	1030	628	270	2	132.18	26.52	48.04	40.48	74.56	67.00
150	500	2 20 + 0 16	2 10 + 1 16	1030	628	358	2	137.26	26.52	48.04	40.48	74.56	67.00
150	530	2 10 + 0 0	Anchor Bars	157	157	–	1	27.05	29.94	24.93	24.93	54.87	54.87
150	530	2 12 + 0 0		226	226	–	1	38.09	29.88	29.10	29.10	58.98	58.98
150	530	2 10 + 1 12		270	157	–	1	44.90	29.88	31.34	24.91	61.22	54.79
150	530	2 12 + 1 10		305	226	–	1	50.12	29.88	32.93	29.10	62.81	58.98
150	530	2 10 + 2 10		314	157	–	2	50.22	29.19	32.88	24.58	62.07	53.77
150	530	2 12 + 1 12		339	226	–	1	55.22	29.88	34.39	29.10	64.27	58.98
150	530	2 10 + 1 16		358	157	–	1	57.69	29.76	35.06	24.85	64.82	54.61
150	530	2 10 + 2 12		383	157	–	2	59.66	29.07	35.52	24.52	64.59	53.59
150	530	2 16 + 0 0		402	402	–	1	63.89	29.76	36.71	36.71	66.47	66.47
150	530	2 12 + 1 16		427	226	–	1	67.34	29.76	37.60	29.03	67.36	58.79
150	530	2 12 + 2 12		452	226	–	2	68.86	29.07	37.90	28.64	66.97	57.71
150	530	3 12 + 1 12		452	339	–	2	68.86	29.07	37.90	33.83	66.97	62.90
150	530	2 16 + 1 12		515	402	–	2	75.97	28.80	39.61	35.99	68.41	64.79
150	530	3 12 + 1 16		540	339	–	2	78.99	28.80	40.33	33.64	69.13	62.44
150	530	2 16 + 2 12		628	402	–	2	89.09	28.80	42.66	35.99	71.46	64.79
150	530	2 20 + 0 0		628	628	–	1	91.81	29.52	43.33	43.33	72.85	72.85
150	530	2 20 + 1 12	2 8 + 0 0	741	628	101	2	104.64	28.32	44.81	42.21	73.13	70.53
150	530	2 20 + 1 12	2 10 + 0 0	741	628	157	2	107.45	28.32	44.81	42.21	73.13	70.53
150	530	2 20 + 1 12	2 12 + 0 0	741	628	226	2	110.45	28.32	44.81	42.21	73.13	70.53
150	530	2 20 + 1 12	2 10 + 1 12	741	628	270	2	112.17	28.32	44.81	42.21	73.13	70.53
150	530	2 20 + 1 12	2 10 + 1 16	741	628	358	2	114.79	28.32	44.81	42.21	73.13	70.53
150	530	2 20 + 1 16	2 8 + 0 0	829	628	101	2	113.56	28.32	46.62	42.21	74.94	70.53
150	530	2 20 + 1 16	2 10 + 0 0	829	628	157	2	116.86	28.32	46.62	42.21	74.94	70.53
150	530	2 20 + 1 16	2 12 + 0 0	829	628	226	2	120.45	28.32	46.62	42.21	74.94	70.53
150	530	2 20 + 1 16	2 10 + 1 12	829	628	270	2	122.54	28.32	46.62	42.21	74.94	70.53
150	530	2 20 + 1 16	2 10 + 1 16	829	628	358	2	125.91	28.32	46.62	42.21	74.94	70.53
150	530	2 20 + 2 12	2 8 + 0 0	855	628	101	2	115.97	28.32	47.10	42.21	75.42	70.53
150	530	2 20 + 2 12	2 10 + 0 0	855	628	157	2	119.41	28.32	47.10	42.21	75.42	70.53
150	530	2 20 + 2 12	2 12 + 0 0	855	628	226	2	123.16	28.32	47.10	42.21	75.42	70.53

Table F-2.4 ***M_{ur} of Singly Reinforced and Doubly Reinforced Beam - 150 mm wide Continued ...***

M_{ur} Singly and Doubly Rein.Sect
M 20 , Fe 415
b = 150 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{st1} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	V_{usumin} kN	V_{uc} kN	V_{uc1} kN	V_{urmin} kN	V_{urmin1} kN
150	530	2 20 + 2 12	2 10 + 1 12	855	628	270	2	125.36	28.32	47.10	42.21	75.42	70.53
150	530	2 20 + 2 12	2 10 + 1 16	855	628	358	2	128.94	28.32	47.10	42.21	75.42	70.53
150	530	2 20 + 3 12	2 8 + 0 0	968	628	101	2	126.03	28.32	49.15	42.21	77.47	70.53
150	530	2 20 + 3 12	2 10 + 0 0	968	628	157	2	130.08	28.32	49.15	42.21	77.47	70.53
150	530	2 20 + 3 12	2 12 + 0 0	968	628	226	2	134.59	28.32	49.15	42.21	77.47	70.53
150	530	2 20 + 3 12	2 10 + 1 12	968	628	270	2	137.27	28.32	49.15	42.21	77.47	70.53
150	530	2 20 + 3 12	2 10 + 1 16	968	628	358	2	141.81	28.32	49.15	42.21	77.47	70.53
150	530	2 25 + 0 0	2 8 + 0 0	982	982	101	1	133.40	29.37	50.59	50.59	79.96	79.96
150	530	2 25 + 0 0	2 10 + 0 0	982	982	157	1	137.53	29.37	50.59	50.59	79.96	79.96
150	530	2 25 + 0 0	2 12 + 0 0	982	982	226	1	142.13	29.37	50.59	50.59	79.96	79.96
150	530	2 25 + 0 0	2 10 + 1 12	982	982	270	1	144.87	29.37	50.59	50.59	79.96	79.96
150	530	2 25 + 0 0	2 10 + 1 16	982	982	358	1	149.53	29.37	50.59	50.59	79.96	79.96
150	530	2 20 + 2 16	2 8 + 0 0	1030	628	101	2	131.06	28.32	50.20	42.21	78.52	70.53
150	530	2 20 + 2 16	2 10 + 0 0	1030	628	157	2	135.46	28.32	50.20	42.21	78.52	70.53
150	530	2 20 + 2 16	2 12 + 0 0	1030	628	226	2	140.39	28.32	50.20	42.21	78.52	70.53
150	530	2 20 + 2 16	2 10 + 1 12	1030	628	270	2	143.33	28.32	50.20	42.21	78.52	70.53
150	530	2 20 + 2 16	2 10 + 1 16	1030	628	358	2	148.41	28.32	50.20	42.21	78.52	70.53
150	550	2 12 + 0 0	Anchor Bars	226	226	–	1	39.72	31.08	29.77	29.77	60.85	60.85
150	550	2 10 + 1 12		270	157	–	1	46.85	31.08	32.07	25.47	63.15	56.55
150	550	2 12 + 1 10		305	226	–	1	52.32	31.08	33.70	29.77	64.78	60.85
150	550	2 10 + 2 10		314	157	–	2	52.49	30.39	33.67	25.15	64.06	55.54
150	550	2 12 + 1 12		339	226	–	1	57.67	31.08	35.21	29.77	66.29	60.85
150	550	2 10 + 1 16		358	157	–	1	60.27	30.96	35.90	25.41	66.86	56.37
150	550	2 10 + 2 12		383	157	–	2	62.43	30.27	36.39	25.09	66.66	55.36
150	550	2 16 + 0 0		402	402	–	1	66.79	30.96	37.60	37.60	68.56	68.56
150	550	2 12 + 1 16		427	226	–	1	70.43	30.96	38.51	29.71	69.47	60.47
150	550	2 12 + 2 12		452	226	–	2	72.12	30.27	38.85	29.32	69.12	59.59
150	550	3 12 + 1 12		452	339	–	2	72.12	30.27	38.85	34.66	69.12	64.93
150	550	2 16 + 1 12		515	402	–	2	79.69	30.00	40.62	36.89	70.62	66.89
150	550	3 12 + 1 16		540	339	–	2	82.89	30.00	41.37	34.47	71.37	64.47
150	550	2 16 + 2 12		628	402	–	2	93.62	30.00	43.77	36.89	73.77	66.89
150	550	2 20 + 0 0		628	628	–	1	96.34	30.72	44.43	44.43	75.15	75.15
150	550	2 20 + 1 12	2 8 + 0 0	741	628	101	2	109.99	29.52	46.02	43.33	75.54	72.85
150	550	2 20 + 1 12	2 10 + 0 0	741	628	157	2	112.80	29.52	46.02	43.33	75.54	72.85
150	550	2 20 + 1 12	2 12 + 0 0	741	628	226	2	115.80	29.52	46.02	43.33	75.54	72.85
150	550	2 20 + 1 12	2 10 + 1 12	741	628	270	2	117.52	29.52	46.02	43.33	75.54	72.85
150	550	2 20 + 1 12	2 10 + 1 16	741	628	358	2	120.14	29.52	46.02	43.33	75.54	72.85
150	550	2 20 + 1 16	2 8 + 0 0	829	628	101	2	119.55	29.52	47.89	43.33	77.41	72.85
150	550	2 20 + 1 16	2 10 + 0 0	829	628	157	2	122.85	29.52	47.89	43.33	77.41	72.85
150	550	2 20 + 1 16	2 12 + 0 0	829	628	226	2	126.43	29.52	47.89	43.33	77.41	72.85
150	550	2 20 + 1 16	2 10 + 1 12	829	628	270	2	128.52	29.52	47.89	43.33	77.41	72.85

Table F-2A M_{ur} *of Singly Reinforced and Doubly Reinforced Beam - 150 mm wide Continued ...*

M_{ur} Singly and Doubly Rein.Sect
M 20 , Fe 415
b = 150 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{st1} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	$V_{usv.min}$ kN	V_{uc} kN	V_{ucl} kN	$V_{ur.min}$ kN	V_{urminl} kN
150	550	2 20 + 1 16	2 10 + 1 16	829	628	358	2	131.89	29.52	47.89	43.33	77.41	72.85
150	550	2 20 + 2 12	2 8 + 0 0	855	628	101	2	122.14	29.52	48.39	43.33	77.91	72.85
150	550	2 20 + 2 12	2 10 + 0 0	855	628	157	2	125.58	29.52	48.39	43.33	77.91	72.85
150	550	2 20 + 2 12	2 12 + 0 0	855	628	226	2	129.33	29.52	48.39	43.33	77.91	72.85
150	550	2 20 + 2 12	2 10 + 1 12	855	628	270	2	131.52	29.52	48.39	43.33	77.91	72.85
150	550	2 20 + 2 12	2 10 + 1 16	855	628	358	2	135.11	29.52	48.39	43.33	77.91	72.85
150	550	2 20 + 3 12	2 8 + 0 0	968	628	101	2	133.01	29.52	50.51	43.33	80.03	72.85
150	550	2 20 + 3 12	2 10 + 0 0	968	628	157	2	137.07	29.52	50.51	43.33	80.03	72.85
150	550	2 20 + 3 12	2 12 + 0 0	968	628	226	2	141.58	29.52	50.51	43.33	80.03	72.85
150	550	2 20 + 3 12	2 10 + 1 12	968	628	270	2	144.25	29.52	50.51	43.33	80.03	72.85
150	550	2 20 + 3 12	2 10 + 1 16	968	628	358	2	148.79	29.52	50.51	43.33	80.03	72.85
150	550	2 25 + 0 0	2 8 + 0 0	982	982	101	1	140.48	30.57	51.95	51.95	82.52	82.52
150	550	2 25 + 0 0	2 10 + 0 0	982	982	157	1	144.61	30.57	51.95	51.95	82.52	82.52
150	550	2 25 + 0 0	2 12 + 0 0	982	982	226	1	149.22	30.57	51.95	51.95	82.52	82.52
150	550	2 25 + 0 0	2 10 + 1 12	982	982	270	1	151.95	30.57	51.95	51.95	82.52	82.52
150	550	2 25 + 0 0	2 10 + 1 16	982	982	358	1	156.61	30.57	51.95	51.95	82.52	82.52
150	550	2 20 + 2 16	2 8 + 0 0	1030	628	101	2	138.50	29.52	51.60	43.33	81.12	72.85
150	550	2 20 + 2 16	2 10 + 0 0	1030	628	157	2	142.90	29.52	51.60	43.33	81.12	72.85
150	550	2 20 + 2 16	2 12 + 0 0	1030	628	226	2	147.83	29.52	51.60	43.33	81.12	72.85
150	550	2 20 + 2 16	2 10 + 1 12	1030	628	270	2	150.77	29.52	51.60	43.33	81.12	72.85
150	550	2 20 + 2 16	2 10 + 1 16	1030	628	358	2	155.85	29.52	51.60	43.33	81.12	72.85
150	580	2 12 + 0 0	Anchor Bars	226	226	–	1	42.17	32.88	30.76	30.76	63.64	63.64
150	580	2 10 + 1 12		270	157	–	1	49.78	32.88	33.14	26.29	66.02	59.17
150	580	2 12 + 1 10		305	226	–	1	55.62	32.88	34.84	30.76	67.72	63.64
150	580	2 10 + 2 10		314	157	–	2	55.89	32.19	34.84	25.98	67.03	58.17
150	580	2 12 + 1 12		339	226	–	1	61.34	32.88	36.41	30.76	69.29	63.64
150	580	2 10 + 1 16		358	157	–	1	64.15	32.76	37.13	26.24	69.89	59.00
150	580	2 10 + 1 12		383	157	–	2	66.58	32.07	37.68	25.92	69.75	57.99
150	580	2 16 + 0 0		402	402	–	1	71.14	32.76	38.90	38.90	71.66	71.66
150	580	2 12 + 1 16		427	226	–	1	75.05	32.76	39.85	30.69	72.61	63.45
150	580	2 12 + 2 12		452	226	–	2	77.02	32.07	40.24	30.32	72.31	62.39
150	580	3 12 + 1 12		452	339	–	2	77.02	32.07	40.24	35.87	72.31	67.94
150	580	2 16 + 1 12		515	402	–	2	85.26	31.80	42.11	38.21	73.91	70.01
150	580	3 12 + 1 16		540	339	–	2	88.74	31.80	42.89	35.69	74.69	67.49
150	580	2 16 + 2 12		628	402	–	2	100.42	31.80	45.40	38.21	77.20	70.01
150	580	2 20 + 0 0		628	628	–	2	103.15	32.52	46.04	46.04	78.56	78.56
150	580	2 20 + 1 12		741	628	–	2	112.17	31.32	47.79	44.97	79.11	76.29
150	580	2 20 + 1 16	2 8 + 0 0	829	628	101	2	128.53	31.32	49.76	44.97	81.08	76.29
150	580	2 20 + 1 16	2 10 + 0 0	829	628	157	2	131.83	31.32	49.76	44.97	81.08	76.29
150	580	2 20 + 1 16	2 12 + 0 0	829	628	226	2	135.41	31.32	49.76	44.97	81.08	76.29
150	580	2 20 + 1 16	2 10 + 1 12	829	628	270	2	137.50	31.32	49.76	44.97	81.08	76.29

Table F-2A M_{ur} of Singly Reinforced and Doubly Reinforced Beam - 150 mm wide Continued ...

M_{ur} Singly and Doubly Rein.Sect
M 20 , Fe 415
b = 150 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{st1} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	$V_{usv.min}$ kN	V_{uc} kN	V_{ucl} kN	$V_{ur.min}$ kN	V_{urminl} kN
150	580	2 20 + 2 12	2 10 + 1 12	855	628	270	2	140.77	31.32	50.29	44.97	81.61	76.29
150	580	2 20 + 2 12	2 10 + 1 16	855	628	358	2	144.36	31.32	50.29	44.97	81.61	76.29
150	580	2 20 + 3 12	2 8 + 0 0	968	628	101	2	143.49	31.32	52.52	44.97	83.84	76.29
150	580	2 20 + 3 12	2 10 + 0 0	968	628	157	2	147.54	31.32	52.52	44.97	83.84	76.29
150	580	2 20 + 3 12	2 10 + 0 0	968	628	226	2	152.05	31.32	52.52	44.97	83.84	76.29
150	580	2 20 + 3 12	2 10 + 1 12	968	628	270	2	154.73	31.32	52.52	44.97	83.84	76.29
150	580	2 20 + 3 12	2 10 + 1 16	968	628	358	2	159.27	31.32	52.52	44.97	83.84	76.29
150	580	2 25 + 0 0	2 8 + 0 0	982	982	101	1	151.11	32.37	53.93	53.93	86.30	86.30
150	580	2 25 + 0 0	2 10 + 0 0	982	982	157	1	154.24	32.37	53.93	53.93	86.30	86.30
150	580	2 25 + 0 0	2 12 + 0 0	982	982	226	1	159.84	32.37	53.93	53.93	86.30	86.30
150	580	2 25 + 0 0	2 10 + 1 12	982	982	270	1	162.58	32.37	53.93	53.93	86.30	86.30
150	580	2 25 + 0 0	2 10 + 1 16	982	982	358	1	167.24	32.37	53.93	53.93	86.30	86.30
150	580	2 20 + 2 16	2 8 + 0 0	1030	628	101	2	149.65	31.32	53.66	44.97	84.98	76.29
150	580	2 20 + 2 16	2 10 + 0 0	1030	628	157	2	154.05	31.32	53.66	44.97	84.98	76.29
150	580	2 20 + 2 16	2 12 + 0 0	1030	628	226	2	158.98	31.32	53.66	44.97	84.98	76.29
150	580	2 20 + 2 16	2 10 + 1 12	1030	628	270	2	161.93	31.32	53.66	44.97	84.98	76.29
150	580	2 20 + 2 16	2 10 + 1 16	1030	628	358	2	167.00	31.32	53.66	44.97	84.98	76.29
150	600	2 12 + 0 0	A	226	226	–	1	43.80	34.08	31.40	31.40	65.48	65.48
150	600	2 10 + 1 12	n	270	157	–	1	51.73	34.08	33.84	26.83	67.92	60.91
150	600	2 12 + 1 10		305	226	–	1	57.82	34.08	35.58	31.40	69.66	65.48
150	600	2 10 + 2 10	c	314	157	–	2	58.15	33.39	35.60	26.52	68.99	59.91
150	600	2 12 + 1 12		339	226	–	1	63.79	34.08	37.19	31.40	71.27	65.48
150	600	2 10 + 1 16	h	358	157	–	1	66.74	33.96	37.94	26.77	71.90	60.73
150	600	2 10 + 2 12	o	383	157	–	2	69.35	33.27	38.52	26.47	71.79	59.74
150	600	2 16 + 0 0		402	402	–	1	74.05	33.96	39.75	39.75	73.71	73.71
150	600	2 12 + 1 16	r	427	226	–	1	78.14	33.96	40.73	31.33	74.69	65.29
150	600	2 12 + 2 12		452	226	–	2	80.29	33.27	41.15	30.97	74.42	64.24
150	600	3 12 + 1 12		452	339	–	2	80.29	33.27	41.15	36.66	74.42	69.93
150	600	2 16 + 1 12	B	515	402	–	2	88.98	33.00	43.08	39.07	76.08	72.07
150	600	3 12 + 1 16	a	540	339	–	2	92.64	33.00	43.88	36.49	76.88	69.49
150	600	2 16 + 2 12	r	628	402	–	2	104.96	33.00	46.47	39.07	79.47	72.07
150	600	2 20 + 0 0	s	628	628	–	1	107.68	33.72	47.10	47.10	80.82	80.82
150	600	2 20 + 1 12		741	628	–	2	117.52	32.52	48.95	46.04	81.47	78.56
150	600	2 20 + 1 16	2 8 + 0 0	829	628	101	2	134.51	32.52	50.98	46.04	83.50	78.56
150	600	2 20 + 1 16	2 10 + 0 0	829	628	157	2	137.81	32.52	50.98	46.04	83.50	78.56
150	600	2 20 + 1 16	2 12 + 0 0	829	628	226	2	141.40	32.52	50.98	46.04	83.50	78.56
150	600	2 20 + 1 16	2 10 + 1 12	829	628	270	2	143.49	32.52	50.98	46.04	83.50	78.56
150	600	2 20 + 1 16	2 10 + 1 16	829	628	358	2	146.86	32.52	50.98	46.04	83.50	78.56
150	600	2 20 + 2 12	2 8 + 0 0	855	628	101	2	137.56	32.52	51.52	46.04	84.04	78.56
150	600	2 20 + 2 12	2 10 + 0 0	855	628	157	2	140.99	32.52	51.52	46.04	84.04	78.56
150	600	2 20 + 2 12	2 12 + 0 0	855	628	226	2	144.75	32.52	51.52	46.04	84.04	78.56

Table F-2A *M_{ur} of Singly Reinforced and Doubly Reinforced Beam - 150 mm wide Continued ...*

M_{ur} Singly and Doubly Rein.Sect
M 20 , Fe 415
b = 150 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{st1} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	$V_{usv.min}$ kN	V_{uc} kN	V_{uc1} kN	$V_{ur.min}$ kN	V_{urmin1} kN
150	580	2 20 + 1 16	2 10 + 1 16	829	628	358	2	140.87	31.32	49.76	44.97	81.08	76.29
150	580	2 20 + 2 12	2 8 + 0 0	855	628	101	2	131.39	31.32	50.29	44.97	81.61	76.29
150	580	2 20 + 2 12	2 10 + 0 0	855	628	157	2	134.83	31.32	50.29	44.97	81.61	76.29
150	580	2 20 + 2 12	2 12 + 0 0	855	628	226	2	138.58	31.32	50.29	44.97	81.61	76.29
150	600	2 20 + 2 12	2 10 + 1 12	855	628	270	2	146.94	32.52	51.52	46.04	84.04	78.56
150	600	2 20 + 2 12	2 10 + 1 16	855	628	358	2	150.53	32.52	51.52	46.04	84.04	78.56
150	600	2 20 + 3 12	2 8 + 0 0	968	628	101	2	150.47	32.52	53.83	46.04	86.35	78.56
150	600	2 20 + 3 12	2 10 + 0 0	968	628	157	2	154.53	32.52	53.83	46.04	86.35	78.56
150	600	2 20 + 3 12	2 12 + 0 0	968	628	226	2	159.03	32.52	53.83	46.04	86.35	78.56
150	600	2 20 + 3 12	2 10 + 1 12	968	628	270	2	161.71	32.52	53.83	46.04	86.35	78.56
150	600	2 20 + 3 12	2 10 + 1 16	968	628	358	2	166.25	32.52	53.83	46.04	86.35	78.56
150	600	2 25 + 0 0	2 8 + 0 0	982	982	101	1	158.20	33.57	55.23	55.23	88.80	88.80
150	600	2 25 + 0 0	2 10 + 0 0	982	982	157	1	162.33	33.57	55.23	55.23	88.80	88.80
150	600	2 25 + 0 0	2 12 + 0 0	982	982	226	1	166.93	33.57	55.23	55.23	88.80	88.80
150	600	2 25 + 0 0	2 10 + 1 12	982	982	270	1	169.66	33.57	55.23	55.23	88.80	88.80
150	600	2 25 + 0 0	2 10 + 1 16	982	982	358	1	174.33	33.57	55.23	55.23	88.80	88.80
150	600	2 20 + 2 16	2 8 + 0 0	1030	628	101	2	157.09	32.52	55.01	46.04	87.53	78.56
150	600	2 20 + 2 16	2 10 + 0 0	1030	628	157	2	161.49	32.52	55.01	46.04	87.53	78.56
150	600	2 20 + 2 16	2 12 + 0 0	1030	628	226	2	166.42	32.52	55.01	46.04	87.53	78.56
150	600	2 20 + 2 16	2 10 + 1 12	1030	628	270	2	169.36	32.52	55.01	46.04	87.53	78.56
150	600	2 20 + 2 16	2 10 + 1 16	1030	628	358	2	174.44	32.52	55.01	46.04	87.53	78.56
150	650	2 12 + 0 0	Anchor Bars	226	226	–	1	47.89	37.08	32.95	32.95	70.03	70.03
150	650	2 10 + 1 12		270	157	–	1	56.60	37.08	35.54	28.13	72.62	65.21
150	650	2 12 + 1 10		305	226	–	1	63.32	37.08	37.08	32.95	74.46	70.03
150	650	2 10 + 2 10		314	157	–	2	63.82	36.39	37.44	27.83	73.83	64.22
150	650	2 12 + 1 12		339	226	–	1	69.91	37.08	39.08	32.95	76.16	70.03
150	650	2 10 + 1 16		358	157	–	1	73.20	36.96	39.89	28.08	76.85	65.04
150	650	2 10 + 2 12		383	157	–	2	76.26	36.27	40.55	27.78	76.82	64.05
150	650	2 16 + 0 0		402	402	–	1	81.30	36.96	41.81	41.81	78.77	78.77
150	650	2 12 + 1 16		427	226	–	1	85.85	36.96	42.85	32.89	79.81	69.85
150	650	2 12 + 2 12		452	226	–	2	88.45	36.27	43.35	32.54	79.62	68.81
150	650	3 12 + 1 12		452	339	–	2	88.45	36.27	43.35	38.38	79.62	74.85
150	650	2 16 + 1 12		515	402	–	2	98.28	36.00	45.43	41.16	81.43	77.16
150	650	3 12 + 1 16		540	339	–	2	102.39	36.00	46.28	38.41	82.28	74.41
150	650	2 16 + 2 12		628	402	–	2	116.30	36.00	49.05	41.16	85.05	77.16
150	650	2 20 + 0 0		628	628	–	1	119.02	36.72	49.65	49.65	86.37	86.37
150	650	2 20 + 1 12		741	628	–	2	130.89	35.52	51.76	48.64	87.28	84.16
150	650	2 20 + 1 16		829	628	–	2	142.78	35.52	53.93	48.64	89.45	84.16
150	650	2 20 + 2 12	2 8 + 0 0	855	628	101	2	152.98	35.52	54.52	48.64	90.04	84.16
150	650	2 20 + 2 12	2 10 + 0 0	855	628	157	2	156.41	35.52	54.52	48.64	90.04	84.16
150	650	2 20 + 2 12	2 12 + 0 0	855	628	226	2	160.17	35.52	54.52	48.64	90.04	84.16

Table F-2A M_{ur} of Singly Reinforced and Doubly Reinforced Beam - 150 mm wide Continued ...

M_{ur} Singly and Doubly Rein.Sect
M 20 , Fe 415
b = 150 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{st1} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	$V_{usv.min}$ kN	V_{uc} kN	V_{uc1} kN	$V_{ur.min}$ kN	$V_{ur.min1}$ kN
150	650	2 20 + 2 12	2 10 + 1 12	855	628	270	2	162.36	35.52	54.52	48.64	90.04	84.16
150	650	2 20 + 2 12	2 10 + 1 16	855	628	358	2	165.94	35.52	54.52	48.64	90.04	84.16
150	650	2 20 + 3 12	2 8 + 0 0	968	628	101	2	167.93	35.52	57.00	48.64	92.52	84.16
150	650	2 20 + 3 12	2 10 + 0 0	968	628	157	2	171.98	35.52	57.00	48.64	92.52	84.16
150	650	2 20 + 3 12	2 12 + 0 0	968	628	226	2	176.49	35.52	57.00	48.64	92.52	84.16
150	650	2 20 + 3 12	2 10 + 1 12	968	628	270	2	179.17	35.52	57.00	48.64	92.52	84.16
150	650	2 20 + 3 12	2 10 + 1 16	968	628	358	2	183.71	35.52	57.00	48.64	92.52	84.16
150	650	2 25 + 0 0	2 8 + 0 0	982	982	101	1	175.91	36.57	58.38	58.38	94.95	94.95
150	650	2 25 + 0 0	2 10 + 0 0	982	982	157	1	180.04	36.57	58.38	58.38	94.95	94.95
150	650	2 25 + 0 0	2 12 + 0 0	982	982	226	1	184.64	36.57	58.38	58.38	94.95	94.95
150	650	2 25 + 0 0	2 10 + 1 12	982	982	270	1	187.38	36.57	58.38	58.38	94.95	94.95
150	650	2 25 + 0 0	2 10 + 1 16	982	982	358	1	192.04	36.57	58.38	58.38	94.95	94.95
150	650	2 20 + 2 16	2 8 + 0 0	1030	628	101	1	175.68	35.52	58.27	48.64	93.79	84.16
150	650	2 20 + 2 16	2 10 + 0 0	1030	628	157	1	180.08	35.52	58.27	48.64	93.79	84.16
150	650	2 20 + 2 16	2 12 + 0 0	1030	628	226	1	185.01	35.52	58.27	48.64	93.79	84.16
150	650	2 20 + 2 16	2 10 + 1 12	1030	628	270	1	187.95	35.52	58.27	48.64	93.79	84.16
150	650	2 20 + 2 16	2 10 + 1 16	1030	628	358	1	193.03	35.52	58.27	48.64	93.79	84.16

Note :- N1 - D1 Represent Number - Daimeter bars going into the Support.

N2 - D2 Represent Number - Diameter of Bent up or curtailed bars.

NC - Diac Represent Number - Diameter of Compression Steel.

A_{st} Total area of Tension Steel in mm^2 Corresponding to N1-D1 + N2-D2

A_{st1} Total area of Tension Steel going into the Support in mm^2 Corresponding to N1-D1

A_{sc} Total area of Compression Steel in mm^2

$V_{usv.min}$ Shear resisted by Minimum Stirrups in kN

V_{uc} Shear resisted by Concrete Corresponding to area of steel A_{st} in kN

V_{uc1} Shear resisted by Concrete Corresponding to area of steel A_{st1} in kN

$V_{ur.min}$ Shear resisted by beam with minimum Stirrups Corresponding to A_{st} in kN

$V_{ur.min1}$ Shear resisted by beam with minimum Stirrups Corresponding to A_{st1} in kN

Table F-2B **Ultimate Moment of Resistance (M_{ur}) in kN m of Singly reinforced and Doubly reinforced Rectangular Beam - 200 mm wide. Mild Environment - Concrete M20, Steel Fe415, (for Nominal Cover See *Table C-I*)**

M_{ur} Singly and Doubly Rein.Sect M 20 , Fe 415
b = 200 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm²	A_{st1} mm²	A_{sc} mm²	RO	M_{ur} kN.m	$V_{usv.min}$ kN	V_{uc} kN	V_{uc1} kN	$V_{ur.min}$ kN	V_{urmin1} kN
200	300	2 10 + 0 0	A	157	157	–	1	14.32	21.52	20.64	20.64	42.16	42.16
200	300	2 12 + 0 0	n	226	226	–	1	19.96	21.44	23.95	23.95	45.39	45.39
200	300	2 10 + 1 12	c	270	157	–	1	23.39	21.44	25.73	20.60	47.17	42.04
200	300	2 12 + 1 10	h	305	226	–	1	25.99	21.44	26.98	23.95	48.42	45.39
200	300	2 10 + 2 10	o	314	157	–	1	26.79	21.52	27.37	20.64	48.89	42.16
200	300	2 12 + 1 12	r	339	226	–	1	28.50	21.44	28.13	23.95	49.57	45.39
200	300	2 10 + 1 16		358	157	–	1	29.57	21.28	28.59	20.51	49.87	41.79
200	300	2 10 + 2 12	B	383	157	–	1	31.56	21.44	29.48	20.60	50.92	42.04
200	300	2 16 + 0 0	a	402	402	–	1	32.53	21.28	29.88	29.88	51.16	51.16
200	300	2 12 + 1 16	r	427	226	–	1	34.16	21.28	30.56	23.85	51.84	45.13
200	300	2 12 + 2 12	s	452	226	–	1	36.07	21.44	31.37	23.95	52.81	45.39
200	300	3 12 + 1 12		452	339	–	1	36.07	21.44	31.37	28.13	52.81	49.57
200	300	2 16 + 1 12	2 8 + 0 0	515	402	101	1	42.01	21.28	32.74	29.88	54.02	51.16
200	300	2 16 + 1 12	2 10 + 0 0	515	402	157	1	43.03	21.28	32.74	29.88	54.02	51.16
200	300	2 16 + 1 12	2 12 + 0 0	515	402	226	1	43.93	21.28	32.74	29.88	54.02	51.16
200	300	2 16 + 1 12	2 10 + 1 12	515	402	270	1	44.40	21.28	32.74	29.88	54.02	51.16
200	300	2 16 + 1 12	2 10 + 1 16	515	402	358	1	44.45	21.28	32.74	29.88	54.02	51.16
200	300	3 12 + 1 16	2 8 + 0 0	540	339	101	1	43.61	21.28	33.31	28.00	54.59	49.28
200	300	3 12 + 1 16	2 10 + 0 0	540	339	157	1	44.74	21.28	33.31	28.00	54.59	49.28
200	300	3 12 + 1 16	2 12 + 0 0	540	339	226	1	45.77	21.28	33.31	28.00	54.59	49.28
200	300	3 12 + 1 16	2 10 + 1 12	540	339	270	1	46.31	21.28	33.31	28.00	54.59	49.28
200	300	3 12 + 1 16	2 10 + 1 16	540	339	358	1	46.61	21.28	33.31	28.00	54.59	49.28
200	300	2 16 + 2 12	2 8 + 0 0	628	402	101	1	48.84	21.28	35.13	29.88	56.41	51.16
200	300	2 16 + 2 12	2 10 + 0 0	628	402	157	1	50.36	21.28	35.13	29.88	56.41	51.16
200	300	2 16 + 2 12	2 12 + 0 0	628	402	226	1	51.83	21.28	35.13	29.88	56.41	51.16
200	300	2 16 + 2 12	2 10 + 1 12	628	402	270	1	52.63	21.28	35.13	29.88	56.41	51.16
200	300	2 16 + 2 12	2 10 + 1 16	628	402	358	1	53.57	21.28	35.13	29.88	56.41	51.16
200	300	2 20 + 0 0	2 8 + 0 0	628	628	101	1	47.94	20.96	34.78	34.78	55.74	55.74
200	300	2 20 + 0 0	2 10 + 0 0	628	628	157	1	49.45	20.96	34.78	34.78	55.74	55.74
200	300	2 20 + 0 0	2 12 + 0 0	628	628	226	1	50.92	20.96	34.78	34.78	55.74	55.74
200	300	2 20 + 0 0	2 10 + 1 12	628	628	270	1	51.72	20.96	34.78	34.78	55.74	55.74
200	300	2 20 + 0 0	2 10 + 1 16	628	628	358	1	52.66	20.96	34.78	34.78	55.74	55.74
200	300	2 20 + 1 12	2 8 + 0 0	741	628	101	1	53.64	20.96	36.80	34.78	57.76	55.74
200	300	2 20 + 1 12	2 10 + 0 0	741	628	157	1	55.62	20.96	36.80	34.78	57.76	55.74
200	300	2 20 + 1 12	2 12 + 0 0	741	628	226	1	57.69	20.96	36.80	34.78	57.76	55.74
200	300	2 20 + 1 12	2 10 + 1 12	741	628	270	1	58.86	20.96	36.80	34.78	57.76	55.74
200	300	2 20 + 1 12	2 10 + 1 16	741	628	358	1	60.49	20.96	36.80	34.78	57.76	55.74
200	300	2 20 + 1 16	2 8 + 0 0	829	628	101	1	57.42	20.96	38.19	34.78	59.15	55.54
200	300	2 20 + 1 16	2 10 + 0 0	829	628	157	1	59.75	20.96	38.19	34.78	59.15	55.54
200	300	2 20 + 1 16	2 12 + 0 0	829	628	226	1	62.27	20.96	38.19	34.78	59.15	55.54
200	300	2 20 + 1 16	2 10 + 1 12	829	628	270	1	63.73	20.96	38.19	34.78	59.15	55.54

Table F-2B M_{ur} **of Singly Reinforced and Doubly Reinforced Beam - 200 mm wide Continued ...**

M_{ur} Singly and Doubly Rein.Sect
M 20 , Fe 415
b = 200 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{st1} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	$V_{usv.min}$ kN	V_{uc} kN	V_{ucl} kN	$V_{ur.min}$ kN	V_{urmin1} kN
200	300	2 20 + 1 16	2 10 + 1 16	829	628	358	1	65.95	20.96	38.19	34.78	59.15	55.54
200	300	2 20 + 2 12	2 8 + 0 0	855	628	101	2	52.22	19.36	36.54	33.02	55.90	52.38
200	300	2 20 + 2 12	2 10 + 0 0	855	628	157	2	54.66	19.36	36.54	33.02	55.90	52.38
200	300	2 20 + 2 12	2 12 + 0 0	855	628	226	2	57.30	19.36	36.54	33.02	55.90	52.38
200	300	2 20 + 2 12	2 10 + 1 12	855	628	270	2	58.84	19.36	36.54	33.02	55.90	52.38
200	300	2 20 + 2 12	2 10 + 1 16	855	628	358	2	61.26	19.36	36.54	33.02	55.90	52.38
200	300	2 20 + 3 12	2 8 + 0 0	968	628	101	2	55.19	19.36	38.00	33.02	57.36	52.38
200	300	2 20 + 3 12	2 10 + 0 0	968	628	157	2	58.09	19.36	38.00	33.02	57.36	52.38
200	300	2 20 + 3 12	2 12 + 0 0	968	628	226	2	61.30	19.36	38.00	33.02	57.36	52.38
200	300	2 20 + 3 12	2 10 + 1 12	968	628	270	2	63.20	19.36	38.00	33.02	57.36	52.38
200	300	2 20 + 3 12	2 10 + 1 16	968	628	358	2	66.34	19.36	38.00	33.02	57.36	52.38
200	300	2 25 + 0 0	2 8 + 0 0	982	982	101	1	61.69	20.76	40.05	40.05	60.81	60.81
200	300	2 25 + 0 0	2 10 + 0 0	982	982	157	1	64.65	20.76	40.05	40.05	60.81	60.81
200	300	2 25 + 0 0	2 12 + 0 0	982	982	226	1	67.93	20.76	40.05	40.05	60.81	60.81
200	300	2 25 + 0 0	2 10 + 1 12	982	982	270	1	69.88	20.76	40.05	40.05	60.81	60.81
200	300	2 25 + 0 0	2 10 + 1 16	982	982	358	1	73.10	20.76	40.05	40.05	60.81	60.81
200	380	2 10 + 0 0	Anchor Bars	157	157		1	18.86	27.92	23.97	23.97	51.89	51.89
200	380	2 12 + 0 0		226	226	–	1	26.49	27.84	27.93	27.93	55.77	55.77
200	380	2 10 + 1 12		270	157	–	1	31.19	27.84	30.06	23.93	57.90	51.77
200	380	2 12 + 1 10		305	226	–	1	34.79	27.84	31.57	27.93	59.41	55.77
200	380	2 10 + 2 10		314	157	–	1	35.86	27.92	32.02	23.97	59.94	51.89
200	380	2 12 + 1 12		339	226	–	1	38.29	27.84	32.97	27.93	60.81	55.77
200	382	2 10 + 1 16		358	157	–	1	39.91	27.68	33.57	23.85	61.25	51.53
200	380	2 10 + 1 16		383	157	–	1	42.62	27.84	34.60	23.93	62.44	51.77
200	380	2 16 + 0 0		402	402	–	1	44.14	27.68	35.13	35.13	62.81	62.81
200	380	2 12 + 1 16		427	226	–	1	46.50	27.68	35.97	27.84	63.65	55.52
200	380	2 12 + 2 12		452	226	–	1	49.13	27.84	36.91	27.93	64.75	55.77
200	380	3 12 + 1 12		452	339	–	1	49.13	27.84	36.91	32.97	64.75	60.81
200	380	2 16 + 1 12		515	402	–	1	54.37	27.68	38.64	35.13	66.32	62.81
200	380	3 12 + 1 16		540	339	–	1	56.51	27.68	39.34	32.85	67.02	60.53
200	380	2 16 + 2 12		628	402	–	1	63.64	27.68	41.60	35.13	69.28	62.81
200	380	2 20 + 0 0		628	628	–	1	62.73	27.36	41.29	41.29	68.65	68.65
200	380	2 20 + 1 12	2 8 + 0 0	741	628	101	1	75.05	27.36	43.82	41.29	71.18	68.65
200	380	2 20 + 1 12	2 10 + 0 0	741	628	157	1	77.02	27.36	43.82	41.29	71.18	68.65
200	380	2 20 + 1 12	2 12 + 0 0	741	628	226	1	79.09	27.36	43.82	41.29	71.18	68.65
200	380	2 20 + 1 12	2 10 + 1 12	741	628	270	1	80.26	27.36	43.82	41.29	71.18	68.65
200	380	2 20 + 1 12	2 10 + 1 16	741	628	358	1	81.89	27.36	43.82	41.29	71.18	68.65
200	380	2 20 + 1 16	2 8 + 0 0	829	628	101	1	81.36	27.36	45.58	41.29	72.94	68.65
200	380	2 20 + 1 16	2 10 + 0 0	829	628	157	1	83.70	27.36	45.58	41.29	72.94	68.65
200	380	2 20 + 1 16	2 12 + 0 0	829	628	226	1	86.21	27.36	45.58	41.29	72.94	68.65
200	380	2 20 + 1 16	2 10 + 1 12	829	628	270	1	87.67	27.36	45.58	41.29	72.94	68.65
200	380	2 20 + 1 16	2 10 + 1 16	829	628	358	1	89.89	27.36	45.58	41.29	72.94	68.65

Table F-2B ***M_{ur} of Singly Reinforced and Doubly Reinforced Beam - 200 mm wide Continued ...***

M_{ur} Singly and Doubly Rein.Sect M 20 , Fe 415 b = 200 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm²	A_{st1} mm²	A_{sc} mm²	RO	M_{ur} kN.m	$V_{usv.min}$ kN	V_{uc} kN	V_{uc1} kN	$V_{ur.min}$ kN	$V_{ur.min1}$ kN
200	380	2 20 + 2 12	2 8 + 0 0	855	628	101	2	76.89	25.76	44.26	39.74	70.02	65.50
200	380	2 20 + 2 12	2 10 + 0 0	855	628	157	2	79.33	25.76	44.26	39.74	70.02	65.50
200	380	2 20 + 2 12	2 12 + 0 0	855	628	226	2	81.97	25.76	44.26	39.74	70.02	65.50
200	380	2 20 + 2 12	2 10 + 1 12	855	628	270	2	83.51	25.76	44.26	39.74	70.02	65.50
200	380	2 20 + 2 12	2 10 + 1 16	855	628	358	2	85.91	25.76	44.26	39.74	70.02	65.50
200	380	2 20 + 3 12	2 8 + 0 0	968	628	101	2	83.13	25.76	46.14	39.74	71.90	65.50
200	380	2 20 + 3 12	2 10 + 0 0	968	628	157	2	86.03	25.76	46.14	39.74	71.90	65.50
200	380	2 20 + 3 12	2 12 + 0 0	968	628	226	2	89.23	25.76	46.14	39.74	71.90	65.50
200	380	2 20 + 3 12	2 10 + 1 12	968	628	270	2	91.13	25.76	46.14	39.74	71.90	65.50
200	380	2 20 + 3 12	2 10 + 1 16	968	628	358	2	94.27	25.76	46.14	39.74	71.90	65.50
200	380	2 25 + 0 0	2 8 + 0 0	982	982	101	1	90.04	27.16	48.03	48.03	75.19	75.19
200	380	2 25 + 0 0	2 10 + 0 0	982	982	157	1	93.00	27.16	48.03	48.03	75.19	75.19
200	380	2 25 + 0 0	2 12 + 0 0	982	982	226	1	96.27	27.16	48.03	48.03	75.19	75.19
200	380	2 25 + 0 0	2 10 + 1 12	982	982	270	1	98.22	27.16	48.03	48.03	75.19	75.19
200	380	2 25 + 0 0	2 10 + 1 16	982	982	358	1	101.44	27.16	48.03	48.03	75.19	75.19
200	380	2 20 + 2 16	2 8 + 0 0	1030	628	101	2	86.17	25.76	47.11	39.74	72.87	65.50
200	380	2 20 + 2 16	2 10 + 0 0	1030	628	157	2	89.33	25.76	47.11	39.74	72.87	65.50
200	380	2 20 + 2 16	2 12 + 0 0	1030	628	226	2	92.85	25.76	47.11	39.74	72.87	65.50
200	380	2 20 + 2 16	2 10 + 1 12	1030	628	270	2	94.96	25.76	47.11	39.74	72.87	65.50
200	380	2 20 + 2 16	2 10 + 1 16	1030	628	358	2	98.49	25.76	47.11	39.74	72.87	65.50
200	400	2 10 + 0 0	Anchor Bars	157	157	–	1	19.99	29.52	24.74	24.74	54.26	54.26
200	400	2 12 + 0 0		226	226	–	1	28.12	29.44	28.85	28.85	58.29	58.29
200	400	2 10 + 1 12		270	157	–	1	33.14	29.44	31.07	24.70	60.51	54.14
200	400	2 12 + 1 10		305	226	–	1	36.98	29.44	32.64	28.85	62.08	58.29
200	400	2 10 + 2 10		314	157	–	1	38.13	29.52	33.10	24.74	62.62	54.26
200	400	2 12 + 1 12		339	226	–	1	40.74	29.44	34.09	28.85	63.53	58.29
200	400	2 10 + 1 16		358	157	–	1	42.49	29.28	34.72	24.62	64.00	53.90
200	400	2 10 + 2 12		383	157	–	1	45.39	29.44	35.79	24.70	65.23	54.14
200	400	2 16 + 0 0		402	402	–	1	47.05	29.28	36.35	36.35	65.63	65.63
200	400	2 12 + 1 16		427	226	–	1	49.58	29.28	37.23	28.76	66.51	58.04
200	400	2 12 + 2 12		452	226	–	1	52.40	29.44	38.19	28.85	67.63	58.29
200	400	3 12 + 1 12		452	339	–	1	52.40	29.44	38.19	34.09	67.63	63.53
200	400	2 16 + 1 12		515	402	–	1	58.09	29.28	40.02	36.35	69.30	65.63
200	400	3 12 + 1 16		540	339	–	1	60.41	29.28	40.75	33.98	70.03	63.26
200	400	2 16 + 2 12		628	402	–	1	68.18	29.28	43.11	36.35	72.39	65.63
200	400	2 20 + 0 0		628	628	–	1	67.27	28.96	42.81	42.81	71.77	71.77
200	400	2 20 + 1 12	2 8 + 0 0	741	628	101	1	80.40	28.96	45.46	42.81	74.42	71.77
200	400	2 20 + 1 12	2 10 + 0 0	741	628	157	1	82.37	28.96	45.46	42.81	74.42	71.77
200	400	2 20 + 1 12	2 12 + 0 0	741	628	226	1	84.44	28.96	45.46	42.81	74.42	71.77
200	400	2 20 + 1 12	2 10 + 1 12	741	628	270	1	85.62	28.96	45.46	42.81	74.42	71.77
200	400	2 20 + 1 12	2 10 + 1 16	741	628	358	1	87.24	28.96	45.46	42.81	74.42	71.77

Table F-2B M_{ur} **of Singly Reinforced and Doubly Reinforced Beam - 200 mm wide Continued ...**

M_{ur} Singly and Doubly Rein.Sect
M 20 , Fe 415
b = 200 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{st1} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	$V_{us,min}$ kN	V_{uc} kN	V_{uc1} kN	$V_{ur,min}$ kN	$V_{ur,min1}$ kN
200	400	2 20 + 1 16	2 8 + 0 0	829	628	101	1	87.35	28.96	47.30	42.81	76.26	71.77
200	400	2 20 + 1 16	2 10 + 0 0	829	628	157	1	89.68	28.96	47.30	42.81	76.26	71.77
200	400	2 20 + 1 16	2 12 + 0 0	829	628	226	1	92.19	28.96	47.30	42.81	76.26	71.77
200	400	2 20 + 1 16	2 10 + 1 12	829	628	270	1	93.65	28.96	47.30	42.81	76.26	71.77
200	400	2 20 + 1 16	2 10 + 1 16	829	628	358	1	95.88	28.96	47.30	42.81	76.26	71.77
200	400	2 20 + 2 12	2 8 + 0 0	855	628	101	2	83.06	27.36	46.05	41.29	73.41	68.65
200	400	2 20 + 2 12	2 10 + 0 0	855	628	157	2	85.50	27.36	46.05	41.29	73.41	68.65
200	400	2 20 + 2 12	2 12 + 0 0	855	628	226	2	88.13	27.36	46.05	41.29	73.41	68.65
200	400	2 20 + 2 12	2 10 + 1 12	855	628	270	2	89.67	27.36	46.05	41.29	73.41	68.65
200	400	2 20 + 2 12	2 10 + 1 16	855	628	358	2	92.07	27.36	46.05	41.29	73.41	68.65
200	400	2 20 + 3 12	2 8 + 0 0	968	628	101	2	90.11	27.36	48.04	41.29	75.40	68.65
200	400	2 20 + 3 12	2 10 + 0 0	968	628	157	2	93.01	27.36	48.04	41.29	75.40	68.65
200	400	2 20 + 3 12	2 12 + 0 0	968	628	226	2	96.22	27.36	48.04	41.29	75.40	68.65
200	400	2 20 + 3 12	2 10 + 1 12	968	628	270	2	98.12	27.36	48.04	41.29	75.40	68.65
200	400	2 20 + 3 12	2 10 + 1 16	968	628	358	2	101.25	27.36	48.04	41.29	75.40	68.65
200	400	2 25 + 0 0	2 8 + 0 0	982	982	101	1	97.12	28.76	49.90	49.90	78.66	78.66
200	400	2 25 + 0 0	2 10 + 0 0	982	982	157	1	100.08	28.76	49.90	49.90	78.66	78.66
200	400	2 25 + 0 0	2 12 + 0 0	982	982	226	1	103.36	28.76	49.90	49.90	78.66	78.66
200	400	2 25 + 0 0	2 10 + 1 12	982	982	270	1	105.31	28.76	49.90	49.90	78.66	78.66
200	400	2 25 + 0 0	2 10 + 1 16	982	982	358	1	108.53	28.76	49.90	49.90	78.66	78.66
200	400	2 20 + 2 16	2 8 + 0 0	1030	628	101	2	93.61	27.36	49.05	41.29	76.41	68.65
200	400	2 20 + 2 16	2 10 + 0 0	1030	628	157	2	96.77	27.36	49.05	41.29	76.41	68.65
200	400	2 20 + 2 16	2 12 + 0 0	1030	628	226	2	100.29	27.36	49.05	41.29	76.41	68.65
200	400	2 20 + 2 16	2 10 + 1 12	1030	628	270	2	102.39	27.36	49.05	41.29	76.41	68.65
200	400	2 20 + 2 16	2 10 + 1 16	1030	628	358	2	105.93	27.36	49.05	41.29	76.41	68.65
200	450	2 12 + 0 0	Anchor Bars	226	226	–	1	32.20	33.44	31.06	31.06	64.50	64.50
200	450	2 10 + 1 12		270	157	–	1	38.02	33.44	33.47	26.54	66.91	59.98
200	450	2 12 + 1 10		305	226	–	1	42.48	33.44	35.19	31.06	68.63	64.50
200	450	2 10 + 2 10		314	157	–	1	43.80	33.52	35.68	26.58	69.20	60.10
200	450	2 12 + 1 12		339	226	–	1	46.86	33.44	36.77	31.06	70.21	64.50
200	450	2 10 + 1 16		358	157	–	1	48.95	33.28	37.48	26.47	70.76	59.75
200	450	2 10 + 2 12		383	157	–	1	52.30	33.44	38.63	26.54	72.07	59.98
200	450	2 16 + 0 0		402	402	–	1	54.30	33.28	39.27	39.27	72.55	72.55
200	450	2 12 + 1 16		427	226	–	1	57.29	33.28	40.23	30.97	73.51	64.25
200	450	2 12 + 2 12		452	226	–	1	60.56	33.44	41.27	31.06	74.71	64.50
200	450	3 12 + 1 12		452	339	–	1	60.56	33.44	41.27	36.77	74.71	70.21
200	450	2 16 + 1 12		515	402	–	1	67.39	33.28	43.30	39.27	76.58	72.55
200	450	3 12 + 1 16		540	339	–	1	70.16	33.28	44.10	36.67	77.38	69.95
200	450	2 12 + 2 12		628	402	–	1	79.51	33.28	46.71	39.27	79.99	72.55
200	450	2 20 + 0 0		628	628	–	1	78.61	32.96	46.43	46.43	79.39	79.39
200	450	2 20 + 1 12		741	628	–	1	89.61	32.96	49.37	46.43	82.33	79.39
200	450	2 20 + 1 16	2 8 + 0 0	829	628	101	1	102.31	32.96	51.42	46.43	84.38	79.39

Table F-2B ***M_{ur} of Singly Reinforced and Doubly Reinforced Beam - 200 mm wide Continued ...***

M_{ur} Singly and Doubly Rein.Sect M 20 , Fe 415 b = 200 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{st1} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	$V_{usv.min}$ kN	V_{uc} kN	V_{uc1} kN	$V_{ur.min}$ kN	$V_{ur.min1}$ kN
200	450	2 20 + 1 16	2 10 + 0 0	829	628	157	1	104.64	32.96	51.42	46.43	84.38	79.39
200	450	2 20 + 1 16	2 12 + 0 0	829	628	226	1	107.16	32.96	51.42	46.43	84.38	79.39
200	450	2 20 + 1 16	2 10 + 1 12	829	628	270	1	108.62	32.96	51.42	46.43	84.38	79.39
200	450	2 20 + 1 16	2 10 + 1 16	829	628	358	1	110.84	32.96	51.42	46.43	84.38	79.39
200	450	2 20 + 2 12	2 8 + 0 0	855	628	101	2	98.48	31.36	50.33	45.01	81.69	76.37
200	450	2 20 + 2 12	2 10 + 0 0	855	628	157	2	101.91	31.36	50.33	45.01	81.69	76.37
200	450	2 20 + 2 12	2 12 + 0 0	855	628	226	2	103.55	31.36	50.33	45.01	81.69	76.37
200	450	2 20 + 2 12	2 10 + 1 12	855	628	270	2	105.09	31.36	50.33	45.01	81.69	76.37
200	450	2 20 + 2 12	2 10 + 1 16	855	628	358	2	107.49	31.36	50.33	45.01	81.69	76.37
200	450	2 20 + 3 12	2 8 + 0 0	968	628	101	2	107.57	31.36	52.56	45.01	83.92	76.37
200	450	2 20 + 3 12	2 10 + 0 0	968	628	157	2	110.47	31.36	52.56	45.01	83.92	76.37
200	450	2 20 + 3 12	2 12 + 0 0	968	628	226	2	113.67	31.36	52.56	45.01	83.92	76.37
200	450	2 20 + 3 12	2 10 + 1 12	968	628	270	2	115.57	31.36	52.56	45.01	83.92	76.37
200	450	2 20 + 3 12	2 10 + 1 16	968	628	358	2	118.71	31.36	52.56	45.01	83.92	76.37
200	450	2 25 + 0 0	2 8 + 0 0	982	982	101	1	114.84	32.76	54.36	54.36	87.12	87.12
200	450	2 25 + 0 0	2 10 + 0 0	982	982	157	1	117.80	32.76	54.36	54.36	87.12	87.12
200	450	2 25 + 0 0	2 12 + 0 0	982	982	226	1	121.07	32.76	54.36	54.36	87.12	87.12
200	450	2 25 + 0 0	2 10 + 1 12	982	982	270	1	123.02	32.76	54.36	54.36	87.12	87.12
200	450	2 25 + 0 0	2 10 + 1 16	982	982	358	1	126.24	32.76	54.36	54.36	87.12	87.12
200	450	2 20 + 2 16	2 8 + 0 0	1030	628	101	2	112.20	31.36	53.71	45.01	85.07	76.37
200	450	2 20 + 2 16	2 10 + 0 0	1030	628	157	2	115.36	31.36	53.71	45.01	85.07	76.37
200	450	2 20 + 2 16	2 12 + 0 0	1030	628	226	2	118.88	31.36	53.71	45.01	85.07	76.37
200	450	2 20 + 2 16	2 10 + 1 12	1030	628	270	2	120.99	31.36	53.71	45.01	85.07	76.37
200	450	2 20 + 2 16	2 10 + 1 16	1030	628	358	2	124.52	31.36	53.71	45.01	85.07	76.37
200	500	2 12 + 0 0	Anchor Bars	226	226	–	1	36.28	37.44	33.14	33.14	70.58	70.58
200	500	2 10 + 1 12		270	157	–	1	42.89	37.44	35.74	28.28	73.18	65.72
200	500	2 12 + 1 10		305	226	–	1	47.98	37.44	37.59	33.14	75.03	70.58
200	500	2 10 + 2 10		314	157	–	1	49.47	37.52	38.12	28.31	75.64	65.83
200	500	2 12 + 1 12		339	226	–	1	52.98	27.44	39.30	33.14	76.74	70.58
200	500	2 10 + 1 16		358	157	–	1	55.41	37.28	40.09	28.21	77.37	65.49
200	500	2 10 + 2 12		383	157	–	1	59.22	37.44	41.32	28.28	78.76	65.72
200	500	2 16 + 0 0		402	402	–	1	61.56	37.28	42.03	42.03	79.31	79.31
200	500	2 12 + 1 16		427	226	–	1	65.00	37.28	43.07	33.06	80.35	70.34
200	500	2 12 + 2 12		452	226	–	1	68.72	37.44	44.18	33.14	81.62	70.58
200	500	3 12 + 1 12		452	339	–	1	68.72	37.44	44.18	39.30	81.62	76.74
200	500	2 16 + 1 12		515	402	–	1	76.68	37.28	46.40	42.03	83.68	79.31
200	500	3 12 + 1 16		540	339	–	1	79.91	37.28	47.28	39.21	84.56	76.49
200	500	2 16 + 2 12		628	402	–	1	90.85	37.28	50.12	42.03	87.40	79.31
200	500	2 20 + 0 0		628	628	–	1	89.94	36.96	49.85	49.85	86.81	86.81
200	500	2 20 + 1 12		741	628	–	1	102.99	36.96	53.07	49.85	90.03	86.81
200	500	2 20 + 1 16		829	628	–	1	112.47	36.96	55.31	49.85	92.27	86.81
200	500	2 20 + 2 12	2 8 + 0 0	855	628	101	2	113.89	35.36	54.36	48.51	89.72	83.87

Table F-2B M_{ur} of Singly Reinforced and Doubly Reinforced Beam - 200 mm wide Continued ...

M_{ur} Singly and Doubly Rein.Sect
M 20 , Fe 415
b = 200 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{st1} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	$V_{usv.min}$ kN	V_{uc} kN	V_{uc1} kN	$V_{ur.min}$ kN	$V_{ur.min1}$ kN
200	500	2 20 + 2 12	2 10 + 0 0	855	628	157	2	116.33	35.36	54.36	48.51	89.72	83.87
200	500	2 20 + 2 12	2 12 + 0 0	855	628	226	2	118.97	35.36	54.36	48.51	89.72	83.87
200	500	2 20 + 2 12	2 10 + 1 12	855	628	270	2	120.51	35.36	54.36	48.51	89.72	83.87
200	500	2 20 + 2 12	2 10 + 1 16	855	628	358	2	122.91	35.36	54.36	48.51	89.72	83.87
200	500	2 20 + 3 12	2 8 + 0 0	968	628	101	2	125.03	35.36	56.83	48.51	92.19	83.87
200	500	2 20 + 3 12	2 10 + 0 0	968	628	157	2	127.93	35.36	56.83	48.51	92.19	83.87
200	500	2 20 + 3 12	2 12 + 0 0	968	628	226	2	131.13	35.36	56.83	48.51	92.19	83.87
200	500	2 20 + 3 12	2 10 + 1 12	968	628	270	2	133.03	35.36	56.83	48.51	92.19	83.87
200	500	2 20 + 3 12	2 10 + 1 16	968	628	358	2	136.17	35.36	56.83	48.51	92.19	83.87
200	500	2 25 + 0 0	2 8 + 0 0	982	982	101	1	132.55	36.76	58.58	58.58	95.34	95.34
200	500	2 25 + 0 0	2 10 + 0 0	982	982	157	1	135.51	36.76	58.58	58.58	95.34	95.34
200	500	2 25 + 0 0	2 12 + 0 0	982	982	226	1	138.78	36.76	58.58	58.58	95.34	95.34
200	500	2 25 + 0 0	2 10 + 1 12	982	982	270	1	140.73	36.76	58.58	58.58	95.34	95.34
200	500	2 25 + 0 0	2 10 + 1 16	982	982	358	1	143.96	36.76	58.58	58.58	95.34	95.34
200	500	2 20 + 2 16	2 8 + 0 0	1030	628	101	2	130.80	35.36	58.10	48.51	93.46	83.87
200	500	2 20 + 0 16	2 10 + 0 0	1030	628	157	2	133.96	35.36	58.10	48.51	93.46	83.87
200	500	2 20 + 0 16	2 12 + 0 0	1030	628	226	2	137.47	35.36	58.10	48.51	93.46	83.87
200	500	2 20 + 0 16	2 10 + 1 12	1030	628	270	2	139.58	35.36	58.10	48.51	93.46	83.87
200	500	2 20 + 0 16	2 10 + 1 16	1030	628	358	2	143.11	35.36	58.10	48.51	93.46	83.87
200	530	2 12 + 0 0	A	226	226	–	1	38.73	39.84	34.33	34.33	74.17	74.17
200	530	2 10 + 1 12	n	270	157	–	1	45.82	39.84	37.04	29.28	76.88	69.12
200	530	2 12 + 1 10		305	226	–	1	51.28	39.84	38.97	34.33	78.81	74.17
200	530	2 10 + 2 10	c	314	157	–	1	52.87	39.92	39.52	29.31	79.44	69.23
200	530	2 12 + 1 12	h	339	226	–	1	56.66	39.84	40.76	34.33	80.60	74.17
200	530	2 10 + 1 16		358	157	–	1	59.29	39.68	41.59	29.21	81.27	68.89
200	530	2 10 + 2 12	o	383	157	–	1	63.37	39.84	42.87	29.28	82.71	69.12
200	530	2 16 + 0 0	r	402	402	–	1	65.91	39.68	43.61	43.61	83.29	83.29
200	530	2 12 + 1 16		427	226	–	1	69.63	39.68	44.70	34.25	84.38	73.93
200	530	2 12 + 2 12		452	226	–	1	73.62	39.84	45.86	34.33	85.70	74.17
200	530	3 12 + 1 12		452	339	–	1	73.62	39.84	45.86	40.76	85.70	80.60
200	530	2 16 + 1 12		515	402	–	1	82.26	39.68	48.19	43.61	87.87	83.29
200	530	3 12 + 1 16	B	540	339	–	1	85.76	39.68	49.10	40.67	88.78	80.35
200	530	2 16 + 2 12	a	628	402	–	1	97.65	39.68	52.08	43.61	91.76	83.29
200	530	2 20 + 0 0		628	628	–	1	96.75	39.36	51.82	51.82	91.18	91.18
200	530	2 20 + 1 12	r	741	628	–	1	111.01	39.36	55.20	51.82	94.56	91.18
200	530	2 20 + 1 16	s	829	628	–	1	121.45	39.36	57.56	51.82	96.92	91.18
200	530	2 20 + 2 12		855	628	–	2	118.16	37.76	56.69	50.52	94.45	88.28
200	530	2 20 + 1 16		829	628	–	1	121.45	39.36	57.56	51.82	96.92	91.18
200	530	2 20 + 2 12		855	628	–	2	118.16	37.76	56.69	50.52	94.45	88.28
200	530	2 20 + 3 12	2 8 + 0 0	968	628	101	2	135.40	37.76	59.29	50.52	97.05	88.28
200	530	2 20 + 3 12	2 10 + 0 0	968	628	157	2	138.40	37.76	59.29	50.52	97.05	88.28
200	530	2 20 + 3 12	2 12 + 0 0	968	628	226	2	141.61	37.76	59.29	50.52	97.05	88.28

Table F-2B M_{ur} *of Singly Reinforced and Doubly Reinforced Beam - 200 mm wide Continued ...*

M_{ur} Singly and Doubly Rein.Sect M 20 , Fe 415 b = 200 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{st1} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	$V_{us,min}$ kN	V_{uc} kN	V_{ucl} kN	$V_{ur,min}$ kN	$V_{ur,min1}$ kN
200	530	2 20 + 3 12	2 10 + 1 12	968	628	270	2	143.51	37.76	59.29	50.52	97.05	88.28
200	530	2 20 + 3 12	2 10 + 1 16	968	628	358	2	146.64	37.76	59.29	50.52	97.05	88.28
200	530	2 25 + 0 0	2 8 + 0 0	982	982	101	1	143.18	39.16	61.01	61.01	100.17	100.17
200	530	2 25 + 0 0	2 10 + 0 0	982	982	157	1	146.14	39.16	61.01	61.01	100.17	100.17
200	530	2 25 + 0 0	2 12 + 0 0	982	982	226	1	149.41	39.16	61.01	61.01	100.17	100.17
200	530	2 25 + 0 0	2 10 + 1 12	982	982	270	1	151.36	39.16	61.01	61.01	100.17	100.17
200	530	2 25 + 0 0	2 10 + 1 16	982	982	358	1	154.58	39.16	61.01	61.01	100.17	100.17
200	530	2 20 + 2 16	2 8 + 0 0	1030	628	101	2	141.95	37.76	60.63	50.52	98.39	88.28
200	530	2 20 + 2 16	2 10 + 0 0	1030	628	157	2	145.11	37.76	60.63	50.52	98.39	88.28
200	530	2 20 + 2 16	2 12 + 0 0	1030	628	226	2	148.63	37.76	60.63	50.52	98.39	88.28
200	530	2 20 + 2 16	2 10 + 1 12	1030	628	270	2	150.73	37.76	60.63	50.52	98.39	88.28
200	530	2 20 + 2 16	2 10 + 1 16	1030	628	358	2	154.27	37.76	60.63	50.52	98.39	88.28
200	550	2 12 + 0 0	Anchor Bars	226	226	–	1	40.36	41.44	35.11	35.11	76.55	76.55
200	550	2 10 + 1 12		270	157	–	1	47.77	41.44	37.89	29.93	79.33	71.37
200	550	2 12 + 1 10		305	226	–	1	53.48	41.44	39.87	35.11	81.31	76.55
200	550	2 10 + 2 10		314	157	–	1	55.14	41.52	40.43	29.96	81.95	71.48
200	550	2 12 + 1 12		339	226	–	1	59.11	41.44	41.71	35.11	83.15	76.55
200	550	2 10 + 1 16		358	157	–	1	61.88	41.28	42.56	29.86	83.84	71.14
200	550	2 10 + 2 12		383	157	–	1	66.13	41.44	43.87	29.93	85.31	71.37
200	550	2 16 + 0 0		402	402	–	1	68.81	41.28	44.64	44.64	85.92	85.92
200	550	2 12 + 1 16		427	226	–	1	72.71	41.28	45.76	35.03	87.04	76.31
200	550	2 12 + 2 12		452	226	–	1	76.89	41.44	46.95	35.11	88.39	76.55
200	550	3 12 + 1 12		452	339	–	1	76.89	41.44	46.95	41.71	88.39	83.15
200	550	2 16 + 1 12		515	402	–	1	85.98	41.28	49.35	44.64	90.63	85.92
200	550	3 12 + 1 16		540	339	–	1	89.66	41.28	50.29	41.62	91.57	82.90
200	550	2 16 + 0 12		628	402	–	1	102.19	41.28	53.36	44.64	94.64	85.92
200	550	2 20 + 0 0		628	628	–	1	101.28	40.96	53.10	53.10	94.06	94.06
200	550	2 20 + 1 12		741	628	–	1	116.36	40.96	56.58	53.10	97.54	94.06
200	550	2 20 + 1 16		829	628	–	1	127.43	40.96	59.02	53.10	99.98	94.06
200	550	2 20 + 2 12		855	628	–	2	124.32	39.36	58.19	51.82	97.55	91.18
200	550	2 20 + 3 12	2 8 + 0 0	968	628	101	2	142.49	39.36	60.89	51.82	100.25	91.18
200	550	2 20 + 3 12	2 10 + 0 0	968	628	157	2	145.39	39.36	60.89	51.82	100.25	91.18
200	550	2 20 + 3 12	2 12 + 0 0	968	628	226	2	148.59	39.36	60.89	51.82	100.25	91.18
200	550	2 20 + 3 12	2 10 + 1 12	968	628	270	2	150.49	39.36	60.89	51.82	100.25	91.18
200	550	2 20 + 3 12	2 10 + 1 16	968	628	358	2	153.63	39.36	60.89	51.82	100.25	91.18
200	550	2 25 + 0 0	2 8 + 0 0	982	982	101	1	150.26	40.76	62.59	62.59	103.35	103.35
200	550	2 25 + 0 0	2 10 + 0 0	982	982	157	1	153.22	40.76	62.59	62.59	103.35	103.35
200	550	2 25 + 0 0	2 12 + 0 0	982	982	226	1	156.50	40.76	62.59	62.59	103.35	103.35
200	550	2 25 + 0 0	2 10 + 1 12	982	982	270	1	158.44	40.76	62.59	62.59	103.35	103.35
200	550	2 25 + 0 0	2 10 + 1 16	982	982	358	1	161.67	40.76	62.59	62.59	103.35	103.35
200	550	2 20 + 2 16	2 8 + 0 0	1030	628	101	1	149.39	39.36	62.28	51.82	101.64	91.18
200	550	2 20 + 2 16	2 10 + 0 0	1030	628	157	2	152.55	39.36	62.28	51.82	101.64	91.18

Table F-2B M_{ur} **of Singly Reinforced and Doubly Reinforced Beam - 200 mm wide Continued ...**

M_{ur} Singly and Doubly Rein.Sect
M 20 , Fe 415
b = 200 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{st1} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	$V_{ur.min}$ kN	V_{uc} kN	V_{uc1} kN	$V_{ur.min}$ kN	$V_{ur.min1}$ kN
200	550	2 20 + 2 16	2 12 + 0 0	1030	628	226	2	156.07	39.36	62.28	51.82	101.64	91.18
200	550	2 20 + 2 16	2 10 + 1 12	1030	628	270	2	158.17	39.36	62.28	51.82	101.64	91.18
200	550	2 20 + 2 16	2 10 + 1 16	1030	628	358	2	161.70	39.36	62.28	51.82	101.64	91.18
200	580	2 12 + 0 0	Anchor Bars	226	226	–	1	42.81	43.84	36.25	36.25	80.09	80.09
200	580	2 10 + 1 12		270	157	–	1	50.69	43.84	39.13	30.88	82.97	74.72
200	580	2 12 + 1 10		305	226	–	1	56.78	43.84	41.19	36.25	85.03	80.09
200	580	2 10 + 2 10		314	157	–	1	58.54	43.92	41.77	30.91	85.69	74.83
200	580	2 12 + 1 12		339	226	–	1	62.78	43.84	43.10	36.25	86.94	80.09
200	580	2 10 + 1 16		358	157	–	1	65.75	43.68	43.99	30.82	87.67	74.50
200	580	2 10 + 2 12		383	157	–	1	70.28	43.84	45.34	30.88	89.18	74.72
200	580	2 16 + 0 0		402	402	–	1	73.17	43.68	46.15	46.15	89.83	89.83
200	580	2 12 + 1 16		427	226	–	1	77.34	43.68	47.32	36.17	91.00	79.85
200	580	2 12 + 2 12		452	226	–	1	81.78	43.84	48.54	36.5	92.38	80.09
200	580	3 12 + 1 12		452	339	–	1	81.78	43.84	48.54	43.10	92.38	86.94
200	580	2 16 + 1 12		515	402	–	1	91.56	43.68	51.05	46.15	94.73	89.83
200	580	3 12 + 1 16		540	339	–	1	95.51	43.68	52.03	43.01	95.71	86.69
200	580	2 16 + 2 12		628	402	–	1	108.99	43.68	55.23	46.15	98.91	89.83
200	580	2 20 + 0 0		628	628	–	1	108.08	43.36	54.98	54.98	98.34	98.34
200	580	2 20 + 1 12		741	628	–	1	124.39	43.36	58.62	54.98	101.98	98.34
200	580	2 20 + 1 16		829	628	–	1	136.41	43.36	61.16	54.98	104.52	98.34
200	580	2 20 + 2 12		855	628	–	2	133.57	41.76	60.41	53.74	102.17	95.50
200	580	2 20 + 0 12		968	628	–	2	147.15	41.76	63.23	53.74	104.99	95.50
200	580	2 25 + 0 0		982	982	–	1	154.98	43.16	64.91	64.91	108.07	108.07
200	580	2 20 + 2 16	2 8 + 0 0	1030	628	101	2	160.54	41.76	64.69	53.74	106.45	95.50
200	580	2 20 + 2 16	2 10 + 0 0	1030	628	157	2	163.70	41.76	64.69	53.74	106.45	95.50
200	580	2 20 + 2 16	2 12 + 0 0	1030	628	226	2	167.22	41.76	64.69	53.74	106.45	95.50
200	580	2 20 + 2 16	2 10 + 1 12	1030	628	270	2	169.32	41.76	64.69	53.74	106.45	95.50
200	580	2 20 + 2 16	2 10 + 1 16	1030	628	358	2	172.86	41.76	64.69	53.74	106.45	95.50
200	600	2 10 + 1 12	Anchor Bars	270	157	–	1	52.64	45.44	39.94	31.50	85.38	76.94
200	600	2 12 + 1 10		305	226	–	1	58.98	45.44	42.05	36.99	87.49	82.43
200	600	2 10 + 2 10		314	157	–	1	60.81	45.52	42.64	31.53	88.16	77.05
200	600	2 12 + 1 12		339	226	–	1	65.23	45.44	44.00	36.99	89.44	82.43
200	600	2 10 + 1 16		358	157	–	1	68.34	45.28	44.92	31.44	90.20	76.72
200	600	2 10 + 2 12		383	157	–	1	73.05	45.44	46.31	31.50	91.75	76.94
200	600	2 16 + 0 0		402	402	–	1	76.07	45.28	47.14	47.14	92.42	92.42
200	600	2 12 + 1 16		427	226	–	1	80.42	45.28	48.33	36.92	93.61	82.20
200	600	2 12 + 2 12		452	226	–	1	85.05	45.44	49.58	36.99	95.02	82.43
200	600	3 12 + 1 12		452	339	–	1	85.05	45.44	49.58	44.00	95.02	89.44
200	600	2 16 + 1 12		515	402	–	1	95.28	45.28	52.16	47.14	97.44	92.42
200	600	3 12 + 1 16		540	339	–	1	99.41	45.28	53.17	43.91	98.45	89.19
200	600	2 16 + 2 12		628	402	–	1	113.52	45.28	56.45	47.14	101.73	92.42
200	600	2 20 + 0 0		628	628	–	1	112.62	44.96	56.21	56.21	101.17	101.17

Table F-2B ***M_{ur} of Singly Reinforced and Doubly Reinforced Beam - 200 mm wide Continued ...***

M_{ur} Singly and Doubly Rein.Sect M 20 , Fe 415 b = 200 mm

b mm	D mm	N1–D1+N2–D2 mm mm	NC– Diac mm	A_{st} mm^2	A_{st1} mm^2	A_{sc} mm^2	RO	M_{ur} kN.m	$V_{usv.min}$ kN	V_{uc} kN	V_{uc1} kN	$V_{ur.min}$ kN	$V_{ur.min1}$ kN
200	600	2 20 + 1 12		741	628	–	1	129.74	44.96	59.94	56.21	104.90	101.17
200	600	2 20 + 1 16	Anchor	829	628	–	1	142.40	44.96	62.56	56.21	107.52	101.17
200	600	2 20 + 2 12		855	628	–	2	139.74	43.36	61.85	54.98	105.21	98.34
200	600	2 20 + 3 12	Bars	968	628	–	2	154.13	43.36	64.76	54.98	108.12	98.34
200	600	2 25 + 0 0		982	982	–	1	162.06	44.76	66.43	66.43	66.43	111.19
200	600	2 20 + 2 16		1030	628	–	2	161.71	43.36	66.26	54.98	109.62	98.34
200	650	2 0 + 1 12	A	270	157	–	1	57.52	49.44	41.91	33.00	91.35	82.44
200	650	2 12 + 1 10		305	226	–	1	64.48	49.44	44.13	38.79	93.57	88.23
200	650	2 10 + 2 10	n	314	157	–	1	66.47	49.52	44.75	33.03	94.27	82.55
200	650	2 12 + 1 12		339	226	–	1	71.35	49.44	46.20	38.79	95.64	88.23
200	650	2 10 + 1 16	c	358	157	–	1	74.80	49.28	47.18	32.95	96.46	82.23
200	650	2 10 + 2 12	h	383	157	–	1	79.97	49.44	48.64	33.00	98.08	82.44
200	650	2 16 + 0 0		402	402	–	1	83.32	49.28	49.53	49.53	98.81	98.81
200	650	2 12 + 1 16	o	427	226	–	1	88.13	49.28	50.80	38.72	100.08	88.00
200	650	2 12 + 2 12		452	226	–	1	93.21	49.44	52.11	38.79	101.55	88.23
200	650	3 12 + 1 12	r	452	339	–	1	93.21	49.44	52.11	46.20	101.55	95.64
200	650	2 16 + 1 12		515	402	–	1	104.57	49.28	54.86	49.53	104.14	98.81
200	650	3 12 + 1 16		540	339	–	1	109.16	49.28	55.93	46.11	105.21	95.39
200	650	2 16 + 2 12	B	628	402	–	1	124.86	49.28	59.41	49.53	108.69	98.81
200	650	2 20 + 0 0		628	628	–	1	123.95	48.96	59.18	59.18	108.14	108.14
200	650	2 20 + 1 12	a	741	628	–	1	143.12	48.96	63.16	59.18	112.12	108.14
200	650	2 20 + 1 16	r	829	628	–	1	157.36	48.96	65.95	59.18	114.91	108.14
200	650	2 20 + 0 12		855	628	–	2	155.16	47.36	65.35	58.01	112.71	105.37
200	650	2 20 + 3 12	s	968	628	–	2	171.59	47.36	68.47	58.01	115.83	105.37
200	650	2 25 + 0 0		982	982	–	1	179.78	48.76	70.11	70.11	118.87	118.87
200	650	2 20 + 2 16		1030	628	–	2	180.30	47.36	70.08	58.01	117.44	105.37

Note :-

N1 - D1	Represent Number - Daimeter bars going into the Support
N2 - D2	Represent Number - Diameter of Bent up bars.
NC - Diac	Represent Number - Diameter of Compression Steel.
A_{st}	Total area of Tension Steel in mm^2 Corresponding to *N1-D1 + N2-D2*
A_{st1}	Total area of Tension Steel going into the Support in mm^2 Corresponding to *N1-D1*
A_{sc}	Total area of Compression Steel in mm^2
$V_{usv.min}$	Shear resisted by Minimum Stirrups in *kN*
V_{uc}	Shear resisted by Concrete Corresponding to area of steel A_{st} in *kN*
V_{uc1}	Shear resisted by Concrete Corresponding to area of steel A_{st1} in *kN*
$V_{ur.min}$	Shear resisted by beam with minimum Stirrups Corresponding to A_{st} in *kN*
$V_{ur.min1}$	Shear resisted by beam with minimum Stirrups Corresponding to A_{st1} in *kN*

Table F-3A **Ultimate Moment of Resistance (M_{ur}) in *kN.m* of Flanged Section for $D_f \geq 100$ *mm* - Beam width 150 *mm***

Mild Environment - Concrete M20, Steel Fe 415
(for Nominal Cover See *Table C-1*)

M_{ur} Flanged Sect
M 20, Fe 415
b_w = 150 mm

b mm	D mm	N1-D1+N2-D2 mm mm	A_{st} mm^2	d mm	RO	b_f/b_w = 3	4	5	6	7	8
150	300	2 10 + 0 0	157.1	269.0	1	14.84	14.94	15.00	15.04	15.07	15.09
150	300	2 12 + 0 0	226.2	268.0	1	21.02	21.24	21.36	21.45	21.51	21.56
150	300	2 10 + 1 12	270.2	268.0	1	24.91	25.22	25.40	25.52	25.61	25.67
150	300	2 12 + 1 10	304.7	268.0	1	27.92	28.31	28.54	28.70	28.81	28.89
150	300	3 12 + 0 0	339.3	268.0	1	30.89	31.37	31.66	31.85	31.99	32.09
150	300	2 12 + 2 10	383.3	254.5	2	32.75	33.36	33.73	33.98	34.15	34.28
150	300	2 16 + 0 0	402.1	266.0	1	35.90	36.58	36.98	37.25	37.44	37.59
150	300	2 12 + 1 16	427.3	266.0	1	37.97	38.73	39.19	39.49	39.71	39.87
150	300	4 12 + 0 0	452.4	254.5	2	38.14	38.99	39.50	39.84	40.09	40.27
150	300	2 16 + 1 12	515.2	250.0	2	42.06	43.16	43.83	44.27	44.58	44.82
150	300	3 12 + 1 16	540.4	250.0	2	43.88	45.10	45.83	46.32	46.66	46.92
150	300	3 16 + 0 0	603.2	250.0	2	48.35	49.87	50.78	51.38	51.82	52.14
150	300	2 16 + 2 12	628.3	250.0	2	50.10	57.15	52.74	53.39	53.86	54.22
150	300	2 20 + 0 0	628.3	262.0	1	52.82	54.47	55.46	56.11	56.58	56.94
150	300	2 20 + 1 12	741.4	242.0	2	55.58	57.87	59.25	60.17	60.82	61.31
150	300	4 16 + 0 0	804.2	250.0	2	61.77	64.47	66.09	67.16	67.93	68.51
150	300	2 20 + 1 16	829.4	242.0	2	60.96	63.83	65.55	66.70	67.51	68.13
150	300	2 20 + 2 12	854.5	242.0	2	-	65.49	67.32	68.54	69.41	70.06
150	300	3 20 + 0 0	942.5	242.0	2	-	71.20	73.42	74.90	75.96	76.75
150	300	2 20 + 3 12	967.6	242.0	2	-	72.79	75.13	76.70	77.81	78.65
150	300	2 25 + 0 0	981.7	259.5	1	-	79.88	82.29	83.90	85.05	85.91
150	300	2 20 + 2 16	1030.4	242.0	2	-	76.71	79.37	81.14	82.40	83.35
150	300	2 25 + 1 12	1094.8	234.5	2	-	77.66	80.66	82.66	84.08	85.15
150	300	2 25 + 2 12	1207.9	234.5	2	-	77.66	87.62	90.06	91.79	93.10
150	350	2 10 + 0 0	157.1	319.0	1	17.67	17.77	17.84	17.88	17.91	17.93
150	350	2 12 + 0 0	226.2	318.0	1	25.10	25.32	25.45	25.53	25.59	25.64
150	350	2 10 + 1 12	270.2	318.0	1	29.79	30.09	30.27	30.40	30.48	30.55
150	350	2 12 + 1 10	304.7	318.0	1	33.42	33.81	34.04	34.20	34.31	34.39
150	350	3 12 + 0 0	339.3	318.0	1	37.02	37.50	37.78	37.98	38.11	38.22
150	350	2 12 + 2 10	383.3	304.5	2	39.67	40.28	40.65	40.89	41.07	41.20
150	350	2 16 + 0 0	402.1	316.0	1	43.16	43.83	44.24	44.51	44.70	44.85
150	350	2 12 + 1 16	427.3	316.0	1	45.68	46.44	46.90	47.20	47.42	47.58
150	350	4 12 + 0 0	452.4	304.5	2	46.30	47.15	47.66	48.00	48.25	48.43
150	350	2 16 + 1 12	515.2	300.0	2	51.35	52.46	53.12	53.57	53.88	54.12
150	350	3 12 + 1 16	540.4	300.0	2	53.63	54.85	55.58	56.06	56.41	56.67
150	350	3 16 + 0 0	603.2	300.0	2	59.23	60.75	61.66	62.27	62.70	63.03
150	350	2 16 + 2 12	628.3	300.0	2	61.44	63.09	64.07	64.73	65.20	65.55
150	350	2 20 + 0 0	628.3	312.0	1	64.16	65.81	66.79	67.45	67.92	68.27
150	350	2 20 + 1 12	741.4	292.0	2	68.96	71.24	72.63	73.54	74.20	74.69
150	350	4 16 + 0 0	804.2	300.0	2	76.28	78.98	80.60	81.68	82.45	83.02
150	350	2 20 + 1 16	829.4	292.0	2	75.93	78.79	80.51	81.66	82.48	83.09
150	350	2 20 + 2 12	854.5	292.0	2	77.87	80.91	82.74	83.96	84.83	85.48

Table F-3A *Ultimate Moment of Resistance of Flanged Section for $D_f \geq 100$ mm - Beam width 150 mm Continued ...*

M_{ur} Flanged Sect
M 20 , Fe 415
b_w = 150 mm

b	D	N1-D1+N2-D2		A_{st}	d	RO	b_f / b_w					
mm	mm	mm	mm	mm^2	mm		3	4	5	6	7	8
150	350	3 20 + 0	0	942.5	292.0	2	-	88.20	90.43	91.91	92.96	93.76
150	350	2 20 + 3	12	967.6	292.0	2	-	90.25	92.59	94.16	95.27	96.11
150	350	2 25 + 0	0	981.7	309.5	1	-	97.60	100.01	101.62	102.76	103.62
150	350	2 20 + 2	16	1030.4	292.0	2	-	95.30	97.96	99.73	100.99	101.94
150	350	2 25 + 1	12	1094.8	284.5	2	-	97.41	100.41	102.41	103.84	104.91
150	350	2 25 + 2	12	1207.9	284.5	2	-	97.41	109.42	111.85	113.59	114.89
150	380	2 10 + 0	0	157.1	349.0	1	19.37	19.47	19.54	19.58	19.61	19.63
150	380	2 12 + 0	0	226.2	348.0	1	27.55	27.77	27.89	27.98	28.04	28.09
150	380	2 10 + 1	12	270.2	348.0	1	32.71	33.02	33.20	33.32	33.41	33.47
150	380	2 12 + 1	10	304.7	348.0	1	36.72	37.11	37.34	37.50	37.61	37.69
150	380	3 12 + 0	0	339.3	348.0	1	40.69	41.17	41.46	41.65	41.79	41.89
150	380	2 12 + 2	10	383.3	334.5	2	43.82	44.43	44.80	45.04	45.22	45.35
150	380	2 16 + 0	0	402.1	346.0	1	47.51	48.19	48.59	48.86	49.05	49.20
150	380	2 12 + 1	16	427.3	346.0	1	50.30	51.06	51.52	51.83	52.04	52.21
150	380	4 12 + 0	0	452.4	334.5	2	51.20	52.05	52.56	52.90	53.15	53.33
150	380	2 16 + 1	12	515.2	330.0	2	56.93	58.04	58.70	59.14	59.46	59.70
150	380	3 12 + 1	16	540.4	330.0	2	59.48	60.70	61.43	61.91	62.26	62.52
150	380	3 16 + 0	0	603.2	330.0	2	65.76	67.28	68.19	68.80	69.23	69.56
150	380	2 16 + 2	12	628.3	330.0	2	68.24	69.89	70.87	71.53	72.00	72.36
150	380	2 20 + 0	0	628.3	342.0	1	70.96	72.61	73.60	74.25	74.72	75.08
150	380	2 20 + 1	12	741.4	322.0	2	76.99	79.28	80.65	81.57	82.22	82.72
150	380	4 16 + 0	0	804.2	330.0	2	84.99	87.69	89.38	90.38	91.15	91.73
150	380	2 20 + 1	16	829.4	322.0	2	84.90	87.77	89.49	90.64	91.46	92.07
150	380	2 20 + 2	12	854.5	322.0	2	87.12	90.16	91.99	93.21	94.08	94.73
150	380	3 20 + 0	0	942.5	322.0	2	-	98.41	100.63	102.11	103.17	103.96
150	380	2 20 + 3	12	967.6	322.0	2	-	100.73	103.07	104.63	105.75	106.58
150	380	2 25 + 0	0	981.7	339.5	1	-	108.23	110.64	112.24	113.39	114.25
150	380	2 20 + 2	16	1030.4	322.0	2	-	106.46	109.11	110.88	112.15	113.10
150	380	2 25 + 1	12	1094.8	314.5	2	-	109.27	112.27	114.26	115.69	116.76
150	380	2 25 + 2	12	1207.9	314.5	2	-	-	122.50	124.93	126.67	127.97
150	400	2 10 + 0	0	157.1	369.0	1	20.51	20.61	20.67	20.71	20.74	20.76
150	400	2 12 + 0	0	226.2	368.0	1	29.19	29.40	29.53	29.61	29.67	29.72
150	400	2 10 + 1	12	270.2	368.0	1	34.66	34.97	35.15	35.27	35.36	35.42
150	400	2 12 + 1	10	304.7	368.0	1	38.92	39.31	39.54	39.69	39.81	39..89
150	400	3 12 + 0	0	339.3	368.0	1	43.14	43.62	43.91	44.10	44.24	44.34
150	400	2 12 + 2	10	383.3	354.5	2	46.58	47.19	47.56	47.81	47.98	48.11
150	400	2 16 + 0	0	402.1	366.0	1	50.42	51.09	51.49	51.76	51.96	52.10
150	400	2 12 + 1	16	427.3	366.0	1	53.39	54.15	54.61	54.91	55.13	55.29
150	400	4 12 + 0	0	452.4	354.5	2	54.46	55.31	55.83	56.17	56.41	56.59
150	400	2 16 + 1	12	515.2	350.0	2	60.65	61.76	62.42	62.86	63.18	63.41
150	400	3 12 + 1	16	540.4	350.0	2	63.38	64.60	65.33	65.81	66.16	66.42

Table F-3A **Ultimate Moment of Resistance of Flanged Section for $D_f \geq 100$ mm - Beam width 150 mm Continued ...**

M_{ur} Flanged Sect
M 20 , Fe 415
b_w = 150 mm

b mm	D mm	N1-D1+N2 mm	D2 mm	A_{st} mm²	d mm	RO	b_f/b_w 3	4	5	6	7	8
150	400	3 16 + 0	0	603.2	350.0	2	70.12	71.64	72.55	73.15	73.59	73.91
150	400	2 16 + 2	12	628.3	350.0	2	72.78	74.42	75.41	76.07	76.54	76.89
150	400	2 20 + 0	0	628.3	362.0	1	75.50	77.14	78.13	78.79	79.26	79.61
150	400	2 20 + 1	12	741.4	342.0	2	82.34	84.63	86.00	86.92	87.58	88.07
150	400	4 16 + 0	0	804.2	350.0	2	90.79	93.49	95.11	96.19	96.96	97.54
150	400	2 20 + 1	16	829.4	342.0	2	90.89	93.76	95.48	96.63	97.44	98.06
150	400	2 20 + 2	12	854.5	342.0	2	93.29	96.33	98.16	99.37	100.24	100.90
150	400	3 20 + 0	0	942.5	342.0	2	-	105.21	107.43	108.91	109.97	110.76
150	400	2 20 + 3	12	967.6	342.0	2	-	107.71	110.05	111.61	112.73	113.57
150	400	2 25 + 0	0	981.7	359.5	1	-	115.31	117.72	119.33	120.48	121.34
150	400	2 20 + 2	16	1030.4	342.0	2	-	113.90	116.55	118.32	119.59	120.54
150	400	2 25 + 1	12	1094.0	334.5	2	-	117.17	120.17	122.17	123.59	124.66
150	400	2 25 + 2	12	1207.9	334.5	2	-	-	131.21	133.65	135.38	136.69
150	450	2 10 + 0	0	157.1	419.0	1	23.34	23.34	23.50	23.55	23.57	23.60
150	450	2 12 + 0	0	226.2	418.0	1	33.27	33.48	33.61	33.69	33.75	33.80
150	450	2 10 + 1	12	270.2	418.0	1	39.54	39.84	40.02	40.15	40.23	40.30
150	450	2 12 + 1	10	304.7	418.0	1	44.42	44.81	45.04	45.19	45.30	45.39
150	450	3 12 + 0	0	339.3	418.0	1	49.26	49.74	50.03	50.28	50.36	50.46
150	450	2 12 + 2	10	383.3	404.5	2	53.50	54.11	54.48	54.72	54.90	55.03
150	450	2 16 + 0	0	402.1	416.0	1	57.67	58.35	58.75	59.02	59.21	59.36
150	450	2 12 + 1	16	427.3	416.0	1	61.10	61.86	62.31	62.62	62.84	63.00
150	450	4 12 + 0	0	452.4	404.5	2	62.62	63.48	63.99	64.93	64.57	64.76
150	450	2 16 + 1	12	515.2	400.0	2	69.94	71.05	71.72	72.16	72.47	72.71
150	450	3 12 + 1	16	540.4	400.0	2	73.13	74.35	75.08	75.56	75.91	76.17
150	450	3 16 + 0	0	603.2	400.0	2	81.00	82.52	83.43	84.04	84.47	84.79
150	450	2 16 + 2	12	628.3	400.0	2	84.11	85.76	86.75	87.41	87.88	88.23
150	450	2 20 + 0	0	628.3	412.0	1	86.83	88.48	89.47	90.13	90.60	90.95
150	450	2 20 + 1	12	741.4	392.0	2	95.72	98.01	99.38	100.30	100.95	101.44
150	450	4 16 + 0	0	804.2	400.0	2	105.31	108.00	109.62	110.70	111.47	112.05
150	450	2 20 + 1	16	829.4	392.0	2	105.86	108.72	110.44	111.59	112.41	113.02
150	450	2 20 + 2	12	854.5	392.0	2	108.70	111.75	113.57	114.79	115.66	116.31
150	450	3 20 + 0	0	942.5	392.0	2	-	122.22	124.44	125.92	126.98	127.77
150	450	2 20 + 3	12	967.6	392.0	2	-	125.17	127.51	129.03	130.19	131.02
150	450	2 25 + 0	0	981.7	409.5	1	-	133.03	135.44	137.04	138.19	139.05
150	450	2 20 + 2	16	1030.4	392.0	2	-	132.49	135.14	136.91	138.18	139.13
150	450	2 25 + 1	12	1094.8	384.5	2	-	136.92	139.92	141.92	143.35	144.42
150	450	2 25 + 2	12	1207.9	384.5	2	-	-	153.01	155.44	157.18	158.48
150	500	2 10 + 0	0	157.1	469.0	1	26.17	26.28	26.34	26.38	26.41	26.43
150	500	2 12 + 0	0	226.2	468.0	1	37.35	37.56	37.69	37.77	37.84	37.88
150	500	2 10 + 1	12	270.2	468.0	1	44.41	44.72	44.90	45.02	45.11	45.17
150	500	2 12 + 1	10	304.7	468.0	1	49.92	50.30	50.54	50.69	50.80	50.89
150	500	3 12 + 0	0	339.3	468.0	1	55.38	55.86	55.15	56.34	56.48	56.58

Table F-3A *Ultimate Moment of Resistance of Flanged Section for $D_f \geq 100$ mm - Beam width 150 mm Continued ...*

M_{ur} Flanged Sect
M 20 , Fe 415
b_w = 150 mm

b mm	D mm	N1-D1+N2-D2 mm mm	A_{st} mm^2	d mm	RO	b_f/b_w 3	4	5	6	7	8
150	500	2 12 + 2 10	383.3	454.5	2	60.41	61.03	61.39	61.64	61.81	61.94
150	500	2 16 + 0 0	402.1	466.0	1	64.93	65.60	66.01	66.28	66.47	66.61
150	500	2 12 + 1 16	427.3	466.0	1	68.81	69.57	70.02	70.33	70.55	70.71
150	500	4 12 + 0 0	452.4	454.5	2	70.79	71.64	72.15	72.49	72.74	72.92
150	500	2 16 + 1 12	515.2	450.0	2	79.24	80.35	81.01	81.45	81.77	82.01
150	500	3 12 + 1 16	540.4	450.0	2	82.88	84.10	84.83	85.31	85.66	85.92
150	500	3 16 + 0 0	603.2	450.0	2	91.89	93.40	94.31	94.92	95.35	95.68
150	500	2 16 + 2 12	628.3	450.0	2	95.45	97.10	98.08	98.74	99.21	99.56
150	500	2 20 + 0 0	628.3	462.0	1	98.17	99.82	100.80	101.46	101.93	102.29
150	500	2 20 + 1 12	741.4	442.0	2	109.09	111.38	112.76	113.68	114.33	114.82
150	500	4 16 + 0 0	804.2	450.0	2	119.82	122.51	124.13	125.21	125.98	126.56
150	500	2 20 + 1 16	829.4	442.0	2	120.82	123.69	125.41	126.55	127.37	127.99
150	500	2 20 + 2 12	854.5	442.0	2	124.12	127.17	128.99	130.21	131.08	131.73
150	500	3 20 + 0 0	942.5	442.0	2	-	139.22	141.44	142.92	143.98	144.78
150	500	2 20 + 3 12	967.6	442.0	2	-	142.63	144.97	146.53	147.65	148.48
150	500	2 25 + 0 0	981.7	459.5	1	-	150.74	153.15	154.76	155.91	156.77
150	500	2 20 + 1 16	1030.4	442.0	2	-	151.08	153.74	155.51	156.77	157.72
150	500	2 25 + 1 12	1094.8	434.5	2	-	156.68	159.68	161.68	163.10	164.17
150	500	2 25 + 2 12	1207.9	434.5	2	-	-	174.80	177.24	178.98	180.28
150	530	2 10 + 0 0	157.1	499.0	1	27.87	27.98	28.04	28.08	28.11	28.13
150	530	2 12 + 0 0	226.2	498.0	1	39.80	40.01	40.14	40.22	40.28	40.33
150	530	2 10 + 1 12	270.2	498.0	1	47.34	47.64	47.82	47.95	48.03	48.10
150	530	2 12 + 1 10	304.7	498.0	1	53.22	53.60	53.84	53.99	54.10	54.18
150	530	3 12 + 0 0	339.3	498.0	1	59.06	59.54	59.82	60.02	60.15	60.26
150	530	2 12 + 2 10	383.3	484.5	2	64.56	65.18	65.54	65.79	65.96	66.09
150	530	2 16 + 0 0	402.1	496.0	1	69.28	69.95	70.36	70.63	70.82	70.97
150	530	2 12 + 1 16	427.3	496.0	1	73.43	74.19	74.65	74.95	75.17	75.33
150	530	4 12 + 0 0	452.4	484.5	2	75.68	76.54	77.05	77.39	77.63	77.82
150	530	2 16 + 1 12	515.2	480.0	2	84.82	85.93	86.59	87.03	87.35	87.59
150	530	3 12 + 1 16	540.4	480.0	2	88.73	89.95	90.68	91.16	91.51	91.77
150	530	3 16 + 0 0	603.2	480.0	2	98.42	99.93	100.84	101.45	101.88	102.21
150	530	2 16 + 2 12	628.3	480.0	2	102.25	103.90	104.89	105.54	106.01	106.37
150	530	2 20 + 0 0	628.3	492.0	1	104.97	106.62	107.61	108.27	108.74	109.09
150	530	2 20 + 1 13	741.4	472.0	2	117.12	119.41	120.79	121.70	122.36	122.85
150	530	4 16 + 0 0	804.2	480.0	2	128.52	131.22	132.84	133.92	134.69	135.27
150	530	2 20 + 1 16	829.4	472.0	2	129.80	132.67	134.39	135.53	136.35	136.97
150	530	2 20 + 2 12	854.5	472.0	2	133.37	136.42	138.24	139.46	140.33	140.98
150	530	3 20 + 0 0	942.5	472.0	2	-	149.42	151.65	153.13	154.19	154.98
150	530	2 20 + 3 12	967.6	472.0	2	-	153.10	155.45	157.01	158.12	158.96
150	530	2 25 + 0 0	981.7	489.5	1	-	161.37	163.78	165.39	166.53	167.39
150	530	2 20 + 2 16	1030.4	472.0	2	-	162.24	164.89	166.66	167.93	168.88
150	530	2 25 + 1 12	1094.8	464.5	2	-	168.53	171.53	173.53	174.96	176.03
150	530	2 25 + 2 12	1207.9	464.5	2	-	-	187.88	190.32	192.05	193.36

Table F-3A ***Ultimate Moment of Resistance of Flanged Section for $D_f \geq 100$ mm - Beam width 150 mm Continued ...***

M_{ur} Flanged Sect
M 20 , Fe 415
b_w = 150 mm

b mm	D mm	N1	-D1+N2 mm	-D2 mm	A_{st} mm^2	d mm	RO	b_f / b_w 3	4	5	6	7	8
150	550	2	12 + 0	0	226.2	518.0	1	41.43	41.64	41.77	41.86	41.92	41.96
150	550	2	10 + 1	12	270.2	518.0	1	49.29	49.59	49.77	49.90	49.98	50.05
150	550	2	12 + 1	10	304.7	518.0	1	55.42	55.80	56.04	56.19	56.30	56.38
150	550	3	12 + 0	0	339.3	518.0	1	61.50	61.98	62.27	62.46	62.60	62.70
150	550	2	12 + 2	10	383.3	504.5	2	67.33	67.94	68.31	68.55	68.73	68.86
150	550	2	16 + 0	0	402.1	516.0	1	72.18	72.86	73.26	73.53	73.72	73.87
150	550	2	12 + 1	16	427.3	516.0	1	76.50	76.28	77.73	78.04	78.25	78.42
150	550	4	12 + 0	0	452.4	504.5	2	78.95	79.80	80.31	80.66	80.90	81.08
150	550	2	16 + 1	12	515.2	500.0	2	88.54	89.64	90.31	90.75	91.75	91.30
150	550	3	12 + 1	16	540.4	500.0	2	92.63	93.85	94.58	95.06	95.41	95.67
150	550	3	16 + 0	0	603.2	500.0	2	102.77	104.29	105.20	105.80	106.24	106.56
150	550	2	16 + 2	12	628.3	500.0	2	106.79	108.43	109.42	110.42	110.55	110.90
150	550	2	20 + 0	0	628.3	512.0	1	109.51	111.51	112.14	112.80	113.27	113.62
150	550	2	20 + 1	12	741.4	492.0	2	122.47	124.76	126.14	127.05	127.71	128.20
150	550	4	16 + 0	0	804.2	500.0	2	134.33	137.03	138.64	139.72	140.49	141.07
150	550	2	20 + 1	16	829.4	492.0	2	135.79	138.65	140.37	141.52	142.34	142.95
150	550	2	20 + 2	12	854.5	492.0	2	139.54	142.59	144.41	145.63	146.50	147.15
150	550	3	20 + 0	0	942.5	492.0	2	-	156.23	158.45	159.93	160.99	161.78
150	550	2	20 + 3	12	967.6	492.0	2	-	160.09	162.43	163.99	165.11	165.94
150	550	2	25 + 0	0	981.7	509.5	1	-	168.45	170.86	172.47	173.62	174.48
150	550	2	20 + 2	16	1030.4	492.0	2	-	169.67	172.33	174.10	175.37	176.31
150	550	2	25 + 1	12	1094.8	484.5	2	-	176.43	179.43	181.43	182.86	183.93
150	550	2	25 + 2	12	1207.9	484.5	2	-	-	196.60	199.03	200.77	202.07
150	600	2	12 + 0	0	226.2	568.0	1	45.51	45.72	45.85	45.94	46.00	46.04
150	600	2	10 + 1	12	270.2	568.0	1	54.16	54.47	54.65	54.77	54.86	54.92
150	600	2	12 + 1	10	304.7	568.0	1	60.91	61.30	61.53	61.69	61.80	61.88
150	600	3	12 + 0	0	339.3	568.0	1	67.63	68.11	68.39	68.59	68.72	68.83
150	600	2	12 + 2	10	383.3	554.5	2	74.24	74.86	75.22	75.47	75.64	75.78
150	600	2	16 + 0	0	402.1	566.0	1	79.44	80.11	80.52	80.79	80.98	81.12
150	600	2	12 + 1	16	427.3	566.0	1	84.22	84.99	85.44	85.75	85.96	86.13
150	600	4	12 + 0	0	452.4	554.5	2	87.11	87.96	88.48	88.82	89.06	89.24
150	600	2	16 + 1	12	515.2	550.0	2	97.83	98.94	99.60	100.05	100.36	100.60
150	600	3	12 + 1	16	540.4	550.0	2	102.38	103.60	104.33	104.81	105.16	105.42
150	600	3	16 + 0	0	603.2	550.0	2	113.17	115.17	116.08	116.69	117.12	117.44
150	600	2	16 + 2	12	628.3	550.0	2	118.12	119.77	120.76	121.42	121.89	122.24
150	600	2	20 + 0	0	628.3	562.0	1	120.85	122.49	123.48	124.14	124.61	124.96
150	600	2	20 + 1	12	741.4	542.0	2	135.85	138.14	139.52	140.43	141.09	141.58
150	600	4	16 + 0	0	804.2	550.0	2	148.84	151.54	153.15	154.23	155.00	155.58
150	600	2	20 + 1	16	829.4	542.0	2	150.75	153.62	155.34	156.48	157.30	157.92
150	600	2	20 + 2	12	854.5	542.0	2	154.96	158.00	159.83	161.05	161.92	162.57
150	600	3	20 + 0	0	942.5	542.0	2	-	173.23	175.45	176.93	177.99	178.79
150	600	2	20 + 3	12	967.6	542.0	2	-	177.55	179.89	182.45	182.57	183.40
150	600	2	25 + 0	0	981.7	559.5	1	-	186.17	188.58	190.19	191.33	192.19

Table F-3A ***Ultimate Moment of Resistance of Flanged Section for $D_f \geq 100$ mm - Beam width 150 mm Continued ...***

M_{ur} Flanged Sect
M 20 , Fe 415
b_w = 150 mm

b mm	D mm	N1-D1+N2-D2 mm mm	A_{st} mm^2	d mm	RO	b_f/b_w 3	4	5	6	7	8
150	600	2 20+ 2 16	1030.4	542.0	2	-	188.27	190.92	192.69	193.96	194.91
150	600	2 25+ 1 12	1094.8	534.5	2	-	196.19	199.19	201.19	202.61	203.68
150	600	2 25+ 2 12	1207.9	534.9	2	-	-	218.40	220.83	222.57	223.87
150	650	2 12 + 0 0	226.2	618.0	1	49.59	49.81	49.93	50.02	50.08	50.13
150	650	2 10 + 1 12	270.2	618.0	1	59.04	59.34	59.52	59.65	59.73	59.80
150	650	2 12 + 1 10	304.7	618.0	1	66.41	66.80	67.03	67.19	67.30	67.38
150	650	3 12 + 0 0	339.3	618.0	1	73.23	74.23	74.52	74.71	74.85	74.95
150	650	2 12 + 2 10	383.3	604.5	2	81.16	.81.77	82.14	82.38	82.56	82.69
150	650	2 16 + 0 0	402.1	616.0	1	86.69	87.37	87.77	88.04	88.23	88.38
150	650	2 12 + 1 16	427.3	616.0	1	91.93	92.69	93.15	93.46	93.67	93.84
150	650	4 12 + 0 0	452.4	604.5	2	95.27	96.13	96.64	96.98	97.22	97.41
150	650	2 16 + 1 12	515.2	600.0	2	107.13	108.24	108.90	109.34	109.66	109.90
150	650	3 12 + 1 16	540.4	600.0	2	112.13	113.35	114.08	114.56	114.91	115.17
150	650	3 16 + 0 0	603.2	600.0	2	.124.54	126.05	126.96	127.57	128.00	128.33
150	.650	2 16 + 2 12	628.3	600.0	2	129.46	131.11	132.10	132.75	133.22	133.58
150	650	2 20 + 0 0	628.3	612.0	1	132.18	133.83	134.82	135.47	135.94	136.30
150	650	2 20 + 1 12	741.4	592.0	2	149.23	151.52	152.89	153.81	154.46	154.96
150	650	4 16 + 0 0	804.2	600.0	2	163.35	166.05	167.67	168.74	169.51	170.09
150	650	2 20 + 1 16	829.4	592.0	2	165.71	168.58	170.30	171.45	172.27	172.88
150	650	2 20 + 2 12	854.5	592.0	2	170.38	173.42	175.25	176.47	177.34	177.99
150	650	3 20 + 0 0	942.5	592.0	2	-	190.24	192.46	193.94	195.00	195.79
150	650	2 20 + 3 12	967.6	592.0	2	-	195.01	197.35	198.91	200.02	200.86
150	650	2 25 + 0 0	981.7	609.5	1	-	203.88	206.29	207.90	209.05	209.91
150	650	2 20 + 2 16	1030.4	592.0	2	-	206.86	209.52	211.29	212.55	213.50
150	650	2 25 + 1 12	1094.8	584.5	2	-	215.94	218.94	220.94	222.37	223.44
150	650	2 25 + 2 12	1207.9	584.5	2	-	-	240.19	242.62	244.36	245.67

Note :- Dashes indicate that for the Number - Diameter combination of bars, Width of flange is such that the *depth of the neutral axis* works out to be greater than the depth of the flange of 100 *mm*.

Table F-3B **Ultimate Moment of Resistance (M_{ur}) in *kN.m* of Flanged Section for $D_f \geq 100$ *mm* - Beam width 200 *mm***
Mild Environment - Concrete M20, Steel Fe 415
(for Nominal Cover See *Table C-1*)

M_{ur} Flanged Sect
M 20 , Fe 415
b_w = 200 mm

b *mm*	*D* *mm*	*N1-D1* *mm*		+	*N2-D2* *mm*		A_{st} mm^2	*d* *mm*	*RO*	b_f / b_w					
										3	4	5	6	7	8
200	300	2	10	+	0	0	157.1	269.0	1	14.94	15.02	15.06	15.09	15.12	15.13
200	300	2	12	+	0	0	226.2	268.0	1	21.24	21.40	21.49	21.56	21.60	21.64
200	300	2	10	+	1	12	270.2	268.0	1	25.22	25.44	25.58	25.67	25.74	25.79
200	300	2	12	+	1	10	304.7	268.0	1	28.31	28.60	28.78	28.89	28.97	29.04
200	300	3	12	+	0	0	339.3	268.0	1	31.37	31.73	31.95	32.09	32.20	32.27
200	300	2	12	+	2	10	383.3	268.0	1	35.23	35.69	35.97	36.15	36.28	36.38
200	300	2	16	+	0	0	402.1	266.0	1	36.58	37.08	37.39	37.59	37.73	37.84
200	300	2	12	+	1	16	427.3	266.0	1	38.73	39.30	39.64	39.87	40.03	40.16
200	300	4	12	+	0	0	452.4	268.0	1	41.19	41.83	42.22	42.47	42.66	42.79
200	300	2	16	+	1	12	515.2	266.0	1	46.14	46.97	47.46	47.80	48.03	48.21
200	300	3	12	+	1	16	540.4	266.0	1	48.22	49.13	49.68	50.04	50.30	50.50
200	300	3	16	+	0	0	603.2	266.0	1	53.35	54.49	55.17	55.63	55.95	56.19
200	300	2	16	+	2	12	628.3	266.0	1	55.38	56.61	57.35	57.84	58.20	58.46
200	300	2	20	+	0	0	628.3	262.0	1	54.47	55.70	56.44	56.94	57.29	57.55
200	300	2	20	+	1	12	741.4	262.0	1	63.22	64.94	65.97	66.66	67.15	67.52
200	300	4	16	+	0	0	804.2	266.0	1	69.11	71.13	72.35	73.16	73.73	74.17
200	300	2	20	+	1	16	829.4	262.0	1	69.81	71.96	73.25	74.11	74.73	75.19
200	300	2	20	+	2	12	854.5	242.0	2	65.49	67.78	69.15	70.06	70.71	71.20
200	300	3	20	+	0	0	942.5	262.0	1	78.00	80.78	82.44	83.56	84.35	84.94
200	300	2	20	+	3	12	967.6	242.0	2	72.79	75.72	77.48	78.65	79.48	80.11
200	300	2	25	+	0	0	981.7	259.5	1	79.88	82.90	84.70	85.91	86.77	87.42
200	300	2	20	+	2	16	1030.4	242.0	2	76.71	80.03	82.02	83.35	84.30	85.01
200	300	2	25	+	1	12	1094.8	259.5	1	88.54	81.28	93.53	95.03	96.10	96.91
200	300	2	25	+	2	12	1207.9	234.5	2	–	88.54	91.27	93.10	94.40	95.38
200	350	2	10	+	0	0	157.1	269.0	1	17.77	17.85	17.90	17.93	17.95	17.97
200	350	2	12	+	0	0	226.2	318.0	1	25.32	25.48	25.57	25.64	25.68	25.72
200	350	2	10	+	1	12	270.2	318.0	1	30.09	30.32	30.46	30.55	30.61	30.66
200	350	2	12	+	1	10	304.7	318.0	1	33.81	34.10	34.27	34.39	34.47	34.53
200	350	3	12	+	0	0	339.3	318.0	1	37.50	37.86	38.07	38.22	38.32	38.40
200	350	2	12	+	2	10	383.3	318.0	1	42.15	42.61	42.88	43.06	43.20	43.29
200	350	2	16	+	0	0	402.1	316.0	1	43.83	44.34	44.64	44.85	44.99	45.10
200	350	2	12	+	1	16	427.3	316.5	1	46.44	47.01	47.35	47.58	47.74	47.87
200	350	4	12	+	0	0	452.4	318.0	1	49.36	50.00	50.38	50.63	50.82	50.95
200	350	2	16	+	1	12	515.2	316.0	1	55.43	56.26	56.76	57.09	57.33	57.51
200	350	3	12	+	1	16	540.4	316.5	1	57.97	58.88	59.43	59.79	60.05	60.25
200	350	3	16	+	0	0	603.2	316.0	1	64.23	65.37	66.05	66.51	66.83	67.08
200	350	2	16	+	2	12	628.3	316.0	1	66.71	67.95	68.69	69.18	69.53	69.80
200	350	2	20	+	0	0	628.3	312.0	1	65.81	67.04	67.78	68.27	68.63	68.89

Table F-3B ***Ultimate Moment of Resistance of Flanged Section for $D_f \geq 100$ mm, Beam width = 200 mm Continued ...***

M_{ur} Flanged Sect
M 20 , Fe 415
b_w = 200 mm

b mm	D mm	N1-D1 + N2-D2 mm mm	A_{st} mm^2	d mm	RO	b_f / b_w 3	4	5	6	7	8
200	350	2 20 + 1 12	741.4	312.0	1	76.60	78.32	79.35	80.04	80.53	80.90
200	350	4 16 + 0 0	804.2	316.0	1	83.62	85.65	86.86	87.67	88.25	88.68
200	350	2 20 + 1 16	829.4	312.0	1	84.78	86.93	88.22	89.09	89.69	90.16
200	350	2 20 + 2 12	854.5	292.0	2	80.91	83.19	84.56	85.48	86.13	86.62
200	350	3 20 + 0 0	942.5	312.0	1	95.01	97.78	99.45	100.56	101.35	101.95
200	350	2 20 + 3 12	967.6	242.0	2	72.79	75.72	77.48	78.65	79.48	80.11
200	350	2 25 + 0 0	981.7	309.5	1	97.60	100.61	102.42	103.62	104.48	105.13
200	350	2 20 + 2 16	1030.4	292.0	2	95.30	98.62	100.61	101.94	102.89	103.60
200	350	2 25 + 1 12	1094.8	309.5	1	107.29	111.04	113.29	114.29	115.86	116.66
200	350	2 25 + 2 12	1207.9	284.5	2	–	110.33	113.07	114.89	116.20	117.17
200	380	2 10 + 0 0	157.1	349.0	1	19.47	19.55	19.60	19.63	19.65	19.67
200	380	2 12 + 0 0	226.2	348.0	1	27.77	27.93	28.02	28.09	28.13	28.17
200	380	2 10 + 1 12	270.2	348.0	1	33.02	33.24	33.38	33.47	33.54	33.59
200	380	2 12 + 1 10	304.7	348.0	1	37.11	37.40	37.57	37.69	37.77	37.83
200	380	3 12 + 0 0	339.3	348.0	1	41.17	41.53	41.75	41.89	41.99	42.07
200	380	2 12 + 2 10	383.3	348.0	1	46.30	46.75	47.03	47.21	47.35	47.44
200	380	2 16 + 0 0	402.1	346.0	1	48.19	48.69	49.00	46.20	49.34	49.45
200	380	2 12 + 1 16	427.3	346.0	1	51.06	51.64	51.98	52.21	52.37	52.49
200	380	4 12 + 0 0	452.4	348.0	1	54.25	54.89	55.28	55.53	55.72	55.85
200	380	2 16 + 1 12	515.2	346.0	1	61.01	61.84	62.34	62.67	62.91	63.09
200	380	3 12 + 1 16	540.4	346.0	1	63.82	64.73	65.28	65.64	65.90	66.10
200	380	3 16 + 0 0	603.2	346.0	1	70.76	71.90	72.58	73.04	73.36	73.61
200	380	2 16 + 2 12	628.3	346.0	1	73.52	74.75	75.49	75.98	76.34	76.60
200	380	2 20 + 0 0	628.3	342.0	1	72.61	73.84	74.58	75.08	75.43	75.69
200	380	2 20 + 1 12	741.4	342.0	1	84.63	86.35	87.38	88.07	88.56	88.93
200	380	4 16 + 0 0	804.2	346.0	1	92.33	94.35	95.57	96.37	96.95	97.39
200	380	2 20 + 1 16	829.4	342.0	1	93.76	95.91	97.20	98.06	98.67	99.13
200	380	2 20 + 2 12	854.5	322.0	2	90.16	92.45	93.82	94.73	95.38	95.87
200	380	3 20 + 0 0	942.5	342.0	1	105.21	107.99	109.65	110.76	111.56	112.15
200	380	3 20 + 3 12	967.6	322.0	2	100.73	103.65	105.41	106.58	107.42	108.05
200	380	2 25 + 0 0	981.7	339.5	1	108.23	111.24	113.05	114.25	115.11	115.76
200	380	2 20 + 2 16	1030.4	322.0	2	106.46	109.78	111.77	113.10	114.05	114.76
200	380	2 25 + 2 12	1094.8	339.5	1	119.14	122.89	125.14	124.64	127.71	128.51
200	380	2 25 + 1 12	1207.9	314.5	2	–	123.41	106.15	127.97	129.27	130.25
200	400	2 12 + 0 0	226.2	368.0	1	29.40	29.56	29.65	29.72	29.76	29.80
200	400	2 10 + 1 12	270.2	368.0	1	34.97	35.19	35.33	35.42	35.49	35.54
200	400	2 12 + 1 10	304.7	368.0	1	39.31	39.60	39.77	39.89	39.97	40.03
200	400	3 12 + 0 0	339.3	368.0	1	43.62	43.98	44.19	44.34	44.44	44.52

Table F-3B **Ultimate Moment of Resistance of Flanged Section for $D_f \geq 100$ mm, Beam width = 200 mm Continued ...**

M_{ur} Flanged Sect
M 20 , Fe 415
b_w = 200 mm

b mm	D mm	N1	D1 mm	+	N2	D2 mm	A_{st} mm^2	d mm	RO	b_f/b_w = 3	4	5	6	7	8
200	400	2	12	+	2	10	383.3	368.0	1	49.06	49.52	49.80	49.98	50.11	50.21
200	400	2	16	+	0	0	402.1	366.0	1	51.09	51.60	51.90	52.10	52.25	55.58
200	400	2	12	+	1	16	427.3	366.0	1	54.15	54.72	55.06	55.29	55.45	55.58
200	400	4	12	+	0	0	452.4	368.0	1	57.52	58.16	58.54	58.80	58.98	59.12
200	400	2	16	+	1	12	515.2	366.0	1	64.73	65.56	66.06	66.39	66.63	66.80
200	400	3	12	+	1	16	540.4	366.0	1	67.72	68.63	69.18	69.54	69.80	70.00
200	400	3	16	+	0	0	603.2	366.0	1	75.12	76.26	76.94	77.39	77.72	77.96
200	400	2	16	+	2	12	628.3	366.0	1	78.05	79.28	80.03	80.52	80.87	81.14
200	400	2	20	+	0	0	628.3	362.0	1	77.14	78.38	79.12	79.61	79.96	80.23
200	400	2	20	+	1	12	741.4	362.0	1	89.98	91.70	92.73	93.42	93.91	94.28
200	400	4	16	+	0	0	804.2	366.0	1	98.13	100.16	101.37	102.18	102.76	103.19
200	400	2	20	+	1	16	829.4	362.0	1	99.74	101.89	103.18	104.04	104.66	105.12
200	400	2	20	+	1	2	854.5	342.0	2	96.33	98.61	99.98	100.90	101.55	102.04
200	400	3	20	+	0	0	942.5	362.0	1	112.01	114.79	116.46	117.57	118.6	118.95
200	400	2	20	+	3	2	967.6	342.0	2	107.41	110.64	112.39	121.57	114.40	115.03
200	400	2	25	+	0	0	981.7	359.0	1	115.31	118.32	120.13	101.34	122.20	122.84
200	400	2	20	+	2	16	1030.4	342.0	2	113.90	117.22	119.21	120.54	121.48	122.20
200	400	2	25	+	1	12	1094.8	359.5	2	127.05	130.79	133.04	134.54	135.61	136.42
200	400	2	25	+	1	12	1207.9	334.0	2	–	132.13	134.86	136.69	137.99	138.97
200	450	2	12	+	0	0	226.2	418.0	1	33.48	33.64	33.74	33.80	33.85	33.88
200	450	2	10	+	1	10	270.2	418.0	1	39.84	40.07	40.21	40.30	40.36	40.41
200	450	2	10	+	1	10	304.7	418.0	1	44.81	45.10	45.27	45.39	45.47	45.53
200	450	3	12	+	0	0	339.3	418.0	1	49.74	50.10	50.32	50.46	50.56	50.64
200	450	2	12	+	2	10	383.3	418.0	1	55.98	56.44	56.71	56.90	57.03	57.13
200	450	2	16	+	0	0	402.1	416.0	1	58.35	58.85	59.15	59.36	59.50	59.61
200	450	2	12	+	1	16	427.3	416.0	1	61.86	62.43	62.77	63.00	63.16	63.28
200	450	4	12	+	0	0	452.4	418.0	1	65.68	66.32	66.70	66.69	67.14	67.28
200	450	2	16	+	2	12	515.2	416.0	1	74.03	74.86	75.35	75.69	75.92	76.10
200	450	3	12	+	1	16	540.4	416.0	1	77.47	78.38	78.93	79.29	79.55	79.75
200	450	3	16	+	0	0	603.2	416.0	1	86.00	87.14	87.82	88.28	88.60	88.85
200	450	2	16	+	2	12	628.3	416.0	1	89.39	90.62	91.36	91.86	92.21	92.47
200	450	2	20	+	0	0	628.3	412.0	1	88.48	89.71	90.46	90.95	91.30	91.57
200	450	2	20	+	1	12	741.4	412.0	1	103.36	105.08	106.11	106.80	107.29	107.65
200	450	4	16	+	0	0	804.2	416.0	1	112.65	114.67	115.88	116.69	117.27	117.70
200	450	2	20	+	1	16	829.4	412.0	1	114.71	116.86	118.15	119.01	119.62	120.08
200	450	2	20	+	2	12	854.5	392.0	2	111.75	114.03	115.40	116.31	116.97	117.46
200	450	3	20	+	0	0	942.5	412.0	1	129.02	131.79	133.46	134.57	135.37	135.96
200	450	2	20	+	3	12	967.6	392.0	2	125.17	128.10	129.85	131.02	131.86	132.49
200	450	2	25	+	0	0	981.7	409.5	1	133.03	136.04	137.85	139.05	139.91	140.56

Table F-3B *Ultimate Moment of Resistance of Flanged Section for $D_f \geq 100$ mm, Beam width = 200 mm Continued ...*

M_{ur} Flanged Sect
M 20 , Fe 415
b_w = 200 mm

b mm	D mm	N1	D1 mm	+	N2	D2 mm	A_{st} mm^2	d mm	RO	b_f / b_w 3	4	5	6	7	8
200	450	2	20	+	2	16	1030.4	392.0	2	132.49	135.81	137.80	139.13	140.08	140.79
200	450	2	25	+	1	12	1094.8	409.5	1	146.80	150.55	152.80	154.30	156.37	156.17
200	450	2	25	+	2	12	1207.9	384.5	2	–	153.92	156.66	158.48	159.79	160.76
200	500	2	12	+	0	0	226.2	468.0	1	37.56	37.72	37.82	37.88	37.97	37.96
200	500	2	10	+	1	12	270.2	468.0	1	44.72	4494	45.08	45.17	45.24	45.29
200	500	2	12	+	1	10	304.7	468.0	1	50.30	50.59	50.77	50.89	50.97	51.03
200	500	3	12	+	0	0	339.3	468.0	1	55.86	56.22	56.44	56.58	56.69	56.76
200	500	2	12	+	2	10	383.3	468.0	1	62.89	63.35	63.63	63.81	63.94	64.04
200	500	2	16	+	0	0	402.1	466.0	1	65.60	66.11	66.41	66.61	66.76	66.86
200	500	4	12	+	0	0	452.4	468.0	1	69.57	70.14	70.48	70.71	70.87	70.99
200	500	2	16	+	1	12	515.2	466.0	1	83.32	84.15	84.65	84.98	85.22	85.40
200	500	3	12	+	1	16	540.4	466.0	1	87.22	88.13	88.68	89.04	89.30	89.50
200	500	3	16	+	0	0	603.2	466.0	1	96.88	98.02	98.70	99.16	99.48	99.73
200	500	2	16	+	2	12	628.3	466.0	1	100.72	101.96	102.70	103.19	103.55	106.81
200	500	2	20	+	0	0	628.3	462.0	1	99.82	101.05	101.79	102.29	102.64	102.90
200	500	2	20	+	1	12	741.4	462.0	1	116.74	118.45	119.49	120.17	120.66	121.03
200	500	4	16	+	0	0	804.2	466.0	1	127.16	129.18	130.39	131.20	131.78	132.21
200	500	2	20	+	1	16	829.4	462.0	1	129.67	131.82	133.11	133.97	134.59	135.05
200	500	2	20	+	2	12	854.5	442.0	2	127.17	129.45	130.82	131.73	132.39	132.87
200	500	3	20	+	0	0	942.5	462.0	1	146.02	140.80	150.47	151.58	152.37	152.97
200	500	2	20	+	3	12	967.6	392.0	2	142.63	145.56	147.31	148.48	149.32	149.95
200	500	2	25	+	0	0	981.7	459.0	1	150.74	153.75	155.56	156.77	157.63	158.27
200	500	2	20	+	2	16	1030.4	442.0	2	151.08	154.40	156.39	157.72	158.67	159.38
200	500	2	25	+	1	12	1094.8	459.5	1	166.56	170.30	172.55	174.05	175.12	175.93
200	500	2	25	+	2	12	1207.9	434.0	2	–	175.72	178.45	180.28	181.58	182.56
200	530	2	12	+	0	0	226.2	498.0	1	40.01	40.17	40.27	40.33	40.38	40.41
200	530	2	0	+	1	12	270.2	498.0	1	47.64	47.87	48.01	48.10	48.16	48.21
200	530	2	12	+	1	10	304.7	498.0	1	53.60	53.89	54.07	54.18	54.27	54.33
200	530	3	12	+	0	0	339.3	498.0	1	59.54	59.90	60.11	60.26	60.36	60.44
200	530	2	12	+	0	10	383.3	498.0	1	67.04	67.50	67.78	67.96	68.09	68.19
200	530	2	16	+	0	0	402.1	496.0	1	69.95	70.46	70.76	70.97	71.11	71.22
200	530	2	12	+	1	16	427.3	496.0	1	74.19	74.19	74.76	75.11	75.33	75.50
200	530	4	12	+	0	0	452.4	498.0	1	78.74	78.74	79.38	79.76	80.02	80.20
200	530	2	16	+	1	12	515.2	496.0	1	88.90	89.73	90.23	90.56	90.80	90.98
200	530	3	12	+	1	16	540.4	496.0	1	93.07	93.98	94.53	94.89	95.15	95.35
200	530	3	16	+	0	0	603.2	496.0	1	103.42	104.55	105.24	105.69	106.02	106.26
200	530	2	16	+	2	12	628.3	496.0	1	107.53	108.76	109.50	111.00	110.35	110.61
200	530	2	20	+	0	0	628.3	492.0	1	103.62	107.85	108.59	109.09	109.44	109.71

Table F-3B Ultimate Moment of Resistance of Flanged Section for $D_f \geq 100$ mm, Beam width = 200 mm Continued ...

M_{ur} Flanged Sect
M 20 , Fe 415
b_w = 200 mm

b mm	D mm	N1	D1 mm	+	N2	D2 mm	A_{st} mm^2	d mm	RO	b_f/b_w 3	4	5	6	7	8
200	530	2	20	+	1	12	741.4	492.0	1	124.76	126.48	127.51	128.20	128.69	129.06
200	530	4	16	+	0	0	804.2	496.0	1	135.86	137.89	139.10	139.91	140.49	140.92
200	530	2	20	+	1	16	829.4	492.0	1	138.65	140.80	142.09	142.95	143.57	144.03
200	530	2	20	+	2	12	854.5	475.0	2	136.42	138.70	140.07	140.98	141.64	142.13
200	530	3	20	+	0	0	942.5	492.0	1	156.23	159.00	160.67	161.78	162.57	163.17
200	530	2	20	+	3	12	967.6	472.0	2	153.10	156.03	157.79	158.96	159.80	160.42
200	530	2	25	+	0	0	981.7	489.5	1	161.37	164.38	166.19	167.39	168.26	168.90
200	530	3	20	+	1	12	1030.4	472.0	2	162.24	165.56	167.55	168.88	169.82	170.54
200	530	2	25	+	1	12	1094.8	489.5	1	178.41	182.16	184.41	185.90	186.98	187.78
200	530	3	20	+	2	10	1207.9	464.0	2	–	188.79	191.53	193.36	194.66	195.64
200	550	2	12	+	0	0	226.2	518.0	1	41.64	41.80	41.90	41.96	42.01	42.04
200	550	2	10	+	1	12	270.2	518.0	1	49.59	49.82	49.96	50.05	50.11	50.16
200	550	2	12	+	1	10	304.7	518.0	1	55.80	56.09	56.27	56.38	56.47	56.53
200	550	3	12	+	0	0	339.3	518.0	1	61.98	62.34	62.56	62.70	62.81	62.88
200	550	2	12	+	2	10	383.3	518.0	1	69.81	70.27	70.54	70.73	70.86	70.96
200	550	2	16	+	0	0	402.1	516.0	1	72.86	73.36	73.67	73.87	74.01	74.12
200	550	2	12	+	1	16	427.3	516.0	1	77.28	77.85	78.19	78.42	78.58	78.70
200	550	4	12	+	0	0	452.4	518.0	1	82.01	82.65	83.03	83.29	83.47	83.61
200	550	2	16	+	1	12	515.2	516.0	1	92.62	93.45	93.95	94.28	94.52	94.69
200	550	3	12	+	1	16	540.4	516.0	1	96.97	97.88	98.43	98.79	99.05	99.25
200	550	3	16	+	0	0	603.2	516.0	1	107.77	108.91	109.59	110.04	110.37	110.61
200	550	2	16	+	2	12	628.3	516.0	1	112.06	113.30	114.04	114.53	114.88	115.15
200	550	2	20	+	0	0	628.3	512.0	1	115.15	112.39	113.13	113.62	113.98	114.24
200	550	2	20	+	1	12	741.4	512.0	1	130.11	131.83	132.86	133.55	134.04	134.41
200	550	4	16	+	0	0	804.2	516.0	1	141.67	143.69	144.90	145.71	146.29	146.72
200	550	2	20	+	1	16	829.4	512.0	1	144.64	146.79	148.08	148.94	149.55	150.01
200	550	2	20	+	2	12	854.5	492.0	2	142.59	144.87	146.24	147.15	147.80	148.29
200	550	3	20	+	0	0	942.5	512.0	1	163.03	165.81	167.47	168.58	169.38	169.97
200	550	2	20	+	3	12	967.6	492.0	2	160.02	163.02	164.77	165.94	166.78	167.41
200	550	2	25	+	0	0	981.7	509.5	1	168.45	171.47	173.28	174.48	174.34	175.99
200	550	3	20	+	1	12	1030.4	492.0	2	169.67	172.99	174.99	176.31	177.26	177.97
200	550	2	25	+	1	12	1094.8	509.5	1	186.31	190.06	192.31	193.81	194.88	195.68
200	550	3	20	+	2	10	1207.9	484.0	2	–	197.51	200.25	202.07	203.38	204.36
200	600	2	20	+	1	12	270.2	568.0	1	54.47	54.69	54.83	54.92	54.99	55.04
200	600	2	12	+	1	10	304.7	568.0	1	61.30	61.59	61.77	61.88	61.96	62.03
200	600	3	12	+	0	0	339.3	568.0	1	68.11	68.47	68.68	68.83	68.93	69.01
200	600	2	12	+	2	10	383.3	568.0	1	76.72	77.18	77.46	77.64	77.77	77.87
200	600	2	16	+	0	0	402.1	566.0	1	80.11	80.62	80.92	81.12	81.27	81.38

Table F-3B *Ultimate Moment of Resistance of Flanged Section for $D_f \geq 100$ mm, Beam width = 200 mm Continued ...*

M_{ur} Flanged Sect
M 20 , Fe 415
b_w = 200 mm

b	D	N1-D1		+	N2-D2		A_{st}	d	RO	b_f / b_w					
mm	mm	mm			mm		mm^2	mm		3	4	5	6	7	8
200	600	2	12	+	1	16	427.3	566.0	1	84.99	85.56	85.90	86.13	86.29	86.41
200	600	4	12	+	0	0	452.4	568.0	1	90.17	90.81	91.19	91.45	91.63	91.77
200	600	2	16	+	1	12	515.2	566.0	1	101.92	102.75	103.24	103.58	103.81	103.99
200	600	3	12	+	1	16	540.4	566.0	1	106.72	107.63	108.18	108.54	108.80	109.00
200	600	3	16	+	0	0	603.2	566.0	1	118.65	119.79	120.47	120.93	121.25	121.50
200	600	2	16	+	2	12	628.3	566.0	1	123.40	124.63	125.37	125.87	126.22	126.48
200	600	2	20	+	0	0	628.3	562.0	1	122.49	123.73	124.47	124.96	125.31	125.58
200	600	2	20	+	1	12	741.4	562.0	1	143.49	145.21	146.24	146.93	147.42	147.79
200	600	4	16	+	0	0	804.2	566.0	1	156.18	158.20	159.42	160.22	160.80	161.24
200	600	2	20	+	1	16	829.4	562.0	1	159.60	161.75	163.04	163.90	164.52	164.98
200	600	2	20	+	2	12	854.5	542.0	2	158.00	160.29	161.66	162.57	163.22	163.71
200	600	3	20	+	0	0	942.5	562.0	1	180.03	182.81	184.48	185.59	186.38	186.98
200	600	2	20	+	3	12	967.6	542.0	2	177.35	180.47	182.23	183.40	184.24	184.87
200	600	2	25	+	0	0	981.7	559.5	1	186.17	189.18	190.99	192.19	193.06	193.70
200	600	2	20	+	2	16	1030.4	542.0	2	188.27	191.59	193.58	194.91	195.85	196.57
200	600	2	25	+	1	12	1094.8	559.5	1	206.08	209.81	212.06	213.56	214.63	215.43
200	600	2	25	+	2	12	1207.9	534.5	2	–	219.31	222.05	223.87	225.17	226.15
200	650	2	20	+	1	12	270.2	618.0	1	59.34	59.57	59.71	59.80	59.86	59.91
200	650	2	12	+	1	10	304.7	618.0	1	66.80	67.09	67.26	67.38	67.46	67.53
200	650	3	12	+	0	0	339.3	618.0	1	74.23	74.59	74.80	74.95	75.05	75.13
200	650	2	12	+	2	10	383.3	618.0	1	83.64	84.10	84.37	84.56	84.69	84.79
200	650	2	16	+	0	0	402.1	616.0	1	87.37	87.87	88.18	88.38	88.52	88.63
200	650	2	12	+	1	16	427.3	616.0	1	92.69	93.27	93.61	93.84	94.00	94.12
200	650	4	12	+	0	0	452.4	618.0	1	98.33	98.97	99.36	99.61	99.79	99.93
200	650	2	16	+	1	12	515.2	616.0	1	111.21	112.04	112.54	1112.87	113.11	113.29
200	650	3	12	+	1	16	540.4	616.0	1	116.47	117.38	117.93	118.29	118.55	118.75
200	650	3	16	+	0	0	603.2	616.0	1	129.54	130.67	131.36	131.81	132.14	132.38
200	650	2	16	+	2	12	628.3	616.0	1	134.74	135.97	136.71	137.20	13756	137.82
200	650	2	20	+	0	0	628.3	612.0	1	133.83	135.06	135.80	136.30	136.65	136.91
200	650	2	20	+	1	12	741.4	612.0	1	156.87	158.59	159.62	160.31	160.80	161.17
200	650	4	16	+	0	0	804.2	616.0	1	170.69	172.71	173.93	174.74	175.31	175.75
200	650	2	20	+	1	16	829.4	612.0	1	174.57	176.72	178.01	178.87	179.48	179.94
200	650	2	20	+	2	12	854.5	592.0	2	173.42	175.71	177.07	177.99	178.64	179.13
200	650	3	20	+	0	0	942.5	612.0	1	197.04	199.82	201.48	202.59	203.39	203.98
200	650	2	20	+	3	12	967.6	592.0	2	195.01	197.93	199.69	200.86	201.70	202.32
200	650	2	25	+	0	0	981.7	609.5	1	203.88	206.90	208.70	209.91	210.77	211.42
200	650	2	20	+	2	12	1030.4	592.0	2	206.86	241.10	243.84	245.67	246.97	247.95
200	650	2	25	+	1	12	1094.8	609.5	1	225.82	229.57	231.82	233.32	234.39	235.19
200	650	2	25	+	2	12	1207.9	584.5	2	–	241.10	243.84	245.67	246.97	247.95

Note :- Dashes indicate that for the Number - Diameter combination of bars, Width of flange is such that the *depth of the neutral axis* works out to be greater than the depth of the flange of 100 *mm*.

Table F-4 **Percentage of Steel p_t% required for given Moment of Resistance Factor, $R_u = M_u/bd^2$ -For Singly Reinforced Section – Mild Environment**

Singly Rein
R_u, p_t%
M20

$M_u/bd^2 = R_u$ N/mm²	f_y N/mm²			$M_u/bd^2 = R_u$ N/mm²	f_y N/mm²		
	250	415	500		250	415	500
0.30	0.140	0.085	0.070	2.16	1.164	0.701	0.582
0.35	0.164	0.099	0.082	2.18	1.177	0.709	0.588
0.40	0.188	0.114	0.094	2.20	1.190	0.717	0.595
0.45	0.213	0.128	0.106	2.22	1.203	0.725	0.602
0.50	0.237	0.143	0.119	2.24	1.216	0.733	0.608
0.55	0.262	0.158	0.131	2.26	1.230	0.741	0.615
0.60	0.286	0.172	0.143	2.28	1.243	0.749	0.621
0.65	0.311	0.187	0.156	2.30	1.256	0.757	0.628
0.70	0.336	0.203	0.168	2.32	1.270	0.765	0.635
0.75	0.361	0.218	0.181	2.34	1.283	0.773	0.642
0.80	0.387	0.233	0.193	2.36	1.297	0.781	0.648
0.85	0.412	0.248	0.206	2.38	1.311	0.790	0.655
0.90	0.438	0.264	0.219	2.40	1.324	0.798	0.662
0.95	0.464	0.280	0.232	2.42	1.338	0.806	0.669
1.00	0.490	0.295	0.245	2.44	1.352	0.814	0.676
1.05	0.517	0.311	0.258	2.46	1.366	0.823	0.683
1.10	0.543	0.327	0.272	2.48	1.380	0.831	0.690
1.15	0.570	0.343	0.285	2.50	1.394	0.840	0.697
1.20	0.597	0.359	0.298	2.52	1.408	0.848	0.704
1.25	0.624	0.376	0.312	2.54	1.423	0.857	0.711
1.30	0.651	0.392	0.326	2.56	1.437	0.866	0.719
1.35	0.679	0.409	0.339	2.58	1.451	0.874	0.726
1.40	0.707	0.426	0.353	2.60	1.466	0.883	0.733
1.45	0.735	0.443	0.367	262	1.481	0.892	0.740
1.50	0.763	0.460	0.382	2.64	1.495	0.901	0.748
1.55	0.792	0.477	0.396	2.66	1.510	0.910	*0.755
1.60	0.821	0.494	0.410	2.68	1.525	0.919	–
1.65	0.850	0.512	0.425	2.70	1.540	0.928	–
1.70	0.879	0.530	0.440	2.72	1.555	0.937	–
1.75	0.909	0.547	0.454	2.74	1.570	0.946	–
1.80	0.939	0.565	0.469	2.76	1.585	*0.955	–
1.85	0.969	0.584	0.484	2.78	1.601	–	–
1.90	1.000	0.602	0.500	2.80	1.616	–	–
1.95	1.030	0.621	0.515	2.82	1.632	–	–
2.00	1.062	0.640	0.531	2.84	1.647	–	–
2.02	1.074	0.647	0.537	2.86	1.663	–	–
2.04	1.087	0.655	0.543	2.88	1.679	–	–
2.06	1.099	0.662	0.550	2.90	1.695	–	–
2.08	1.112	0.670	0.556	2.92	1.711	–	–
2.10	1.125	0.678	0.562	2.94	1.727	–	–
2.12	1.138	0.685	0.569	2.96	1.743	–	–
2.14	1.151	0.693	0.575	2.98	*1.760	–	–

**Last values represent the percentage of steel for balanced section. Thereafter the section becomes over reinforced hence the values of percentage of reinforcement are inadmissible which are shown by dashes.*

Table F-5 **Percentage of Steel (p_t% and p_c%) required for given Moment of Resistance Factor, $R_u = M_u/bd^2$ - For Doubly Reinforced Section – Mild Environment**

Doubly Rein
R_u, p_t%, p_c%
M 20, Fe 415

$M_u/bd^2 = R_u$ N/mm²	d'/d = 0.05		d'/d = 0.10		d'/d = 0.15		d'/d = 0.20	
	p_t%	p_c%	p_t%	p_c%	p_t%	p_c%	p_t%	p_c%
2.77	0.958	0.002	0.958	0.002	0.959	0.003	0.959	0.003
2.80	0.967	0.011	0.968	0.012	0.968	0.013	0.969	0.015
2.90	0.996	0.042	0.998	0.045	1.001	0.049	1.004	0.054
3.00	1.025	0.072	1.029	0.077	1.034	0.084	1.038	0.093
3.10	1.055	0.103	1.060	0.109	1.066	0.119	1.073	0.132
3.20	1.084	0.133	1.091	0.142	1.099	0.154	1.108	0.171
3.30	1.113	0.164	1.122	0.174	1.131	0.190	1.142	0.210
3.40	1.142	0.194	1.152	0.207	1.164	0.225	1.177	0.249
3.50	1.171	0.224	1.183	0.239	1.197	0.260	1.212	0.288
3.60	1.200	0.255	1.214	0.271	1.229	0.295	1.246	0.327
3.70	1.230	0.285	1.245	0.304	1.262	0.331	1.281	0.366
3.80	1.259	0.316	1.276	0.336	1.294	0.366	1.315	0.405
3.90	1.288	0.346	1.306	0.369	1.327	0.401	1.350	0.444
4.00	1.317	0.376	1.337	0.401	1.360	0.437	1.385	0.483
4.10	1.346	0.407	1.368	0.433	1.392	0.472	1.419	0.522
4.20	1.375	0.437	1.399	0.466	1.425	0.507	1.454	0.561
4.30	1.405	0.468	1.429	0.498	1.457	0.542	1.489	0.600
4.40	1.434	0.498	1.460	0.530	1.490	0.578	1.523	0.640
4.50	1.463	0.528	1.491	0.563	1.523	0.613	1.558	0.679
4.60	1.492	0.559	1.522	0.595	1.555	0.648	1.593	0.718
4.70	1.521	0.589	1.553	0.628	1.588	0.683	1.627	0.757
4.80	1.550	0.620	1.583	0.660	1.620	0.719	1.662	0.796
4.90	1.580	0.650	1.614	0.692	1.653	0.754	1.696	0.835
5.00	1.609	0.680	1.645	0.725	1.686	0.789	1.731	0.874
5.10	1.638	0.711	1.676	0.757	1.718	0.825	1.766	0.913
5.20	1.667	0.741	1.707	0.790	1.751	0.860	1.800	0.952
5.30	1.696	0.772	1.737	0.822	1.783	0.895	1.835	0.991
5.40	1.725	0.802	1.768	0.854	1.816	0.930	1.870	1.030
5.50	1.755	0.832	1.799	0.887	1.849	0.966	1.904	1.069
5.60	1.784	0.863	1.830	0.919	1.881	1.001	1.939	1.108
5.70	1.813	0.893	1.861	0.952	1.914	1.036	1.974	1.147
5.80	1.842	0.924	1.891	0.984	1.946	1.071	2.008	1.186
5.90	1.871	0.954	1.922	1.016	1.979	1.107	2.043	1.225
6.00	1.900	0.985	1.953	1.049	2.012	1.142	2.078	1.264
6.10	1.930	1.015	1.984	1.081	2.044	1.177	2.112	1.303
6.20	1.959	1.045	2.014	1.114	2.077	1.213	2.147	1.342
6.30	1.988	1.076	2.045	1.146	2.109	1.248	2.181	1.381
6.40	2.017	1.106	2.076	1.178	2.142	1.283	2.216	1.421
6.50	2.046	1.137	2.107	1.211	2.175	1.318	2.251	1.460
6.60	2.075	1.167	2.138	1.243	2.207	1.354	2.285	1.499
6.70	2.105	1.197	2.168	1.276	2.240	1.389	2.320	1.538
6.80	2.134	1.228	2.199	1.308	2.272	1.424	2.355	1.577
6.90	2.163	1.258	2.230	1.340	2.305	1.459	2.389	1.616
7.00	2.192	1.289	2.261	1.373	2.338	1.495	2.424	1.655
7.10	2.221	1.319	2.292	1.405	2.370	1.530	2.459	1.694

Table F-6 **Minimum Shear Reinforcemant using 2-Legged Stirrups in Beams of Different Widths, Corresponding to Bar diameter and Width of Beam**

Grade of steel = Fe 250

Values of Spacing in mm

Dia. in mm	*Area of 2-Legs mm^2*	*Width of Beam in mm*									
		150	200	230	250	300	350	380	400	450	500
ϕ6	56.5	200	150	130	120	100	80	80	0	0	0
ϕ8	100.5	300	270	230	210	180	150	140	130	120	100
ϕ10	157.1	300	300	300	300	280	240	220	210	180	170
ϕ12	226.2	300	300	300	300	300	300	300	300	270	240
ϕ16	402.1	300	300	300	300	300	300	300	300	300	300

Grade of steel = Fe 415 and Fe 500

Values of Spacing in mm

Dia. in mm	*Area of 2-Legs mm^2*	*Width of Beam in mm*									
		150	200	230	250	300	350	380	400	450	500
#6	56.5	300	250	220	200	170	140	130	120	110	100
#8	100.5	300	300	300	300	300	250	230	220	200	180
#10	157.1	300	300	300	300	300	300	300	300	300	280
#12	226.2	300	300	300	300	300	300	300	300	300	300
#16	402.1	300	300	300	300	300	300	300	300	300	300

Note : 1. *#6 mm bars are not available in Market at present*
2. *Maximum spacing shall also not exceed 0.75 d. where, d = effective depth of beam.*

Table F-7 **Shear Strength of 2-legged Vertical Stirrups – Values of V_{usv}/d in kN/m or N/mm corresponding to spacing of stirrups**

Shear Strength of 2-legged Vertical Stirrups										*Values of V_{usv}/d in kN/m or N/mm*
Spacing s mm	Fe 415					Fe 250				
	Diameter in mm					Diameter in mm				
	#6*	#8	#10	#12	#16	φ6	φ8	φ10	φ12	φ16
80	255	453	709	1020	1814	154	273	427	615	1093
90	227	403	630	907	1612	137	243	379	546	971
100	204	363	567	816	1451	123	219	341	492	874
110	186	330	515	742	1319	112	199	310	447	795
120	170	302	472	680	1209	102	182	285	410	728
130	157	279	436	628	1116	95	168	263	378	672
140	146	259	405	583	1037	88	156	244	351	624
150	136	242	378	544	967	82	146	228	328	583
160	128	227	354	510	907	77	137	213	307	546
170	120	213	333	480	854	72	129	201	289	514
180	113	202	315	453	806	68	121	190	273	486
190	107	191	298	430	764	65	115	180	259	460
200	102	181	283	408	726	61	109	171	246	437
210	97	173	270	389	691	59	104	163	234	416
220	93	165	258	371	660	56	99	155	224	397
230	89	158	246	355	631	53	95	148	214	380
240	85	151	236	340	605	51	91	142	205	364
250	82	145	227	327	580	49	87	137	197	350
260	78	140	218	314	558	47	84	131	189	336
270	76	134	210	302	537	46	81	126	182	324
280	73	130	202	292	518	44	78	122	176	312
290	70	125	195	281	500	42	75	118	170	301
300	68	121	189	272	484	41	73	114	164	291

Note : **#6 mm bars of Fe415 are not available in market at present.*

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APPENDIX - G TABLES AND CHARTS FOR COLUMN

Table G-1 Load Carrying Capacity of Short Column (minimum eccentricity 20*mm*)
Concrete M20, Steel Fe 415

[A] For $b = 200$ *mm, Reduction Factor = 0.8 is used*

b x D (mm mm)	N - #	4-12	6-12	4-16	8-12	4-16 +2-12	6-16	4-16 +4-12	6-16 +2-12	8-16	8-16 +2-12	10-16
	A_{sc} mm^2	452	678	804	904	1030	1206	1256	1432	1608	1834	2010
200 x 200		354	403	430	451	479	517	527	565	603	652	690
200 x 230		392	441	468	490	517	555	566	604	642	691	729
200 x 300		482	531	558	579	607	645	655	693	731	780	818
200 x 350		546	595	622	643	671	709	719	757	795	844	882
200 x 380		584	633	660	682	709	747	758	796	834	883	921
200 x 400		610	659	686	707	735	773	783	821	859	908	946
200 x 450		674	723	750	771	799	837	847	885	923	972	1010
200 x 500		738	787	814	835	863	901	911	949	987	1036	1074
200 x 530		776	825	852	874	901	939	950	988	1026	1075	1113
200 x 550		802	851	878	899	927	965	975	1013	1051	1100	1138
200 x 600		866	915	942	963	991	1029	1039	1077	1115	1164	1202
200 x 650		930	979	1006	1027	1055	1093	1103	1141	1179	1228	1266
200 x 700		894	1030	1070	1091	1119	1157	1167	1205	1243	1292	1330

[B] For $b = 230$ *mm, Reduction factor = 0.9 is used*

b x D (mm mm)	N - #	4-12	6-12	4-16	8-12	4-16 +2-12	6-16	4-16 +4-12	6-16 +2-12	8-16	8-16 +2-12	10-16
	A_{sc} mm^2	452	678	804	904	1030	1206	1256	1432	1608	1834	2010
230 x 230		491	546	576	601	631	674	686	729	772	827	870
230 x 300		607	662	692	717	747	790	802	845	888	943	985
230 x 350		690	745	775	800	830	873	885	928	971	1026	1068
230 x 380		739	794	825	849	880	922	935	977	1020	107	1118
230 x 400		772	827	858	882	913	956	968	1011	1053	1108	1151
230 x 450		855	910	948	965	996	1038	1051	1093	1136	1191	1233
230 x 500		938	993	1023	1048	1078	1121	1133	1176	1219	1274	1317
230 x 530		988	1043	1073	1098	1128	1171	1183	1226	1269	1324	1366
230 x 550		1021	1076	1106	1131	1161	1204	1216	1259	1302	1357	1399
230 x 600		1104	1159	1189	1214	1244	1287	1299	1342	1385	1440	1482
230 x 650		1186	1241	1272	1296	1327	1370	1382	1425	1467	1522	1565
230 x 700		1269	1324	1355	1379	1410	1452	1465	1507	1550	1605	1648

Note : *1) Values below zig-zag lines are for area of steel less than minimum percentage of 0.8 % of b D*
2) If the columns has larger cross-sectional area than that required to support the load the minimum percentage shall be based upon the concrete area required to resist the direct stress and not upon the actual area.

Table G-2a **Allowable Combinations of Ultimate axial Compression P_u in *kN* and Ultimate Moment about major axis of Bending, M_{ux} in *kN.m* for Rectangular Columns - *200 mm* wide × Depth 'D' Concrete M20, Steel Fe 415**

P_u - M_u
M 20, Fe 415
b = 200 mm

Depth D (mm)	Steel N1 D1 + N2 D2 (mm)	Area (mm²)	0.4 P_u	0.4 M_u	0.5 P_u	0.5 M_u	0.6 P_u	0.6 M_u	0.7 P_u	0.7 M_u	0.8 P_u	0.8 M_u	0.9 P_u	0.9 M_u
			Neutral Axis Factor $k_u = x_u/D$											
200	4 #12 + 0 #0	452	89	14.1	142	15.1	208	13.9	267	12.4	318	10.8	363	8.9
	4 #12 + 2 #12	679	50	14.1	142	15.1	234	13.9	311	12.4	375	10.8	429	8.9
	6 #12 + 0 #0	679	76	17.3	141	18.4	226	16.5	299	14.4	361	12.3	415	10.0
	4 #16 + 0 #0	804	62	18.3	141	19.5	239	17.3	321	15.0	388	12.8	446	10.4
	4 #12 + 4 #12	905	13	15.1	141	15.9	259	14.6	355	13.0	427	11.2	488	9.1
	6 #12 + 2 #12	905	36	17.3	141	18.4	251	16.5	343	14.4	418	12.3	480	10.0
	4 #16 + 2 #12	1030	23	18.3	141	19.5	265	17.3	364	15.0	445	12.8	512	10.4
	6 #12 + 4 #12	1131	-0	18.3	140	19.2	277	17.2	387	15.0	471	12.7	540	10.3
230	4 #12 + 0 #0	452	117	19.4	164	20.3	227	19.0	291	17.0	347	14.7	397	12.1
	4 #12 + 2 #12	679	78	19.4	164	20.3	252	19.0	335	17.0	404	14.7	463	12.1
	6 #12 + 0 #0	679	110	24.0	163	24.8	241	22.8	320	19.8	387	16.9	446	13.9
	4 #16 + 0 #0	804	103	25.8	162	26.7	252	24.1	339	20.9	412	17.7	476	14.5
	4 #12 + 4 #12	905	44	20.8	163	21.5	278	20.0	378	17.8	455	15.3	521	12.5
	6 #12 + 2 #12	905	70	24.0	163	24.8	266	22.8	364	19.8	445	16.9	512	13.9
	4 #16 + 2 #12	1030	63	25.8	162	26.7	277	24.1	383	20.9	470	17.7	541	14.5
	6 #12 + 4 #12	1131	36	25.4	162	26.0	292	23.8	408	20.7	496	17.5	570	14.3
	6 #16 + 0 #0	1206	88	33.6	161	34.4	278	30.5	393	25.7	486	21.5	564	17.4
	4 #16 + 2 #16	1206	32	25.8	162	26.7	297	24.1	417	20.9	515	17.7	593	14.5
	4 #16 + 4 #12	1257	28	27.1	161	27.8	303	25.1	427	21.7	521	18.3	600	14.8
	4 #20 + 0 #0	1257	77	33.2	161	34.3	287	30.2	404	25.5	499	21.3	578	17.2
300	4 #12 + 2 #12	679	124	32.4	215	33.8	301	32.8	395	29.3	476	25.5	546	21.1
	6 #12 + 0 #0	679	159	39.9	214	41.2	284	39.5	375	34.5	455	29.5	525	24.3
	4 #16 + 0 #0	804	156	43.4	213	44.6	291	42.4	391	36.7	477	31.2	552	25.6
	4 #12 + 4 #12	905	95	34.9	213	36.0	327	34.7	437	30.8	525	26.6	601	21.8
	6 #12 + 2 #12	905	119	39.9	214	41.2	310	39.5	419	34.5	512	29.5	591	24.3
	4 #16 + 2 #12	1030	116	43.4	213	44.6	317	42.4	435	36.7	535	31.2	618	25.6
	6 #12 + 4 #12	1131	90	42.4	212	43.4	335	41.3	461	36.0	561	30.6	646	25.0
	6 #16 + 0 #0	1206	147	56.5	211	57.4	307	53.8	435	45.6	542	38.2	633	31.1
	4 #16 + 2 #16	1206	85	43.4	213	44.6	337	42.4	469	36.7	579	31.2	669	25.6
	4 #16 + 4 #12	1257	86	45.8	212	46.7	342	44.1	477	38.1	583	32.3	674	26.3

For details of arrangement of longitudinal Reinforcement see NOTES at the end of *Table G-2A*.

Table G-2a **Allowable Combinations of Ultimate axial Compression P_u in *kN* and Ultimate Moment about major axis of Bending, M_{ux} in *kN.m* for Rectangular Columns - *200 mm* wide × Depth 'D' Concrete M20, Steel Fe 415**

P_u-M_u
M 20, Fe 415
b = 200 mm

Depth D mm	Steel N1 D1 + N2 D2 (mm, mm)	Area mm²	1.00 P_u	1.00 M_u	1.05 P_u	1.05 M_u	1.10 P_u	1.10 M_u	1.15 P_u	1.15 M_u	1.20 P_u	1.20 M_u	1.30 P_u	1.30 M_u
			Neutral Axis Factor $k_u = x_u/D$											
200	4 #12 + 0 #0	452	405	6.6	419	5.7	431	4.9	440	4.3	447	3.8	454	3.4
	4 #12 + 2 #12	679	474	6.6	489	5.7	500	4.9	510	4.3	517	3.8	524	3.4
	6 #12 + 0 #0	679	463	7.5	479	6.5	492	5.6	503	4.9	511	4.3	519	3.9
	4 #16 + 0 #0	804	498	7.7	515	6.6	528	5.8	539	5.1	549	4.5	557	4.0
	4 #12 + 4 #12	905	541	6.8	557	5.8	569	5.0	578	4.4	587	3.9	594	3.4
	6 #12 + 2 #12	905	532	7.5	549	6.5	562	5.6	572	4.9	581	4.3	589	3.9
	4 #16 + 2 #12	1030	567	7.7	584	6.6	598	5.8	609	5.1	619	4.5	627	4.0
	6 #12 + 4 #12	1131	600	7.6	616	6.6	630	5.7	641	5.0	651	4.4	659	3.9
230	4 #12 + 0 #0	452	444	9.1	460	7.8	473	6.8	483	6.0	492	5.3	499	4.7
	4 #12 + 2 #12	679	513	9.1	529	7.8	542	6.8	553	6.0	562	5.3	569	4.7
	6 #12 + 0 #0	679	500	10.5	518	9.0	532	7.9	544	6.9	554	6.2	563	5.5
	4 #16 + 0 #0	804	533	10.9	552	9.4	567	8.2	580	7.2	590	6.4	599	5.7
	4 #12 + 4 #12	905	579	9.3	597	8.0	610	7.0	622	6.1	631	5.4	639	4.8
	6 #12 + 2 #12	905	569	10.5	587	9.0	602	7.9	614	6.9	624	6.2	633	5.5
	4 #16 + 2 #12	1030	602	10.9	621	9.4	637	8.2	650	7.2	660	6.4	669	5.7
	6 #12 + 4 #12	1131	635	10.7	655	9.2	670	8.0	683	7.1	693	6.3	702	5.6
	6 #16 + 0 #0	1206	633	13.1	656	11.4	674	10.0	689	8.8	702	7.9	713	7.0
	4 #16 + 2 #16	1206	656	10.9	675	9.4	691	8.2	704	7.2	715	6.4	724	5.7
	4 #16 + 4 #12	1257	669	11.1	689	9.6	705	8.4	718	7.4	729	6.5	739	5.8
	4 #20 + 0 #0	1257	649	12.9	672	11.2	690	9.8	705	8.7	718	7.7	729	6.9
300	4 #12 + 2 #12	679	607	16.0	627	13.8	644	12.0	657	10.6	669	9.4	678	8.4
	6 #12 + 0 #0	679	590	18.5	613	16.0	631	14.1	646	12.5	658	11.1	669	9.9
	4 #16 + 0 #0	804	620	19.5	644	17.0	664	14.9	679	13.2	693	11.8	704	10.6
	4 #12 + 4 #12	905	671	16.4	694	14.1	711	12.3	725	10.9	737	9.6	747	8.6
	6 #12 + 2 #12	905	659	18.5	682	16.0	701	14.1	716	12.5	728	11.1	739	9.9
	4 #16 + 2 #12	1030	689	19.5	714	17.0	733	14.9	749	13.2	763	11.8	774	10.6
	6 #12 + 4 #12	1131	723	18.9	748	16.4	768	14.4	783	12.7	796	11.3	807	10.2
	6 #16 + 0 #0	1206	714	23.8	742	20.8	765	18.4	784	16.4	799	14.6	812	13.2
	4 #16 + 2 #16	1206	743	19.5	768	17.0	787	14.9	804	13.2	817	11.8	828	10.6
	4 #16 + 4 #12	1257	754	19.9	780	17.3	801	15.2	817	13.5	831	12.0	842	10.8

For details of arrangement of longitudinal Reinforcement see NOTES at the end of *Table G-2A*.

Table G-2a **Allowable Combinations of Ultimate axial Compression P_u in *kN* and Ultimate Moment M_u in *kN* continued . . .**

P_u - M_u
M 20, Fe 415
b = 200 mm

Depth D mm	Steel N1 D1 + N2 D2 mm mm	Area mm²	0.4 P_u	0.4 M_u	0.5 P_u	0.5 M_u	0.6 P_u	0.6 M_u	0.7 P_u	0.7 M_u	0.8 P_u	0.8 M_u	0.9 P_u	0.9 M_u
			Neutral Axis Factor $k_u = x_u/D$											
300	4 #20 + 0 #0	1257	144	57.0	211	58.0	313	53.9	444	45.6	553	38.2	646	31.1
	6 #16 + 2 #12	1433	107	56.5	211	57.4	332	53.8	479	45.6	600	38.2	699	31.1
	4 #20 + 2 #12	1483	104	57.0	211	58.0	339	53.9	488	45.6	611	38.2	712	31.1
	4 #16 + 4 #16	1608	32	47.7	211	48.4	382	45.5	543	39.3	666	33.1	768	26.9
	4 #20 + 2 #16	1659	74	57.0	211	58.0	358	53.9	522	45.6	656	38.2	763	31.1
	4 #20 + 4 #12	1709	74	59.3	210	60.0	364	55.6	530	47.0	660	39.2	768	31.8
350	4 #12 + 2 #12	679	155	42.7	251	44.7	340	44.1	441	39.6	529	34.5	607	28.6
	6 #12 + 0 #0	679	190	52.3	250	54.2	320	52.9	419	46.5	506	39.9	584	32.9
	4 #16 + 0 #0	804	187	56.9	249	58.8	324	57.0	433	49.6	526	42.3	609	34.8
	4 #12 + 4 #12	905	128	46.0	249	47.7	365	46.6	482	41.6	577	35.9	661	29.6
	6 #12 + 2 #12	905	151	52.3	250	54.2	345	52.9	462	46.5	563	39.9	650	32.9
	4 #16 + 2 #12	1030	148	56.9	249	58.8	350	57.0	476	49.6	584	42.3	675	34.8
	6 #12 + 4 #12	1131	124	55.5	248	57.2	371	55.4	503	48.5	611	41.3	704	33.8
	6 #16 + 0 #0	1206	180	73.6	247	75.3	335	72.2	472	61.5	587	51.6	687	42.2
	4 #16 + 2 #16	1206	117	56.9	249	58.8	369	57.0	510	49.6	629	42.3	727	34.8
	4 #16 + 4 #12	1257	120	60.1	248	61.7	375	59.4	517	51.5	632	43.7	730	35.7
	4 #20 + 0 #0	1257	178	74.7	247	76.3	340	72.8	480	61.8	598	51.9	699	42.4
	6 #16 + 2 #12	1433	141	73.6	247	75.3	360	72.2	516	61.5	645	51.6	753	42.2
	4 #20 + 2 #12	1483	138	74.7	247	76.3	365	72.8	524	61.8	656	51.9	765	42.4
	4 #16 + 4 #16	1608	68	62.6	247	63.9	415	61.2	583	53.0	714	44.8	824	36.5
	4 #20 + 2 #16	1659	108	74.7	247	76.3	385	72.8	558	61.8	700	51.9	816	42.4
	4 #20 + 4 #12	1709	110	77.8	246	79.1	391	75.1	565	63.7	704	53.3	820	43.3
	4 #25 + 0 #0	1963	161	99.9	244	100.9	373	94.1	562	78.3	717	64.8	845	52.5
	4 #20 + 4 #16	2061	58	80.2	245	81.2	430	76.9	631	65.2	786	54.4	914	44.0
380	4 #12 + 2 #12	679	173	49.4	272	51.8	365	51.3	469	46.3	562	40.4	644	33.5
	6 #12 + 0 #0	679	209	60.2	271	62.4	344	61.2	445	54.2	537	46.6	620	38.4
	4 #16 + 0 #0	804	206	65.5	271	67.7	348	66.1	459	57.9	557	49.5	645	40.7
	4 #12 + 4 #12	905	147	53.1	271	55.2	390	54.1	509	48.6	609	42.0	698	34.6
	6 #12 + 2 #12	905	169	60.2	271	62.4	369	61.2	489	54.2	595	46.6	686	38.4
	4 #16 + 2 #12	1030	167	65.5	271	67.7	373	66.1	502	57.9	614	49.5	711	40.7

For details of arrangement of longitudinal Reinforcement see NOTES at the end of *Table G-2A*.

Table G-2a **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_{ux} in kN continued . . .**

P_u - M_u
M 20, Fe 415
b = 200 mm

Depth D mm	Steel N1 D1 + N2 D2 (mm mm)	Area mm²	1.00 P_u	1.00 M_u	1.05 P_u	1.05 M_u	1.10 P_u	1.10 M_u	1.15 P_u	1.15 M_u	1.20 P_u	1.20 M_u	1.30 P_u	1.30 M_u
			Neutral Axis Factor $k_u = x_u/D$											
300	4 #20 + 0 #0	1257	728	23.8	757	20.8	780	18.4	799	16.3	814	14.6	828	13.1
	6 #16 + 2 #12	1433	783	23.8	811	20.8	835	18.4	853	16.4	869	14.6	883	13.2
	4 #20 + 2 #12	1483	797	23.8	826	20.8	849	18.4	868	16.3	884	14.6	898	13.1
	4 #16 + 4 #16	1608	858	20.2	886	17.6	907	15.4	924	13.7	938	12.2	950	10.9
	4 #20 + 2 #16	1659	851	23.8	880	20.8	903	18.4	923	16.3	939	14.6	952	13.1
	4 #20 + 4 #12	1709	862	24.2	893	21.1	917	18.6	936	16.6	953	14.8	967	13.3
350	4 #12 + 2 #12	679	675	21.7	699	18.7	718	16.4	733	14.4	746	12.8	757	11.5
	6 #12 + 0 #0	679	656	25.1	682	21.8	703	19.1	720	17.0	734	15.1	746	13.6
	4 #16 + 0 #0	804	686	26.6	713	23.1	735	20.4	753	18.1	768	16.1	780	14.5
	4 #12 + 4 #12	905	738	22.3	764	19.2	785	16.8	801	14.8	814	13.2	825	11.8
	6 #12 + 2 #12	905	725	25.1	752	21.8	773	19.1	790	17.0	804	15.1	816	13.6
	4 #16 + 2 #12	1030	755	26.6	782	23.1	805	20.4	823	18.1	838	16.1	850	14.5
	6 #12 + 4 #12	1131	789	25.7	817	22.3	840	19.6	858	17.3	872	15.5	885	13.9
	6 #16 + 0 #0	1206	776	32.4	808	28.4	833	25.2	855	22.5	872	20.1	887	18.2
	4 #16 + 2 #16	1206	809	26.6	836	23.1	859	20.4	877	18.4	892	26.1	905	14.5
	4 #16 + 4 #12	1257	818	27.2	848	23.6	872	20.8	891	18.4	906	16.5	919	14.8
	4 #20 + 0 #0	1257	790	32.6	822	28.5	848	25.3	869	22.5	887	20.2	902	18.2
	6 #16 + 2 #12	1433	845	32.4	877	28.4	903	25.2	925	22.5	942	20.1	957	18.2
	4 #20 + 2 #12	1483	859	32.6	891	28.5	917	25.3	939	22.5	957	20.2	972	18.2
	4 #16 + 4 #16	1608	921	27.6	953	24.0	978	21.1	997	18.7	1013	16.7	1027	15.0
	4 #20 + 2 #16	1659	912	32.6	945	28.5	971	25.3	993	22.5	1011	20.2	1027	18.2
	4 #20 + 4 #12	1709	922	33.1	957	29.0	984	25.6	1007	22.9	1025	20.5	1040	18.5
	4 #25 + 0 #0	1963	957	40.4	997	35.6	1029	31.7	1055	28.3	1077	25.4	1095	23.0
	4 #20 + 4 #16	2061	1026	33.5	1062	29.4	1091	25.9	1114	23.1	1132	20.8	1148	18.7
380	4 #12 + 2 #12	679	717	25.4	742	22.0	763	19.2	779	16.9	793	15.1	804	13.5
	6 #12 + 0 #0	679	697	29.4	725	25.5	747	22.4	765	19.9	781	17.8	793	16.0
	4 #16 + 0 #0	804	726	31.2	755	27.1	778	23.9	798	21.2	814	19.0	827	17.1
	4 #12 + 4 #12	905	779	26.1	807	22.5	829	19.7	846	17.4	861	15.4	872	13.8
	6 #12 + 2 #12	905	776	29.4	794	25.5	817	22.4	835	19.9	851	17.8	863	16.0
	4 #16 + 2 #12	1030	795	31.2	824	27.1	848	23.9	868	21.2	884	19.0	897	17.1

For details of arrangement of longitudinal Reinforcement see NOTES at the end of *Table G-2A*.

Table G-2a **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u in kN continued . . .**

P_u - M_u
M 20, Fe 415
b = 200 mm

Depth	*Steel*			*Neutral Axis Factor $k_u = x_u/D$*											
D	*N1 D1*	*+ N1 D2*	*Area*	0.4		0.5		0.6		0.7		0.8		0.9	
mm	*mm*	*mm*	*mm²*	P_u	M_u	P_u	M_u	P_u	M_u	P_u	M_u	P_u	M_u	P_u	M_u
380	6 #12	+ 0 #0	1131	143	63.9	270	65.9	395	64.1	529	56.5	641	48.3	740	39.6
	6 #16	+ 0 #0	1206	200	84.4	269	86.4	357	83.4	496	71.6	616	60.3	721	49.3
	4 #16	+ 2 #16	1206	136	65.5	270	67.7	393	66.1	536	57.9	659	49.5	762	40.7
	4 #16	+ 4 #12	1257	140	69.2	269	71.1	399	68.8	543	60.1	662	51.1	765	41.8
	4 #20	+ 0 #0	1257	198	85.7	269	87.7	360	84.5	504	72.2	626	60.7	732	49.6
	6 #16	+ 2 #12	1433	160	84.4	269	86.4	382	83.4	540	71.6	673	60.3	786	49.3
	4 #20	+ 2 #12	1483	158	85.7	269	87.7	385	84.5	547	72.2	684	60.7	798	49.6
	4 #16	+ 4 #16	1608	89	72.0	268	73.7	438	71.0	608	61.8	743	52.4	858	42.7
	4 #20	+ 2 #16	1659	128	85.7	269	87.7	405	84.5	581	72.2	728	60.7	849	49.6
	4 #20	+ 4 #12	1709	131	89.3	267	90.9	411	87.2	588	74.4	731	62.3	853	50.7
	4 #25	+ 0 #0	1963	182	114.7	266	116.2	385	110.0	581	91.8	741	76.1	875	61.7
	4 #20	+ 4 #16	2061	80	92.0	266	93.5	450	89.3	654	76.1	813	63.5	946	51.5
	4 #25	+ 2 #12	2190	142	114.7	266	116.2	411	110.0	625	91.8	799	76.1	941	61.7
400	4 #12	+ 2 #12	679	185	54.0	287	56.7	381	56.3	487	51.0	584	44.5	669	36.9
	6 #12	+ 0 #0	679	221	65.6	286	68.2	360	67.0	463	59.6	558	51.3	644	42.3
	4 #16	+ 0 #0	804	219	71.4	285	73.9	364	72.3	476	63.6	578	54.5	669	44.8
	4 #12	+ 4 #12	905	160	58.1	285	60.5	407	59.4	527	53.5	630	46.3	722	38.2
	6 #12	+ 2 #12	905	182	65.6	286	68.2	386	67.0	507	59.6	616	51.3	710	42.3
	4 #16	+ 2 #12	1030	179	71.4	285	73.9	389	72.3	520	63.6	635	54.5	735	44.8
	6 #12	+ 4 #12	1131	156	69.7	284	72.0	411	70.2	547	62.1	662	53.1	764	43.6
	6 #16	+ 0 #0	1206	213	91.7	283	94.0	373	91.1	512	78.5	635	66.2	744	54.2
	4 #16	+ 2 #16	1206	148	71.4	285	73.6	409	72.3	554	63.6	680	54.5	786	44.8
	4 #16	+ 2 #12	1257	153	75.4	284	77.6	415	75.3	560	66.1	682	56.2	789	46.0
	4 #20	+ 0 #0	1257	211	93.2	283	95.5	375	92.3	519	79.3	645	66.8	755	54.6
	6 #16	+ 2 #12	1433	173	91.7	283	94.0	398	91.1	556	78.5	693	66.2	809	54.2
	4 #20	+ 2 #12	1483	171	93.2	283	95.5	400	92.3	563	79.3	703	66.8	821	54.6
	4 #16	+ 4 #16	1608	103	78.5	283	80.4	455	77.7	625	67.9	763	57.6	882	47.0
	4 #20	+ 2 #16	1659	141	93.2	283	95.5	420	92.3	597	79.3	748	66.8	872	54.6
	4 #20	+ 4 #12	1709	145	97.1	282	99.1	426	95.3	604	81.7	750	68.5	875	55.8
	4 #25	+ 0 #0	1963	196	124.8	280	126.6	395	120.8	594	101.0	757	83.8	895	68.0

For details of arrangement of longitudinal Reinforcement see NOTES at the end of *Table G-2A*.

Table G-2a **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u in kN continued . . .**

P_u - M_u
M 20, Fe 415
b = 200 mm

Depth D mm	Steel N1 D1 + N2 D2 mm mm	Area mm²	Neutral Axis Factor $k_u = x_u/D$ 1.00 P_u	1.00 M_u	1.05 P_u	1.05 M_u	1.10 P_u	1.10 M_u	1.15 P_u	1.15 M_u	1.20 P_u	1.20 M_u	1.30 P_u	1.30 M_u
380	6 #12 + 4 #12	1131	829	30.1	859	26.1	883	22.9	903	20.3	918	18.2	932	16.3
	6 #16 + 2 #10	1206	814	38.0	848	33.3	876	29.5	898	26.3	917	23.7	933	21.3
	4 #16 + 0 #16	1206	849	31.2	878	27.1	902	23.9	922	21.2	938	19.0	952	17.1
	4 #16 + 4 #12	1257	858	31.8	889	27.7	915	24.4	935	21.6	952	19.4	965	17.4
	4 #20 + 0 #0	1257	828	38.2	862	33.5	889	29.7	912	26.5	931	23.8	947	21.4
	6 #16 + 2 #12	1433	884	38.0	918	33.3	945	29.5	968	26.3	987	23.7	1003	21.3
	4 #20 + 2 #12	1483	897	38.2	931	33.5	959	29.7	982	26.5	1001	23.8	1017	21.4
	4 #16 + 4 #16	1608	961	32.4	994	28.1	1021	24.7	1042	22.0	1059	19.7	1073	17.7
	4 #20 + 4 #16	1659	951	38.2	985	33.15	1013	29.7	1039	26.5	1056	23.8	1072	21.4
	4 #20 + 4 #12	1709	960	38.9	996	34.0	1026	30.1	1050	26.9	1069	24.1	1086	21.8
	4 #25 + 0 #0	1963	992	44.7	1034	42.0	1068	37.4	1096	33.5	1119	30.1	1139	27.2
	4 #20 + 4 #16	2061	1063	39.4	1101	34.5	1132	30.5	1156	27.2	1177	24.4	1193	22.0
	4 #25 + 2 #12	2190	1061	47.7	1103	42.0	1137	37.4	1165	33.5	1189	30.1	1209	27.2
400	4 #12 + 2 #12	679	745	28.0	771	24.2	792	21.2	810	18.7	824	16.6	836	14.9
	6 #12 + 0 #0	679	724	32.4	753	28.1	777	24.7	796	21.9	811	19.6	825	17.6
	4 #16 + 0 #0	804	753	34.3	783	29.9	808	26.3	828	23.4	844	20.9	858	18.8
	4 #12 + 4 #12	905	807	28.8	836	24.9	859	21.7	877	19.1	892	17.0	904	15.3
	6 #12 + 2 #12	905	794	32.4	823	28.1	846	24.7	865	21.9	881	19.6	895	17.6
	4 #16 + 2 #12	1030	822	34.3	853	29.9	877	26.3	897	23.4	914	20.9	928	18.8
	6 #12 + 4 #12	1131	856	33.1	887	28.8	912	25.3	933	22.4	949	20.0	963	18.0
	6 #16 + 0 #0	1206	841	41.8	876	36.7	904	32.5	927	29.0	947	26.1	963	23.5
	4 #16 + 2 #16	1206	876	34.3	907	29.9	931	26.3	952	23.4	969	20.9	983	18.8
	4 #16 + 4 #12	1257	885	35.1	917	30.5	944	26.9	965	23.9	982	21.4	996	19.2
	4 #20 + 0 #0	1257	854	42.1	889	36.9	918	32.7	941	29.2	961	26.3	978	23.7
	6 #16 + 2 #12	1433	910	41.8	945	36.7	974	32.5	997	29.0	1017	26.1	1033	23.5
	4 #20 + 2 #12	1483	923	42.1	958	36.9	987	32.7	1011	29.2	1031	26.3	1048	23.7
	4 #16 + 4 #16	1608	987	35.7	1021	31.0	1049	27.3	1072	24.2	1089	21.7	1104	19.5
	4 #20 + 2 #16	1659	976	42.1	1012	36.9	1041	32.7	1066	29.2	1086	26.3	1102	23.7
	4 #20 + 4 #12	1709	985	42.8	1023	37.5	1054	33.2	1079	29.7	1099	26.7	1116	24.1
	4 #25 + 0 #0	1963	1016	52.7	1059	46.5	1094	41.3	1123	37.1	1147	33.4	1168	30.2

For details of arrangement of longitudinal Reinforcement see NOTES at the end of *Table G-2A.*

Table G-2a **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u in kN continued . . .**

P_u - M_u
M 20, Fe 415
b = 200 mm

Depth D mm	Steel N1 D1 + N2 D2 mm mm	Area mm²	0.4 P_u	0.4 M_u	0.5 P_u	0.5 M_u	0.6 P_u	0.6 M_u	0.7 P_u	0.7 M_u	0.8 P_u	0.8 M_u	0.9 P_u	0.9 M_u
400	4 #20 + 4 #16	2061	94	100.2	281	101.8	466	97.6	669	83.5	831	69.8	968	56.7
	4 #25 + 2 #12	2190	157	124.8	280	126.6	421	120.8	638	101.2	815	83.8	961	68.0
	4 #25 + 2 #16	2366	126	124.8	280	126.6	441	120.8	672	101.0	860	83.8	1012	68.0
450	4 #16 + 0 #0	804	249	86.9	321	90.1	404	88.6	521	78.8	630	67.6	730	55.8
	4 #12 + 4 #12	905	191	71.2	321	74.3	449	73.2	574	66.4	685	57.7	784	47.6
	6 #12 + 2 #12	905	212	79.9	322	83.3	427	82.2	553	73.8	669	63.7	772	52.7
	4 #16 + 2 #12	1030	210	86.9	321	90.1	430	88.6	565	78.8	688	67.6	795	55.8
	6 #12 + 4 #12	1131	188	84.8	320	87.9	453	86.0	592	76.8	715	65.9	825	54.2
	6 #16 + 0 #0	1206	244	110.8	320	113.8	412	111.0	554	96.8	686	81.9	802	67.1
	4 #16 + 2 #16	1206	179	86.9	321	90.1	450	88.6	599	78.8	733	67.6	847	55.8
	4 #16 + 4 #12	1257	186	91.7	320	94.6	455	92.3	604	81.7	734	69.8	849	57.3
	4 #20 + 0 #0	1257	243	112.8	319	115.7	414	112.7	561	97.9	695	82.7	813	67.8
	6 #16 + 2 #12	1433	205	110.8	320	113.8	437	111.0	598	96.8	743	81.9	868	67.1
	4 #20 + 2 #12	1483	203	112.8	319	115.7	439	112.7	604	97.9	752	82.7	879	67.8
	4 #16 + 4 #16	1608	136	95.4	319	98.1	495	95.2	669	84.0	815	71.4	941	58.4
	4 #20 + 2 #16	1659	173	112.8	319	115.7	459	112.7	638	97.9	797	82.7	930	67.8
	4 #20 + 4 #12	1709	179	117.5	318	120.1	465	116.3	644	100.8	799	84.8	932	69.2
	4 #25 + 0 #0	1963	230	150.7	316	153.2	432	147.4	630	124.9	801	104.0	948	84.6
	4 #20 + 4 #16	2061	129	121.2	317	123.5	505	119.1	709	103.0	880	86.5	1025	70.3
	4 #25 + 4 #12	2190	191	150.7	316	153.2	457	147.4	673	124.9	859	104.0	1014	84.6
	4 #25 + 2 #16	2366	160	150.7	316	153.2	477	147.4	707	124.9	904	104.0	1066	84.6
	4 #25 + 4 #12	2416	165	155.2	315	157.4	483	150.8	713	127.7	906	106.0	1068	86.0
500	4 #16 + 0 #0	804	279	103.2	357	107.4	446	105.9	567	95.1	684	81.8	791	67.6
	4 #12 + 4 #12	905	222	85.2	358	89.3	491	88.1	622	80.5	740	70.1	847	58.0
	6 #12 + 2 #12	905	242	95.2	358	99.4	469	98.4	600	89.1	724	77.1	834	63.8
	4 #16 + 2 #12	1030	240	103.2	357	107.4	472	105.9	611	95.1	742	81.8	857	67.6
	6 #12 + 4 #12	1131	219	100.9	357	104.8	495	102.8	638	92.6	769	79.7	886	65.6
	6 #16 + 0 #0	1206	275	130.7	356	134.7	453	131.7	597	116.2	737	98.6	862	81.0
	4 #16 + 2 #16	1206	209	103.2	357	107.4	491	105.9	645	95.1	786	81.1	908	67.6
	4 #16 + 4 #12	1257	217	108.8	356	112.7	497	110.3	650	98.5	787	84.4	910	69.3

Neutral Axis Factor $k_u = x_u / D$

For details of arrangement of longitudinal Reinforcement see NOTES at the end of *Table G-2A*.

Table G-2a **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u in kN continued . . .**

P_u - M_u
M 20, Fe 415
b = 200 mm

Depth D mm	Steel N1 D1 + N1 D2 mm	Area mm²	Neutral Axis Factor $k_u = x_u/D$: 1.00 P_u	1.00 M_u	1.05 P_u	1.05 M_u	1.10 P_u	1.10 M_u	1.15 P_u	1.15 M_u	1.20 P_u	1.20 M_u	1.30 P_u	1.30 M_u
400	4 #20 + 4 #16	2061	1088	43.4	1128	38.0	1160	33.6	1185	30.0	1206	27.0	1224	24.3
	4 #25 + 2 #12	2190	1085	52.7	1128	46.5	1164	41.3	1193	37.1	1217	33.4	1238	30.2
	4 #25 + 2 #16	2366	1139	52.7	1182	46.5	1218	41.3	1247	37.1	1272	33.4	1292	30.2
450	4 #16 + 0 #0	804	821	42.8	855	37.3	881	32.8	903	29.1	921	26.1	937	23.5
	4 #12 + 4 #12	905	876	36.0	908	31.1	933	27.1	954	23.9	970	21.3	983	19.1
	6 #12 + 2 #12	905	863	40.3	895	35.0	920	30.8	941	27.3	959	24.4	974	21.9
	4 #16 + 2 #12	1030	890	42.8	924	37.3	951	32.8	973	29.1	991	26.1	1007	23.5
	6 #12 + 4 #12	1131	924	41.3	959	35.8	986	31.4	1009	27.8	1027	24.9	1042	22.4
	6 #16 + 0 #0	1206	907	51.9	945	45.5	976	40.3	1001	36.0	1022	32.4	1040	29.3
	4 #16 + 2 #16	1206	944	42.8	978	37.3	1005	32.8	1027	29.1	1046	26.1	1061	23.5
	4 #16 + 4 #12	1257	952	43.7	988	38.0	1017	33.5	1040	29.7	1059	26.6	1075	24.0
	4 #20 + 0 #0	1257	919	52.4	958	45.9	989	40.7	1015	36.3	1036	32.7	1055	29.6
	6 #16 + 2 #12	1433	976	51.9	1014	45.5	1045	40.3	1071	36.0	1092	32.4	1111	29.3
	4 #20 + 2 #12	1483	989	52.4	1027	45.9	1059	40.7	1085	36.3	1106	32.7	1125	29.6
	4 #16 + 4 #16	1608	1054	44.4	1092	38.7	1122	34.0	1147	30.1	1166	27.0	1182	24.3
	4 #20 + 2 #16	1659	1042	52.4	1081	45.9	1113	40.7	1139	36.3	1161	32.7	1179	29.6
	4 #20 + 4 #12	1709	1051	53.3	1092	46.7	1125	41.3	1152	36.9	1174	33.2	1193	30.3
	4 #25 + 0 #0	1963	1077	65.7	1124	58.0	1162	51.6	1193	46.3	1220	41.8	1242	37.9
	4 #20 + 4 #16	2061	1153	54.0	1195	47.3	1230	41.8	1259	37.3	1281	33.6	1300	30.4
	4 #25 + 2 #12	2190	1147	65.7	1193	58.0	1231	51.6	1263	46.3	1290	41.8	1312	37.9
	4 #25 + 2 #16	2366	1200	65.7	1247	58.0	1285	51.6	1317	46.3	1344	41.8	1367	37.9
	4 #25 + 4 #12	2416	1209	66.6	1258	58.7	1298	52.2	1330	46.8	1358	42.3	1380	38.3
500	4 #16 + 0 #0	804	890	51.9	926	45.2	955	39.7	979	35.3	999	31.6	1015	28.5
	4 #12 + 4 #12	905	946	43.8	981	37.8	1008	33.0	1031	29.1	1048	25.8	1063	23.2
	6 #12 + 2 #12	905	932	48.9	967	42.5	995	37.3	1018	33.0	1037	29.5	1053	26.6
	4 #16 + 2 #12	1030	959	51.9	996	45.2	1025	39.7	1049	35.3	1069	31.6	1086	28.5
	6 #12 + 4 #12	1131	994	50.0	1031	43.4	1060	38.1	1085	33.7	1104	30.1	1121	27.1
	6 #16 + 0 #0	1206	974	62.7	1016	54.9	1049	48.6	1076	43.4	1099	39.1	1118	35.3
	4 #16 + 2 #16	1206	1013	51.9	1050	45.2	1079	39.7	1103	35.3	1123	31.6	1140	28.5
	4 #16 + 4 #12	1257	1021	53.0	1059	46.1	1090	40.5	1116	36.0	1136	32.2	1154	29.0

For details of arrangement of longitudinal Reinforcement see NOTES at the end of *Table G-2A*.

Table G-2a **Allowable Combinations of Ultimate axial Compression P_u in *kN* and Ultimate Moment M_u in *kN* continued . . .**

P_u - M_u
M 20, Fe 415
b = 200 mm

Depth	*Steel*		Neutral Axis Factor $k_u = x_u/D$											
D	*N1 D1 + N2 D2*	*Area*	0.4		0.5		0.6		0.7		0.8		0.9	
mm	*mm mm*	*mm²*	P_u	M_u	P_u	M_u	P_u	M_u	P_u	M_u	P_u	M_u	P_u	M_u
500	4 #20 + 0 #0	1257	274	133.2	355	137.1	454	133.9	604	117.7	746	99.7	873	81.8
	6 #16 + 2 #12	1433	235	130.7	356	134.7	478	131.7	641	116.2	795	98.6	928	81.0
	4 #20 + 2 #12	1483	234	133.2	355	137.1	480	133.9	647	117.7	804	99.7	938	81.8
	4 #16 + 4 #16	1608	168	113.2	355	116.8	537	113.7	714	101.2	868	86.3	1002	70.7
	4 #20 + 2 #16	1659	203	133.2	355	137.1	500	133.9	681	117.7	848	99.7	990	81.8
	4 #20 + 4 #12	1709	211	138.7	354	142.3	505	138.2	687	121.1	849	102.2	991	83.6
	4 #25 + 0 #0	1963	263	177.6	352	180.9	469	175.1	667	150.1	848	125.3	1004	102.1
	4 #20 + 4 #16	2061	162	143.0	353	146.4	545	141.5	751	123.7	930	104.1	1083	84.9
	4 #25 + 2 #12	2190	224	177.6	352	180.9	495	175.1	711	150.1	906	125.3	1070	102.1
	4 #25 + 2 #16	2366	193	177.6	352	180.9	514	175.1	745	150.1	951	125.3	1121	102.1
	4 #25 + 4 #12	2416	199	182.9	351	185.9	520	179.2	751	153.3	952	127.7	1123	103.8
	4 #25 + 4 #16	2768	149	187.0	350	189.8	560	182.4	816	155.9	1033	129.5	1216	105.0
	4 #25 + 2 #25	2945	91	177.6	352	180.9	579	175.1	857	150.1	1098	125.3	1290	102.1
530	4 #12 + 4 #12	905	240	94.1	379	98.7	517	97.6	650	89.4	773	78.0	885	64.5
	6 #12 + 2 #12	905	259	104.8	380	109.5	495	108.6	628	98.8	757	85.7	872	71.0
	4 #16 + 2 #12	1030	258	113.5	379	118.2	497	116.7	638	105.4	774	90.9	894	75.1
	6 #12 + 4 #12	1131	237	111.0	378	115.4	520	113.5	667	102.6	802	88.5	924	72.9
	6 #16 + 0 #0	1206	293	143.2	377	147.7	478	144.6	624	128.4	769	109.1	898	89.7
	4 #16 + 2 #16	1206	227	113.5	379	118.2	517	116.7	672	105.4	819	90.9	945	75.1
	4 #16 + 4 #12	1257	235	119.6	378	124.0	522	121.5	677	109.1	820	93.6	947	77.0
	4 #20 + 0 #0	1257	292	145.9	377	150.4	479	147.1	630	130.1	777	110.4	909	90.7
	6 #16 + 2 #12	1433	253	143.2	377	147.7	503	144.6	668	128.4	826	109.1	964	89.7
	4 #20 + 2 #12	1483	252	145.9	377	150.4	505	147.1	674	130.1	835	110.4	975	90.7
	4 #16 + 4 #16	1608	187	124.4	377	128.5	562	125.3	741	112.0	900	95.7	1039	78.5
	4 #20 + 2 #16	1659	221	145.9	377	150.4	524	147.1	708	130.1	880	110.4	1026	90.7
	4 #20 + 4 #12	1709	229	151.9	379	156.1	530	151.8	713	133.8	881	113.1	1027	92.6
	4 #25 + 0 #0	1963	282	194.1	374	198.1	493	192.0	691	165.7	877	138.6	1038	113.1
	4 #20 + 4 #16	2061	181	156.6	375	160.6	570	155.5	777	136.6	961	115.3	1119	94.0
	4 #25 + 2 #12	2190	242	194.1	374	198.1	518	192.0	735	165.7	935	138.6	1104	113.1
	4 #25 + 2 #16	2366	212	194.1	374	198.1	538	192.0	769	165.7	980	138.6	1155	113.1

For details of arrangement of longitudinal Reinforcement see NOTES at the end of *Table G-2A*.

Table G-2a **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u in kN continued . . .**

P_u - M_u
M 20, Fe 415
b = 200 mm

Depth D mm	Steel N1 D1 + N2 D2 mm mm	Area mm²	Neutral Axis Factor $k_u = x_u/D$ 1.00 P_u	1.00 M_u	1.05 P_u	1.05 M_u	1.10 P_u	1.10 M_u	1.15 P_u	1.15 M_u	1.20 P_u	1.20 M_u	1.30 P_u	1.30 M_u
500	4 #20 + 0 #0	1257	987	63.3	1028	55.5	1062	49.1	1089	43.9	1113	39.5	1132	35.7
	6 #16 + 2 #12	1433	1044	62.7	1085	54.9	1118	48.6	1146	43.4	1169	39.1	1188	35.3
	4 #20 + 2 #12	1483	1056	63.3	1098	55.5	1131	49.1	1159	43.9	1183	39.5	1202	35.7
	4 #16 + 4 #16	1608	1122	53.9	1163	46.8	1195	41.1	1222	36.5	1243	32.7	1261	29.4
	4 #20 + 2 #16	1659	1109	63.3	1151	55.5	1185	49.1	1214	43.9	1237	39.5	1257	35.7
	4 #20 + 4 #12	1709	1117	64.4	1161	56.4	1197	49.9	1226	44.5	1250	40.1	1270	36.3
	4 #25 + 0 #0	1963	1141	79.5	1191	70.1	1231	62.4	1265	56.0	1293	50.6	1317	46.0
	4 #20 + 4 #16	2061	1219	65.3	1265	57.1	1302	50.5	1333	45.0	1357	40.5	1378	36.7
	4 #25 + 2 #12	2190	1210	79.5	1260	70.1	1301	62.4	1335	56.0	1363	50.6	1387	46.0
	4 #25 + 2 #16	2366	1264	79.5	1314	70.1	1355	62.4	1389	56.0	1418	50.6	1442	46.0
	4 #25 + 4 #12	2416	1272	80.5	1324	71.0	1367	63.2	1402	56.7	1431	51.2	1456	46.5
	4 #25 + 4 #16	2768	1374	81.3	1428	71.7	1472	63.7	1509	57.1	1583	51.6	1563	46.9
	4 #25 + 2 #25	2945	1441	79.5	1492	70.1	1533	62.4	1568	56.0	1597	50.6	1622	46.0
530	4 #12 + 4 #12	905	989	48.8	1025	42.1	1053	36.7	1077	32.3	1096	28.8	1111	25.8
	6 #12 + 2 #12	905	974	54.3	1011	47.2	1040	41.4	1064	36.7	1084	32.8	1100	29.5
	4 #16 + 2 #12	1030	1001	57.6	1039	50.2	1070	44.1	1095	39.2	1116	35.1	1133	31.6
	6 #12 + 4 #12	1131	1035	55.6	1074	48.2	1105	42.3	1131	37.5	1151	33.5	1168	30.1
	6 #16 + 0 #0	1206	1015	69.4	1058	60.8	1093	53.8	1121	48.1	1145	43.2	1165	39.1
	4 #16 + 2 #16	1206	1055	57.6	1093	50.2	1124	44.1	1149	39.2	1170	35.1	1188	31.6
	4 #16 + 4 #12	1257	1062	58.9	1103	51.2	1135	45.0	1162	39.9	1183	35.7	1201	32.2
	4 #20 + 0 #0	1257	1027	70.2	1071	61.5	1106	54.5	1135	48.6	1159	43.8	1179	39.6
	6 #16 + 2 #12	1433	1085	69.4	1128	60.8	1162	53.8	1191	48.1	1215	43.2	1235	39.1
	4 #20 + 2 #12	1483	1096	70.2	1140	61.5	1175	54.5	1204	48.6	1229	43.8	1249	39.6
	4 #16 + 4 #16	1608	1164	59.8	1206	52.0	1240	45.7	1268	40.5	1290	36.2	1308	32.7
	4 #20 + 2 #16	1659	1150	70.2	1194	61.5	1229	54.5	1259	48.6	1283	43.8	1304	39.6
	4 #20 + 4 #12	1709	1158	71.4	1204	62.5	1241	55.3	1271	49.4	1296	44.4	1317	40.2
	4 #25 + 0 #0	1963	1180	88.1	1232	77.7	1274	69.2	1309	62.1	1338	56.1	1363	51.0
	4 #20 + 4 #16	2061	1259	72.3	1307	63.3	1346	56.0	1377	49.9	1403	44.9	1424	40.7
	4 #25 + 2 #12	2190	1210	79.5	1260	70.1	1301	62.4	1335	56.0	1363	50.6	1387	46.0
	4 #25 + 2 #16	2366	1303	88.1	1355	77.7	1397	69.2	1433	62.1	1462	56.1	1488	51.0

For details of arrangement of longitudinal Reinforcement see NOTES at the end of *Table G-2A*.

Table G-2a **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u in kN continued . . .**

P_u - M_u
M 20, Fe 415
b = 200 mm

Depth D mm	Steel N1 D1 + N2 D2 mm	Area mm²	0.4 P_u	0.4 M_u	0.5 P_u	0.5 M_u	0.6 P_u	0.6 M_u	0.7 P_u	0.7 M_u	0.8 P_u	0.8 M_u	0.9 P_u	0.9 M_u
530	4 #25 + 4 #12	2416	219	199.9	372	203.5	544	196.5	774	169.3	981	141.2	1157	114.9
	4 #25 + 4 #16	2768	169	204.4	371	207.8	584	200.0	839	172.0	1061	143.2	1249	116.3
	4 #25 + 2 #25	2945	110	194.1	374	198.1	603	192.0	881	165.7	1127	138.6	1324	113.1
550	4 #12 + 4 #12	905	252	100.2	394	105.1	534	104.1	670	95.6	795	83.5	910	69.1
	6 #12 + 2 #12	905	271	111.4	394	116.4	512	115.6	647	105.5	779	91.6	897	75.9
	4 #16 + 2 #12	1030	270	120.6	394	125.6	514	124.2	657	112.4	796	97.1	919	80.3
	6 #12 + 4 #12	1131	250	117.9	393	122.7	537	120.8	685	109.5	824	94.5	949	77.9
	6 #16 + 0 #0	1206	305	151.7	392	156.5	494	153.5	642	136.7	790	116.4	923	95.7
	4 #16 + 2 #16	1206	239	120.6	394	125.6	534	124.2	691	112.4	841	97.1	970	80.3
	4 #16 + 4 #12	1257	248	127.0	392	131.7	539	129.3	696	116.4	841	100.0	971	82.3
	4 #20 + 0 #0	1257	304	154.6	391	159.4	496	156.1	648	138.6	798	117.8	933	96.8
	6 #16 + 2 #12	1433	265	151.7	392	156.5	520	153.5	686	136.7	848	116.4	988	95.7
	4 #20 + 2 #12	1483	264	154.6	391	159.4	521	156.1	692	138.6	856	117.8	999	96.8
	4 #16 + 4 #16	1608	200	132.0	391	136.5	579	133.2	760	119.5	921	102.2	1063	83.9
	4 #20 + 2 #16	1659	234	154.6	391	159.4	541	156.1	726	138.6	901	117.8	1050	96.8
	4 #20 + 4 #12	1709	242	160.9	390	165.5	547	161.1	731	142.5	901	120.6	1051	98.8
	4 #25 + 0 #0	1963	294	205.3	388	209.7	509	203.4	708	176.4	897	147.7	1061	120.6
	4 #20 + 4 #16	2061	194	165.9	389	170.2	586	165.0	795	145.5	982	122.9	1143	100.3
	4 #25 + 2 #12	2190	255	205.3	388	209.7	534	203.4	751	176.4	954	147.7	1127	120.6
	6 #20 + 2 #16	2287	227	202.7	389	207.3	551	201.4	772	175.9	982	147.5	1159	120.5
	6 #20 + 4 #12	2337	235	209.1	387	213.3	556	206.4	777	179.8	983	150.3	1160	122.5
	4 #25 + 2 #16	2366	224	205.3	388	209.7	554	203.4	785	176.4	999	147.7	1178	120.6
	4 #25 + 4 #12	2416	232	211.4	387	215.5	560	208.2	791	180.2	1000	150.4	1180	122.5
	6 #20 + 4 #16	2689	187	214.0	386	218.0	596	210.2	841	182.9	1063	152.5	1252	124.0
	4 #25 + 4 #16	2768	183	216.2	386	220.0	600	211.9	855	183.1	1081	152.6	1272	124.0
600	4 #16 + 2 #12	1030	299	138.9	430	144.8	556	143.6	705	130.9	851	113.3	982	93.8
	6 #12 + 4 #12	1131	280	135.9	429	141.6	580	139.8	733	127.5	879	110.3	1011	91.1
	6 #16 + 0 #0	1206	335	173.6	428	179.3	536	176.4	688	158.4	844	135.1	984	111.3
	4 #16 + 2 #16	1206	268	138.9	430	144.8	576	143.6	739	130.9	896	113.3	1033	93.8
	4 #16 + 4 #12	1257	278	146.1	428	151.8	582	149.4	743	135.4	896	116.6	1034	96.1

For details of arrangement of longitudinal Reinforcement see NOTES at the end of *Table G-2A*.

Table G-2a **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u in kNcontinued . . .**

P_u - M_u
M 20, Fe 415
b = 200 mm

Depth D mm	Steel N1 D1 mm	+ N2 D2 mm	Area mm²	Neutral Axis Factor $k_u = x_u/D$ 1.00 P_u	1.00 M_u	1.05 P_u	1.05 M_u	1.10 P_u	1.10 M_u	1.15 P_u	1.15 M_u	1.20 P_u	1.20 M_u	1.30 P_u	1.30 M_u
530	4 #25	+ 4 #12	2416	1311	89.3	1365	78.7	1409	70.0	1446	62.8	1476	56.7	1501	51.6
	4 #25	+ 4 #16	2768	1413	90.1	1469	79.4	1514	70.6	1552	63.3	1583	57.2	1609	52.0
	4 #25	+ 2 #25	2945	1480	88.1	1533	77.7	1576	69.2	1612	62.1	1642	56.1	1668	51.0
550	4 #12	+ 4 #12	905	1017	52.3	1054	45.1	1084	39.3	1108	34.6	1127	30.8	1143	27.6
	6 #12	+ 2 #12	905	1002	58.1	1040	50.4	1070	44.3	1095	39.2	1115	35.0	1132	31.5
	4 #16	+ 2 #12	1030	1029	61.6	1068	53.6	1100	47.1	1125	41.8	1147	37.4	1165	33.7
	6 #12	+ 4 #12	1131	1063	59.4	1103	51.6	1135	45.2	1161	40.0	1183	35.7	1200	32.2
	6 #16	+ 0 #0	1206	1043	74.1	1087	64.9	1122	57.4	1152	51.3	1176	46.1	1196	41.7
	4 #16	+ 2 #16	1206	1083	61.6	1122	53.6	1154	47.1	1180	41.8	1201	37.4	1219	33.7
	4 #16	+ 4 #12	1257	1090	62.9	1132	54.7	1165	48.1	1192	42.6	1214	38.1	1233	34.4
	4 #20	+ 0 #0	1257	1055	74.9	1099	65.7	1135	58.1	1165	51.9	1189	46.7	1210	42.2
	6 #16	+ 2 #12	1433	1112	74.1	1156	64.9	1192	57.4	1221	51.3	1246	46.1	1267	41.7
	4 #20	+ 2 #12	1483	1124	74.9	1169	65.7	1205	58.1	1234	51.9	1259	46.7	1280	42.2
	4 #16	+ 4 #16	1608	1191	63.9	1235	55.6	1270	48.8	1298	43.2	1321	38.7	1340	34.9
	4 #20	+ 2 #16	1659	1178	74.9	1223	65.7	1259	58.1	1289	51.9	1314	46.7	1335	42.2
	4 #20	+ 4 #12	1709	1185	76.2	1232	66.7	1270	59.0	1301	52.7	1327	47.4	1348	42.9
	4 #25	+ 0 #0	1963	1206	94.0	1259	82.9	1302	73.8	1338	66.2	1368	59.8	1394	54.5
	4 #20	+ 4 #16	2061	1286	77.2	1335	67.6	1375	59.7	1407	53.2	1434	47.9	1456	43.4
	4 #25	+ 2 #12	2190	1276	94.0	1329	82.9	1372	73.8	1408	66.2	1438	59.8	1464	54.5
	4 #25	+ 2 #16	2366	1329	94.0	1383	82.9	1426	73.8	1462	66.2	1492	59.8	1518	54.5
	4 #25	+ 4 #12	2416	1337	95.2	1392	83.9	1437	74.7	1475	66.9	1506	60.5	1532	55.0
	4 #25	+ 4 #16	2768	1439	96.2	1496	84.7	1542	75.3	1581	67.5	1613	61.0	1639	55.4
	4 #25	+ 2 #25	2945	1506	94.0	1560	82.9	1604	73.8	1641	66.2	1672	59.8	1698	54.5
	4 #32	+ 0 #0	3217	1476	127.5	1544	113.2	1599	101.4	1646	91.4	1685	83.0	1719	75.7
	4 #25	+ 4 #20	3220	1569	97.4	1629	85.7	1677	76.2	1718	68.2	1750	61.7	1777	56.0
600	4 #16	+ 2 #12	1030	1099	72.0	1141	62.6	1175	55.0	1202	48.8	1225	43.7	1244	39.3
	6 #12	+ 4 #12	1131	1133	69.5	1176	60.2	1210	52.8	1238	46.7	1261	41.7	1280	37.5
	6 #16	+ 0 #0	1206	1112	86.1	1159	75.4	1197	66.7	1228	59.5	1253	53.5	1275	48.4
	4 #16	+ 2 #16	1206	1153	72.0	1195	62.6	1229	55.0	1257	48.8	1280	43.7	1299	39.3
	4 #16	+ 4 #12	1257	1160	73.5	1204	63.9	1240	56.1	1269	49.7	1292	44.5	1312	40.1

For details of arrangement of longitudinal Reinforcement see NOTES at the end of *Table G-2A*.

Table G-2a **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u in kN continued . . .**

P_u - M_u
M 20, Fe 415
b = 200 mm

Depth D mm	Steel N1 D1 + N2 D2 mm mm	Area mm^2	0.4 P_u	0.4 M_u	0.5 P_u	0.5 M_u	0.6 P_u	0.6 M_u	0.7 P_u	0.7 M_u	0.8 P_u	0.8 M_u	0.9 P_u	0.9 M_u
			Neutral Axis Factor $k_u = x_u/D$											
600	4 #20 + 0 #0	1257	334	177.0	428	182.6	537	179.5	694	160.6	852	136.9	994	112.6
	6 #16 + 2 #12	1433	295	173.6	428	179.3	561	176.4	732	158.4	901	135.1	1050	111.3
	4 #20 + 2 #12	1483	294	177.0	428	182.6	563	179.5	737	160.6	909	136.9	1060	112.6
	4 #16 + 4 #16	1608	231	151.8	427	157.2	621	153.9	807	138.9	976	119.1	1125	97.9
	4 #20 + 2 #16	1659	264	177.0	428	182.6	583	179.5	771	160.6	954	136.9	1111	112.6
	4 #20 + 4 #12	1709	273	184.1	426	189.5	588	185.1	776	165.0	954	140.1	1112	114.9
	4 #25 + 0 #0	1963	325	234.0	424	239.4	550	232.8	750	203.9	947	171.2	1120	140.0
	4 #20 + 4 #16	2061	225	189.7	425	194.8	628	189.6	839	168.4	1034	142.6	1203	116.7
	4 #25 + 2 #12	2190	286	234.0	424	239.4	575	232.8	793	203.9	1005	171.2	1186	140.0
	4 #25 + 2 #16	2366	255	234.0	424	239.4	595	232.8	827	203.9	1050	171.2	1237	140.0
	4 #25 + 4 #12	2416	263	240.9	423	246.6	601	238.3	832	208.2	1050	174.3	1238	142.1
	4 #25 + 4 #16	2768	215	246.3	422	251.1	640	242.5	896	211.5	1130	176.7	1330	143.8
	4 #25 + 2 #25	2945	154	234.0	424	239.4	660	232.8	939	203.9	1197	171.2	1406	140.0
	4 #32 + 0 #0	3217	310	334.6	419	339.5	572	327.0	850	280.2	1117	231.6	1344	188.1
	4 #25 + 4 #20	3220	153	253.3	420	257.7	691	248.0	979	215.8	1233	179.9	1448	146.0

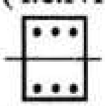
Notes : Arrangement of Longitudinal Steel.

N1 - D1 Represent Total Number of bars in first and Last row having same diameter.

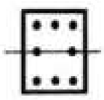
N2 - D2 Represent Total Number of bars BETWEEN first and Last row

For Example

6 #20 + 0 -#0 (i.e.N1 = 6 and N_2 = 0), means 3 No. of 20 *mm* diameter bars in first row and 3 No. of 20 *mm* in the last row.
Thus, total No. of rows = 2



6 #20 + 2 -#12 (i.e.N1 = 6 and N_2 = 2), means 3 No. of 20 *mm* diameter bars in first row and 3 No.of 20 *mm* in the last row and 2 No. of 12 *mm* bars at mid-depth
Thus, total No. of rows = 3



6 #20 + 4 -#12 (i.e.N1 = 6 and N_2 = 4), means 3 No of 20 *mm* diameter bars in first row and 3 No. of 20 *mm* in the last row and 2 No. of 12 *mm* bars in 2nd row and 2 No. of 12 *mm* in 3rd row equispaced.
Thus, total No. of rows = 4

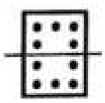


Table G-2a **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u in kN continued . . .**

P_u - M_u
M 20, Fe 415
b = 200 mm

Depth D mm	Steel N1 D1 + N2 D2 mm	Area mm^2	Neutral Axis Factor $k_u = x_u/D$ 1.00 P_u	1.00 M_u	1.05 P_u	1.05 M_u	1.10 P_u	1.10 M_u	1.15 P_u	1.15 M_u	1.20 P_u	1.20 M_u	1.30 P_u	1.30 M_u
600	4 #20 + 0 #0	1257	1123	87.2	1171	76.4	1209	67.5	1240	60.3	1267	54.2	1289	49.1
	6 #16 + 2 #12	1433	1181	86.1	1228	75.4	1266	66.7	1297	59.5	1323	53.5	1345	48.4
	4 #20 + 2 #12	1483	1193	87.2	1240	76.4	1279	67.5	1310	60.3	1337	54.2	1359	49.1
	4 #16 + 4 #16	1608	1261	74.6	1307	64.8	1344	56.9	1375	50.4	1399	45.1	1419	40.6
	4 #20 + 2 #16	1659	1246	87.2	1294	76.4	1333	67.5	1365	60.3	1391	54.2	1414	49.1
	4 #20 + 4 #12	1709	1253	88.6	1303	77.6	1344	68.6	1377	61.2	1404	55.0	1427	49.8
	4 #25 + 0 #0	1963	1273	109.2	1329	96.2	1374	85.6	1412	76.8	1444	69.4	1471	63.1
	4 #20 + 4 #16	2061	1354	89.8	1406	78.6	1448	69.4	1483	61.9	1511	55.6	1534	50.3
	4 #25 + 2 #12	2190	1342	109.2	1398	96.2	1444	85.6	1482	76.8	1514	69.4	1541	63.1
	4 #25 + 2 #16	2366	1396	109.2	1452	96.2	1498	85.6	1536	76.8	1568	69.4	1595	63.1
	4 #25 + 4 #12	2416	1403	110.6	1461	97.4	1509	86.6	1548	77.7	1581	70.2	1609	63.8
	4 #25 + 4 #16	2768	1504	111.7	1565	98.3	1614	87.4	1655	78.3	1688	70.8	1716	64.3
	4 #25 + 2 #25	2945	1573	109.2	1630	96.2	1676	85.6	1715	76.8	1747	69.4	1775	63.1
	4 #32 + 0 #0	3217	1538	147.8	1609	131.2	1668	117.5	1716	106.0	1758	96.2	1793	87.8
	4 #25 + 4 #20	3220	1635	113.1	1697	99.5	1748	88.4	1791	79.2	1825	71.5	1854	65.0

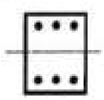
Notes : Arrangement of Longitudinal Steel.

N1 - D1 Represent Total Number of bars in first and Last row having Same diameter.

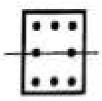
N2 - D2 Represent Total Number of bars BETWEEN first and Last row

For Example

6 #20 + 0 -#0 (i.e.N1 = 6 and *N2* = 0), means 3 No. of 20 *mm* diameter bars in first row and 3 No. of 20 *mm* in the last row.
Thus, total No. of rows = 2



6 #20 + 2 -#12 (i.e.N1 = 6 and *N2* = 2), means 3 No. of 20 *mm* diameter bars in first row and 3 No.of 20 *mm* in the last row, and 2 No. of 12 *mm* bars at mid-depth
Thus, total No. of rows = 3



6 #20 + 4 -#12 (i.e.N1 = 6 and *N2* = 4), means 3 No of 20 *mm* diameter bars in first row and 3 No. of 20 *mm* in the last row, and 2 No. of 12 *mm* bars in 2^{nd} row and 2 No. of 12 *mm* in 3^{rd} row equispaced.
Thus, total No. of rows = 4

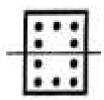


Table G-2b **Allowable Combinations of Ultimate axial Compression P_u in *kN* and Ultimate Moment about major axis of Bending M_{ux} in *kN.m* for Rectangular Columns - *230 mm wide* × Depth 'D' Concrete M20, Steel Fe 415**

P_u - M_u
M 20, Fe 415
b = 230 mm

Depth D mm	Steel N1 D1 + N2 D2 mm mm	Area mm²	Neutral Axis Factor $k_u = x_u/D$ 0.4 P_u	0.4 M_u	0.5 P_u	0.5 M_u	0.6 P_u	0.6 M_u	0.7 P_u	0.7 M_u	0.8 P_u	0.8 M_u	0.9 P_u	0.9 M_u
230	4 #12 + 0 #0	452	137	20.9	189	22.0	257	20.7	326	18.6	387	16.2	442	13.4
	4 #12 + 0 #12	679	98	20.9	189	22.0	282	20.7	370	18.6	447	16.2	508	13.4
	6 #12 + 0 #0	679	130	25.5	188	26.5	271	24.5	355	21.5	427	18.5	491	15.2
	4 #16 + 0 #0	804	123	27.3	187	28.3	282	25.9	374	22.6	452	19.3	520	15.8
	4 #12 + 4 #12	905	64	22.3	188	23.31	308	21.7	413	19.5	495	6.8	566	13.8
	6 #12 + 2 #12	905	90	25.5	188	26.5	296	24.5	399	21.5	485	18.5	557	15.2
	4 #16 + 2 #12	1030	83	27.3	187	28.3	307	25.9	418	22.6	510	19.3	586	15.8
	6 #12 + 4 #12	1131	56	26.9	187	27.7	322	25.5	442	22.4	536	19.1	615	15.6
	6 #16 + 0 #0	1206	108	35.1	186	36.1	308	32.2	428	27.4	525	23.0	609	18.7
	4 #16 + 2 #16	1206	52	27.3	187	28.3	327	25.9	452	22.6	555	193	637	15.8
	4 #16 + 4 #12	1257	48	28.7	186	29.5	333	26.8	462	23.3	561	19.8	645	16.1
	4 #20 + 0 #0	1257	97	34.7	185	36.0	317	31.9	439	27.1	539	22.8	623	18.5
	6 #16 + 2 #12	1433	68	35.1	186	36.1	333	32.2	471	27.4	583	23.0	674	18.7
	4 #20 + 2 12	1483	57	34.7	185	36.0	342	31.9	482	27.1	596	22.8	689	18.5
300	4 #12 + 2 #12	679	150	35.0	247	36.6	340	35.8	441	32.2	528	28.1	604	23.3
	6 #12 + 0 #0	679	185	42.5	246	44.0	323	42.4	421	37.3	507	32.2	583	26.5
	4 #16 + 0 #0	804	182	46.0	246	47.5	330	45.3	437	39.6	529	33.9	610	27.8
	4 #12 + 4 #12	905	121	37.5	246	38.8	366	37.6	482	33.7	577	29.2	660	24.0
	6 #12 + 2 #12	905	145	42.5	246	44.0	349	42.4	465	37.3	564	32.2	649	26.5
	4 #16 + 2 #12	1030	142	46.0	246	47.5	356	45.3	480	39.6	586	33.9	676	27.8
	6 #12 + 4 #12	1131	116	45.0	245	46.2	374	44.2	506	38.8	613	33.2	705	27.2
	6 #16 + 0 #0	1206	173	59.1	244	60.3	346	56.7	481	48.4	594	40.8	691	33.3
	4 #16 + 2 #16	1206	111	46.0	246	47.5	375	45.3	514	39.6	631	33.9	727	27.8
	4 #16 + 4 #12	1257	112	48.4	244	49.6	381	47.0	522	41.0	635	34.9	732	28.5
	4 #20 + 0 #0	1257	170	59.6	243	60.8	352	56.9	490	48.5	605	40.8	705	33.3
	6 #16 + 2 #12	1433	133	59.1	244	60.3	371	56.7	524	48.4	652	40.8	757	33.3
	4 #20 + 2 #12	1483	130	59.6	243	60.8	378	56.9	534	48.5	663	40.8	770	33.3
	4 #16 + 4 #16	1608	58	50.3	243	51.2	421	48.4	589	42.1	718	35.7	827	29.1
	4 #20 + 2 #16	1659	100	59.6	243	60.8	397	56.9	568	48.5	708	40.8	822	33.3
	4 #20 + 4 #12	1709	100	61.9	242	62.9	403	58.5	576	49.9	712	41.8	827	34.0
	4 #25 + 0 #0	1963	147	78.2	240	79.2	396	71.8	582	60.0	733	49.7	858	40.3
	4 #20 + 4 #16	2061	46	63.7	241	64.5	443	59.9	643	51.0	795	42.6	922	34.5
350	4 #12 + 2 #12	679	185	46.3	289	48.6	385	48.1	494	43.5	590	38.0	675	31.6
	6 #12 + 0 #0	679	221	55.8	288	58.0	365	56.9	472	50.4	566	43.4	652	35.9

For details of arrangement of longitudinal Reinforcement see NOTES at the end of Table G-2B.

Table G-2b **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment about major axis of Bending, M_{ux} in kN.m for Rectangular Columns - *230 mm wide* × Depth 'D' Concrete M20, Steel Fe 415**

P_u - M_u
M 20, Fe 415
b = 230 mm

Depth D mm	Steel N1 D1 + N2 D2 mm mm	Area mm²	1.00 P_u	1.00 M_u	1.05 P_u	1.05 M_u	1.10 P_u	1.10 M_u	1.15 P_u	1.15 M_u	1.20 P_u	1.20 M_u	1.30 P_u	1.30 M_u
			Neutral Axis Factor $k_u = x_u/D$											
230	4 #12 + 0 #0	452	494	10.1	512	8.6	526	7.5	537	6.6	547	5.8	555	5.2
	4 #12 + 2 #12	679	563	10.1	581	8.6	595	7.5	607	6.6	617	5.8	625	5.2
	6 #12 + 0 #0	679	550	11.4	569	9.8	585	8.6	598	7.5	609	6.7	619	6.0
	4 #16 + 0 #0	804	583	11.8	604	10.2	620	8.9	634	7.8	646	6.9	655	6.2
	4 #12 + 4 #12	905	629	10.3	648	8.8	663	7.7	676	6.7	686	5.9	695	5.3
	6 #12 + 2 #12	905	619	11.4	639	9.8	655	8.6	668	7.5	679	6.7	689	6.0
	4 #16 + 2 #12	1030	652	11.8	673	10.2	690	8.9	704	7.8	716	6.9	725	6.2
	6 #12 + 4 #12	1131	685	11.6	706	10.0	723	8.7	737	7.7	748	6.8	758	6.1
	6 #16 + 0 #0	1206	683	14.1	707	12.2	727	10.7	743	9.4	757	8.4	769	7.5
	4 #16 + 2 #16	1206	706	11.8	727	10.2	744	8.9	758	7.8	770	6.9	780	6.2
	4 #16 + 4 #12	1257	719	12.0	740	10.4	758	9.1	772	8.0	784	7.1	795	6.3
	4 #20 + 0 #0	1257	699	13.9	723	12.0	743	10.5	760	9.3	773	8.2	785	7.4
	6 #16 + 2 #12	1433	752	14.1	777	12.2	797	10.7	813	9.4	827	8.4	839	7.5
	4 #20 + 2 #12	1483	768	13.9	793	12.0	813	10.5	829	9.3	843	8.2	855	7.4
300	4 #12 + 2 #12	679	671	17.6	695	15.2	713	13.2	728	11.6	740	10.3	751	9.2
	6 #12 + 0 #0	679	655	20.1	680	17.4	700	15.3	717	13.5	730	12.0	742	10.7
	4 #16 + 0 #0	804	685	21.1	712	18.4	733	16.1	750	14.2	764	12.7	776	11.3
	4 #12 + 4 #12	905	736	18.0	761	15.5	781	13.5	796	11.9	809	10.5	819	9.4
	6 #12 + 2 #12	905	724	20.1	749	17.4	770	15.3	786	13.5	800	12.0	812	10.7
	4 #16 + 2 #12	1030	754	21.1	781	18.4	803	16.1	820	14.2	834	12.7	847	11.3
	6 #12 + 4 #12	1131	788	20.5	815	17.8	837	15.6	854	13.8	868	12.2	880	11.0
	6 #16 + 0 #0	1206	779	25.4	809	22.2	834	19.6	854	17.4	871	15.5	885	14.0
	4 #16 + 2 #16	1206	808	21.1	835	18.4	857	16.1	874	14.2	889	12.7	901	11.3
	4 #16 + 4 #12	1257	819	21.6	847	18.7	870	16.4	888	14.5	903	12.9	915	11.6
	4 #20 + 0 #0	1257	793	25.4	824	22.2	849	19.6	869	17.3	886	15.5	901	13.9
	6 #16 + 2 #12	1433	848	25.4	879	22.2	904	19.6	924	17.4	941	15.5	955	14.0
	4 #20 + 2 #12	1483	862	25.4	894	22.2	919	19.6	939	17.3	956	15.5	971	13.9
	4 #16 + 4 #16	1608	923	21.9	953	19.0	976	16.6	995	14.7	1010	13.1	1023	11.7
	4 #20 + 2 #16	1659	916	25.4	947	22.2	973	19.6	993	17.3	1011	15.5	1025	13.9
	4 #20 + 4 #12	1709	927	25.8	960	22.5	986	19.8	1007	17.6	1025	15.7	1040	14.1
	4 #25 + 0 #0	1963	969	30.8	1006	27.0	1035	23.8	1060	21.2	1081	18.9	1099	17.0
	4 #20 + 4 #16	2061	1031	26.1	1066	22.8	1093	20.0	1114	17.8	1132	15.9	1148	14.3
350	4 #12 + 2 #12	679	751	23.9	777	20.6	799	18.0	816	15.8	830	14.0	842	12.5
	6 #12 + 0 #0	679	732	27.3	761	23.7	784	20.8	803	18.4	818	16.4	831	14.7

For details of arrangement of longitudinal Reinforcement see NOTES at the end of Table G-2B.

Table G-2b Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u $kN.m$ continued . . .

P_u - M_u
M 20, Fe 415
b = 230 mm

Depth D mm	Steel N1 D1 + N2 D2 (mm)	Area mm^2	Neutral Axis Factor $k_u = x_u/D$: 0.4		0.5		0.6		0.7		0.8		0.9	
			P_u	M_u	P_u	M_u	P_u	M_u	P_u	M_u	P_u	M_u	P_u	M_u
350	4 #16 + 0 #0	804	218	60.5	287	62.7	370	61.0	486	53.5	587	45.9	678	37.8
	4 #12 + 4 #12	905	158	49.5	287	51.6	411	50.6	535	45.5	638	39.5	730	32.6
	6 #12 + 2 #12	905	181	55.8	288	58.0	391	56.9	516	50.4	624	43.4	718	35.9
	4 #16 + 2 #12	1030	178	60.5	287	62.7	395	61.0	530	53.5	645	45.9	744	37.8
	6 #12 + 4 #12	1131	154	59.1	286	61.0	416	59.3	556	52.3	671	44.9	773	36.8
	6 #16 + 0 #0	1206	210	77.2	285	79.1	380	76.2	525	65.3	648	55.2	755	45.2
	4 #16 + 2 #16	1206	147	60.5	287	62.7	415	61.0	564	53.5	689	45.9	795	37.8
	4 #16 + 4 #12	1257	151	63.7	286	65.5	421	63.4	570	55.4	692	47.3	798	38.7
	4 #20 + 0 #0	1257	208	78.2	285	80.1	385	76.8	533	65.7	659	55.5	267	45.4
	6 #16 + 2 #12	1433	171	77.2	285	79.1	406	76.2	569	65.3	706	55.2	821	45.2
	4 #20 + 2 #12	1483	169	78.2	285	80.1	410	76.8	577	65.7	716	55.5	833	45.4
	4 #16 + 4 #16	1608	98	66.1	285	67.8	460	65.2	636	56.9	774	48.4	892	39.5
	4 #20 + 2 #16	1659	138	78.2	285	80.1	430	76.8	611	65.7	761	55.5	884	45.4
	4 #20 + 4 #12	1709	141	81.3	284	82.9	436	79.1	618	67.6	764	56.8	888	46.3
	4 #25 + 0 #0	1963	191	103.4	282	104.8	418	98.1	616	82.2	778	68.4	914	55.5
	4 #20 + 4 #16	2061	88	83.7	283	85.1	476	80.9	684	69.1	846	57.9	982	47.0
	4 #25 + 2 #12	2190	152	103.4	282	104.8	444	98.1	659	82.2	835	68.4	979	55.5
	4 #25 + 2 #16	2366	121	103.4	282	104.8	464	98.1	693	82.2	880	68.4	1031	55.5
380	4 #16 + 0 #0	804	239	69.7	312	72.3	397	70.8	516	62.4	623	53.7	719	44.2
	4 #12 + 4 #12	905	180	57.3	312	59.8	440	58.8	567	53.1	675	46.2	772	38.2
	6 #12 + 2 #12	905	202	64.3	312	67.0	419	65.9	547	58.8	660	50.8	760	42.0
	4 #16 + 2 #12	1030	200	69.7	312	72.3	423	70.8	560	62.4	680	53.7	785	44.2
	6 #12 + 4 #12	1131	176	68.1	311	70.5	445	68.8	587	61.1	707	52.4	814	43.1
	6 #16 + 0 #0	1206	233	88.5	310	90.9	406	88.1	553	76.1	682	64.4	795	52.8
	4 #16 + 2 #16	1206	169	69.7	312	72.3	442	70.8	594	62.4	725	53.7	836	44.2
	4 #16 + 4 #12	1257	173	73.3	311	75.6	448	73.5	600	64.7	727	55.3	839	45.3
	4 #20 + 0 #0	1257	231	89.9	310	92.2	409	89.2	561	76.8	692	64.9	806	53.1
	6 #16 + 2 #12	1433	193	88.5	310	90.9	432	88.1	597	76.1	739	64.4	860	52.8
	4 #20 + 2 #12	1483	191	89.9	310	92.2	434	89.2	605	76.8	749	64.9	872	53.1
	4 #16 + 4 #16	1608	122	76.2	310	78.3	488	75.7	666	66.4	809	56.5	933	46.2
	4 #20 + 2 #16	1659	160	89.9	310	92.2	454	89.2	639	76.8	794	64.9	923	53.1
	4 #20 + 4 #12	1709	164	93.4	309	95.5	460	91.9	646	78.9	797	66.5	927	54.2
	4 #25 + 0 #0	1963	215	118.9	307	120.8	435	114.7	639	96.4	807	80.3	949	65.2
	4 #20 + 4 #16	2061	112	96.2	308	98.0	500	94.0	711	80.6	878	67.7	1020	55.0
	4 #25 + 2 #12	2190	175	118.9	307	120.8	460	114.7	682	96.4	864	80.3	1015	65.2
	4 #25 + 2 #16	2366	145	118.9	307	120.8	480	114.7	716	96.4	909	80.3	1066	65.2
	4 #25 + 4 #12	2416	147	122.3	305	123.9	486	117.2	724	98.4	912	81.7	1070	66.2

For details of arrangement of longitudinal Reinforcement see NOTES at the end of Table G-2B.

Table G-2b **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u kN.m continued . . .**

P_u - M_u
M 20, Fe 415
b = 230 mm

Depth D mm	Steel N1 D1 + N2 D2 mm	Area mm²	Neutral Axis Factor $k_u = x_u/D$ 1.00 P_u	M_u	1.05 P_u	M_u	1.10 P_u	M_u	1.15 P_u	M_u	1.20 P_u	M_u	1.30 P_u	M_u
350	4 #16 + 0 #0	804	762	28.8	792	25.0	816	22.0	836	19.5	852	17.4	865	15.6
	4 #12 + 4 #12	905	814	24.5	843	21.1	865	18.4	883	16.2	898	14.4	910	12.8
	6 #12 + 2 #12	905	801	27.3	830	23.7	854	20.8	873	18.4	888	16.4	901	14.7
	4 #16 + 2 #12	1030	831	28.8	861	25.0	885	22.0	905	19.5	922	17.4	935	15.6
	6 #12 + 4 #12	1131	864	27.9	895	24.2	920	21.2	940	18.7	956	16.7	970	15.0
	6 #16 + 0 #0	1206	852	34.7	886	30.3	914	26.8	937	23.9	956	21.4	972	19.2
	4 #16 + 2 #16	1206	884	28.8	915	25.0	940	22.0	960	19.5	976	17.4	990	15.6
	4 #16 + 4 #12	1257	894	29.4	926	25.5	952	22.4	973	19.8	990	17.7	1004	15.9
	4 #20 + 0 #0	1257	865	34.8	900	30.4	928	26.9	952	23.9	971	21.4	987	19.3
	6 #16 + 2 #12	1433	921	34.7	956	30.3	984	26.8	1007	23.9	1026	21.4	1042	19.2
	4 #20 + 2 #12	1483	934	34.8	970	30.4	998	26.9	1021	23.9	1041	21.4	1057	19.3
	4 #16 + 4 #16	1608	997	29.8	1031	25.9	1059	22.7	1080	20.1	1097	18.0	1112	16.1
	4 #20 + 2 #16	1659	988	34.8	1024	30.4	1052	26.9	1076	23.9	1095	21.4	1112	19.3
	4 #20 + 4 #12	1709	998	35.3	1035	30.9	1065	27.3	1089	24.3	1109	21.7	1126	19.6
	4 #25 + 0 #0	1963	1033	42.7	1075	37.5	1110	33.3	1137	29.7	1161	26.6	1180	24.0
	4 #20 + 4 #16	2061	1101	35.8	1140	31.2	1172	27.6	1196	24.5	1216	22.0	1233	19.8
	4 #25 + 2 #12	2190	1102	42.7	1145	37.5	1179	33.3	1207	29.7	1231	26.6	1251	24.0
	4 #25 + 2 #16	2366	1156	42.7	1199	37.5	1233	33.3	1261	29.7	1285	26.6	1305	24.0
380	4 #12 + 0 #0	804	808	33.8	840	29.4	866	25.8	887	22.9	905	20.4	919	18.3
	4 #12 + 4 #12	905	862	28.7	892	24.8/	917	21.6	936	19.0	952	16.9	965	15.1
	6 #12 + 2 #12	905	848	32.0	880	27.8	904	24.3	925	21.5	942	19.2	956	17.2
	4 #16 + 2 #12	1030	877	33.8	910	29.4	936	25.8	957	22.9	975	20.4	989	18.3
	6 #12 + 4 #0	1131	911	32.7	944	28.3	971	24.8	992	22.0	1009	19.6	1024	17.6
	6 #16 + 0 #0	1206	897	40.6	933	35.5	963	31.4	988	28.0	1008	25.1	1025	22.6
	4 #16 + 2 #16	1206	931	33.8	964	29.4	990	25.8	1011	22.9	1029	20.4	1044	18.3
	4 #16 + 4 #12	1257	940	34.5	975	29.9	1002	26.3	1025	23.3	1043	20.8	1058	18.7
	4 #20 + 0 #0	1257	910	40.8	947	35.7	977	31.6	1002	28.1	1022	25.2	1039	22.7
	6 #16 + 2 #12	1433	966	40.6	1003	35.5	1033	31.4	1057	28.0	1078	25.1	1095	22.6
	4 #20 + 2 #12	1483	979	40.8	1016	35.7	1047	31.6	1072	28.1	1092	25.2	1110	22.7
	4 #16 + 4 #16	1608	1043	35.0	1079	30.4	1108	26.6	1131	23.6	1150	21.1	1165	18.9
	4 #20 + 2 #16	1659	1033	40.8	1070	35.7	1101	31.6	1126	28.1	1147	25.2	1164	22.7
	4 #20 + 4 #12	1709	1042	41.5	1082	36.3	1114	32.0	1139	28.5	1160	25.6	1178	23.1
	4 #25 + 0 #0	1963	1074	50.3	1119	44.3	1155	39.3	1185	35.2	1210	31.6	1231	28.5
	4 #20 + 4 #16	2061	1145	42.0	1186	36.7	1220	32.4	1246	28.9	1268	25.9	1286	23.3
	4 #25 + 2 #12	2190	1144	50.3	1188	44.3	1225	39.3	1255	35.2	1280	31.6	1301	28.5
	4 #25 + 2 #16	2366	1197	50.3	1242	44.3	1279	39.3	1309	35.2	1334	31.6	1356	28.5
	4 #25 + 4 #12	2416	1207	50.9	1254	44.8	1292	39.7	1323	35.5	1348	31.9	1369	28.8

For details of arrangement of longitudinal Reinforcement see NOTES at the end of Table G-2B.

Table G-2b **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u $kN.m$ continued . . .**

P_u - M_u
M 20, Fe 415
b = 230 mm

Depth D mm	Steel N1 D1 + N2 D2 mm	Area mm²	0.4 P_u	0.4 M_u	0.5 P_u	0.5 M_u	0.6 P_u	0.6 M_u	0.7 P_u	0.7 M_u	0.8 P_u	0.8 M_u	0.9 P_u	0.9 M_u
400	4 #16 + 0 #0	804	253	76.0	329	79.0	416	77.5	537	68.7	647	59.1	747	48.7
	4 #12 + 4 #12	905	194	62.7	329	65.5	459	64.6	588	58.5	700	50.9	800	42.1
	6 #12 + 2 #12	905	216	70.3	329	73.2	438	72.2	568	64.6	685	55.9	788	46.2
	4 #16 + 2 #12	1030	214	76.0	329	79.0	441	77.5	580	68.7	705	59.1	813	48.7
	6 #12 + 4 #12	1131	191	74.3	328	77.0	463	75.4	608	67.1	732	57.7	842	47.5
	6 #16 + 0 #0	1206	247	96.3	327	99.0	425	96.3	573	83.6	705	70.9	821	58.1
	4 #16 + 2 #16	1206	183	76.0	329	79.0	461	77.5	614	68.7	749	59.1	864	48.7
	4 #16 + 4 #12	1257	188	80.0	327	82.6	467	80.5	621	71.1	751	60.9	867	50.0
	4 #20 + 0 #0	1257	246	97.9	327	100.5	427	97.5	580	84.4	715	71.4	833	58.5
	6 #16 + 2 #12	1433	208	96.3	327	99.0	450	96.3	616	83.6	762	70.9	887	58.1
	4 #20 + 2 #12	1483	206	97.9	327	100.5	452	97.5	624	84.4	772	71.4	899	58.5
	4 #16 + 4 #16	1608	137	83.1	326	85.5	507	82.9	686	73.0	833	62.2	960	50.9
	4 #20 + 2 #16	1659	175	97.9	327	100.5	472	97.5	658	84.4	817	71.4	950	58.5
	4 #20 + 4 #12	1709	180	101.8	325	104.1	478	100.5	664	86.7	819	73.1	953	59.7
	4 #25 + 0 #0	1963	231	129.4	323	131.7	447	126.0	655	106.1	827	88.4	973	71.9
	4 #20 + 4 #16	2061	128	104.8	324	106.9	518	102.8	730	88.6	901	74.5	1046	60.6
	4 #25 + 2 #12	2190	191	129.4	323	131.7	473	126.0	699	106.1	884	88.4	1039	71.9
	4 #25 + 2 #16	2366	160	129.4	323	131.7	493	126.0	733	106.1	929	88.4	1090	71.9
	4 #25 + 4 #12	2416	164	133.1	322	135.0	498	128.8	740	108.3	932	90.1	1094	73.1
450	4 #12 + 4 #12	905	230	77.0	370	80.7	507	79.8	642	72.8	763	63.6	872	52.6
	6 #12 + 2 #12	905	251	85.8	371	89.7	486	88.8	621	80.2	747	69.6	859	57.6
	4 #16 + 2 #12	1030	249	92.5	370	96.5	488	95.2	633	85.2	766	73.5	883	60.7
	6 #12 + 0 #12	1131	227	90.7	369	94.3	511	92.6	660	83.2	793	71.8	912	59.1
	6 #16 + 0 #0	1206	283	116.6	368	120.2	470	117.6	622	103.2	746	87.8	890	72.1
	4 #16 + 2 #16	1206	218	92.7	370	96.5	508	95.2	667	85.2	812	73.5	834	60..7
	4 #16 + 4 #12	1257	225	97.5	369	101.0	514	98.9	672	88.1	812	75.6	936	62.2
	4 #20 + 0 #0	1257	282	118.6	368	122.1	473	119.3	629	104.3	773	88.6	901	72.7
	6 #16 + 2 #12	1433	244	116.6	368	120.2	496	117.6	666	103.2	821	87.8	956	72.1
	4 #20 + 2 #12	1483	242	118.6	368	122.1	498	119.3	673	104.3	830	88.6	967	72.7
	4 #16 + 4 #16	1608	175	101.2	368	104.5	554	101.8	737	90.4	893	77.3	1029	63.4
	4 #20 + 2 #16	1659	212	118.6	368	122.1	518	119.3	707	104.3	875	88.6	1018	72.7
	4 #20 + 4 #12	1709	218	123.3	367	126.5	524	122.9	712	107.2	877	90.7	1020	74.2
	4 #25 + 0 #0	1963	269	156.6	365	159.6	490	154.0	698	131.4	879	109.8	1036	89.6
	4 #20 + 4 #16	2061	168	127.0	366	129.9	563	125.7	777	109.4	958	92.3	1113	75.3
	4 #25 + 2 #12	2190	230	156.6	365	159.6	516	154.0	741	131.4	937	109.8	1102	89.6
	4 #25 + 2 #16	2366	199	156.6	365	159.6	536	154.0	775	131.4	982	109.8	1153	89.6

For details of arrangement of longitudinal Reinforcement see NOTES at the end of Table G-2B.

Table G-2b **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u kN.m continued . . .**

P_u - M_u
M 20, Fe 415
b = 230 mm

Depth D mm	Steel N1 D1 + N2 D2 mm mm	Area mm²	Neutral Axis Factor $k_u = x_u/D$ 1.00 P_u	M_u	1.05 P_u	M_u	1.10 P_u	M_u	1.15 P_u	M_u	1.20 P_u	M_u	1.30 P_u	M_u
400	4 #16 + 0 #0	804	840	37.3	873	32.4	900	28.4	922	25.2	940	22.5	955	20.2
	4 #12 + 4 #12	905	894	31.7	925	27.3	951	23.8	971	21.0	988	18.6	1001	16.7
	6 #12 + 2 #12	905	880	35.3	913	30.6	939	26.8	960	23.7	977	21.2	992	19.0
	4 #16 + 2 #12	1030	909	37.3	942	32.4	970	28.4	992	25.2	1010	22.5	1025	20.2
	6 #12 + 4 #12	1131	943	36.1	977	31.2	1005	27.4	1027	24.2	1045	21.6	1060	19.4
	6 #16 + 0 #0	1206	927	44.7	965	39.1	996	34.6	1022	30.8	1043	27.7	1060	25.0
	4 #16 + 2 #16	1206	962	37.3	996	32.4	1024	28.4	1046	25.2	1065	22.5	1080	20.2
	4 #16 + 4 #12	1257	971	38.0	1007	33.0	1036	29.0	1059	25.7	1078	23.0	1094	20.6
	4 #20 + 0 #0	1257	940	45.0	979	39.4	1010	34.8	1036	31.0	1057	27.9	1075	25.1
	6 #16 + 2 #12	1433	996	44.7	1035	39.1	1066	34.6	1091	30.8	1113	27.7	1130	25.0
	4 #20 + 2 #12	1483	1009	45.0	1048	39.4	1080	34.8	1105	31.0	1127	27.9	1145	25.1
	4 #16 + 4 #16	1608	1074	38.6	1111	33.5	1142	29.4	1166	26.0	1185	23.3	1201	20.9
	4 #20 + 2 #16	1659	1063	45.0	1102	39.4	1134	34.8	1160	31.0	1181	27.9	1199	25.1
	4 #20 + 4 #12	1709	1072	45.7	1113	40.0	1146	35.3	1173	31.5	1195	28.3	1213	25.5
	4 #25 + 0 #0	1963	1103	55.6	1149	48.9	1186	43.5	1217	38.9	1243	35.0	1265	31.6
	4 #20 + 4 #16	2061	1175	46.3	1217	41.5	1252	35.7	1280	31.8	1302	28.6	1321	25.8
	4 #25 + 2 #12	2190	1172	55.6	1218	48.9	1256	43.5	1287	38.9	1313	35.0	1335	31.6
	4 #25 + 2 #16	2366	1225	55.6	1272	48.9	1310	43.5	1342	38.9	1360	35.0	1390	31.6
	4 #25 + 4 #12	2416	1235	56.3	1283	49.5	1323	43.9	1355	39.3	1381	35.4	1403	32.0
450	4 #12 + 4 #12	905	974	39.7	1009	34.2	1037	29.8	1060	26.2	1078	23.3	1093	20.9
	6 #12 + 2 #12	905	960	44.0	996	38.1	1024	33.4	1047	29.6	1067	26.4	1083	23.7
	4 #16 + 2 #12	1030	988	46.5	1025	40.4	1055	35.5	1079	31.4	1099	28.1	1116	25.3
	6 #12 + 4 #12	1131	1022	44.9	1060	38.9	1090	34.1	1115	30.2	1134	26.9	1151	24.2
	6 #16 + 0 #0	1206	1004	55.6	1046	48.6	1080	43.0	1107	38.3	1130	24.4	1150	31.1
	4 #16 + 2 #16	1206	1042	46.5	1079	40.4	1109	35.5	1133	31.4	1154	28.1	1171	25.3
	4 #16 + 4 #12	1257	1050	47.4	1089	41.2	1120	36.1	1146	32.0	1167	28.6	1184	25.7
	4 #20 + 0 #0	1257	1017	56.1	1059	49.0	1093	43.3	1121	38.6	1144	34.7	1164	31.3
	6 #16 + 2 #12	1433	1074	55.6	1115	48.6	1149	43.0	1177	38.3	1200	34.4	1220	31.1
	4 #20 + 2 #12	1483	1086	56.1	1128	49.0	1163	43.3	1191	38.6	1214	34.7	1234	31.3
	4 #16 + 4 #16	1608	1152	48.1	1193	41.8	1226	36.6	1253	32.4	1274	29.0	1292	26.1
	4 #20 + 2 #16	1659	1140	56.1	1182	49.0	1217	43.3	1245	38.6	1269	34.7	1289	31.3
	4 #20 + 4 #12	1709	1148	57.0	1193	49.8	1229	44.0	1258	39.2	1282	35.2	1302	31.8
	4 #25 + 0 #0	1963	1175	69.4	1225	61.1	1266	54.3	1299	48.6	1328	43.9	1351	39.7
	4 #20 + 4 #16	2061	1250	57.7	1297	50.4	1334	44.5	1365	39.6	1389	35.6	1410	32.1
	4 #25 + 2 #12	2190	1244	69.4	1294	61.1	1335	54.3	1369	48.6	1398	43.9	1421	39.7
	4 #25 + 2 #16	2366	1298	69.4	1348	61.1	1389	54.3	1423	48.6	1452	43.9	1476	39.7

For details of arrangement of longitudinal Reinforcement see NOTES at the end of Table G-2B.

Table G-2b **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u $kN.m$ continued . . .**

P_u - M_u
M 20, Fe 415
b = 230 mm

Depth D mm	Steel N1 D1 + N2 D2 (mm, mm)	Area mm²	k_u = 0.4 P_u	0.4 M_u	0.5 P_u	0.5 M_u	0.6 P_u	0.6 M_u	0.7 P_u	0.7 M_u	0.8 P_u	0.8 M_u	0.9 P_u	0.9 M_u
450	4 #25 + 4 #12	2416	204	161.1	364	163.8	541	157.4	782	134.1	984	111.8	1156	90.9
	4 #25 + 4 #16	2768	153	164.5	362	167.0	581	160.1	847	136.2	1065	113.4	1249	92.0
	4 #25 + 2 #25	2945	97	156.6	365	159.6	601	154.0	888	131.4	1129	109.8	1322	89.6
500	4 #16 + 2 #12	1030	283	110.4	412	115.3	537	114.0	686	103.0	828	89.1	954	73.7
	6 #12 + 0 #12	1131	262	108.1	411	112.7	560	111.0	714	100.5	856	86.9	984	71.7
	6 #16 + 0 #0	1206	318	138.0	410	142.6	518	139.8	673	124.1	824	105.9	959	87.1
	4 #16 + 2 #16	1206	252	110.4	412	115.3	556	114.0	720	103.0	873	89.1	1006	73.7
	4 #16 + 4 #12	1257	260	116.1	410	120.6	562	118.4	725	106.4	874	91.6	1007	75.5
	4 #20 + 0 #0	1257	317	140.4	410	145.0	519	142.0	680	125.6	833	107.0	970	88.0
	6 #16 + 2 #12	1433	279	138.0	410	142.6	543	139.8	717	124.1	882	105.9	1025	87.1
	4 #20 + 2 #12	1483	277	140.4	410	145.0	545	142.0	723	125.6	890	107.0	1036	88.0
	4 #16 + 4 #16	1608	211	120.4	409	124.7	602	121.8	790	109.1	954	93.6	1099	76.8
	4 #20 + 2 #16	1659	247	140.4	410	145.0	565	142.0	757	125.6	935	107.0	1087	88.0
	4 #20 + 4 #12	1709	254	145.9	408	150.2	570	146.3	763	129.0	936	109.5	1089	89.7
	4 #25 + 0 #0	1963	306	184.8	406	188.8	534	183.2	743	158.0	935	132.5	1102	108.3
	4 #20 + 4 #16	2061	205	150.2	407	154.3	610	149.7	827	131.6	1017	111.4	1181	91.0
	4 #25 + 2 #12	2190	267	184.8	406	188.8	560	183.2	787	158.0	992	132.5	1167	108.3
	4 #25 + 2 #16	2366	236	184.8	406	188.8	579	183.2	821	158.0	1037	132.5	1219	108.3
	4 #25 + 4 #12	2416	242	190.1	405	193.8	585	187.3	827	161.2	1039	134.9	1221	109.9
	4 #25 + 4 #16	2768	193	194.2	404	197.7	625	190.5	892	163.8	1119	136.7	1313	111.2
	4 #25 + 2 #25	2945	135	184.8	406	188.8	644	183.2	933	158.0	1185	132.5	1387	108.3
	4 #32 + 0 #0	3217	287	262.9	401	266.0	563	255.4	857	215.0	1117	177.4	1336	143.9
	4 #25 + 4 #20	3220	129	199.5	403	202.6	676	194.6	975	167.0	1223	139.1	1432	112.8
530	4 #16 + 2 #12	1030	304	121.6	436	127.1	566	125.9	719	114.3	866	99.0	998	82.0
	6 #12 + 4 #12	1131	283	119.1	436	124.3	589	122.6	747	111.5	894	96.6	1027	79.8
	6 #16 + 0 #0	1206	339	151.3	435	156.6	547	153.8	705	137.3	861	117.3	1002	96.6
	4 #16 + 2 #16	1206	273	121.6	436	127.1	586	125.9	753	114.3	911	99.0	1049	82.0
	4 #16 + 4 #12	1257	281	127.7	435	132.9	591	130.7	758	118.0	911	101.7	1050	83.9
	4 #20 + 0 #0	1257	338	154.0	434	159.3	548	156.2	711	139.0	869	118.6	1012	97.6
	6 #16 + 2 #12	1433	299	151.3	435	156.6	572	153.8	748	137.3	918	117.3	1067	96.6
	4 #20 + 2 #12	1483	298	154.0	434	159.3	573	156.2	754	139.0	927	118.6	1078	97.6
	4 #16 + 4 #16	1608	233	132.5	434	137.4	631	134.4	822	120.9	992	103.9	1142	85.4
	4 #20 + 2 #16	1659	267	154.0	434	159.3	593	156.2	788	139.0	972	118.6	1129	97.6
	4 #20 + 4 #12	1709	275	160.0	433	165.0	599	160.9	793	142.7	972	121.3	1130	99.5
	4 #25 + 0 #0	1963	328	202.2	431	207.0	562	201.1	772	174.6	969	146.7	1142	120.0
	4 #20 + 4 #16	2061	227	164.7	432	169.5	639	164.6	858	145.5	1053	123.4	1222	100.9

For details of arrangement of longitudinal Reinforcement see NOTES at the end of Table G-2B.

Table G-2b **Allowable Combinations of Ultimate axial Compression P_u in *kN* and Ultimate Moment M_u *kN. m*continued . . .**

P_u - M_u
M 20, Fe 415
b = 230 mm

Depth D mm	Steel N1 D1 + N2 D2 mm mm	Area mm²	Neutral Axis Factor $k_u = x_u/D$ 1.00 P_u	M_u	1.05 P_u	M_u	1.10 P_u	M_u	1.15 P_u	M_u	1.20 P_u	M_u	1.30 P_u	M_u
450	4 #25 + 4 #12	2416	1307	70.3	1359	61.8	1401	54.9	1436	49.1	1465	44.3	1490	40.1
	4 #25 + 4 #16	2768	1409	70.9	1463	62.4	1507	55.3	1543	49.5	1573	44.7	1597	40.4
	4 #25 + 2 #25	2945	1475	69.4	1526	61.1	1568	54.3	1602	48.6	1631	43.9	1656	39.7
500	4 #16 + 2 #12	1030	1068	56.4	1108	49.0	1140	43.0	1167	38.1	1189	34.1	1207	30.7
	6 #12 + 4 #12	1131	1102	54.6	1143	47.3	1176	41.4	1203	36.6	1224	32.6	1242	29.3
	6 #16 + 0 #0	1206	1083	67.2	1128	58.8	1164	51.9	1194	46.3	1219	41.5	1240	37.6
	4 #16 + 2 #16	1206	1121	56.4	1162	49.0	1195	43.0	1221	38.1	1243	34.1	1262	30.7
	4 #16 + 2 #12	1257	1129	57.5	1172	50.0	1206	53.8	1234	38.8	1256	34.7	1275	31.2
	4 #20 + 0 #0	1257	1095	67.9	1140	59.4	1177	52.4	1207	46.7	1232	42.0	1254	38.0
	6 #16 + 2 #12	1433	1152	67.2	1197	58.8	1234	51.9	1264	46.3	1289	41.5	1310	37.6
	4 #20 + 2 #12	1483	1164	67.9	1210	59.4	1247	52.4	1277	46.7	1302	42.0	1324	38.0
	4 #16 + 4 #16	1608	1231	58.4	1275	50.7	1311	44.4	1340	39.3	1363	35.2	1382	31.7
	4 #20 + 2 #16	1659	1218	67.9	1264	59.4	1301	52.4	1331	46.7	1357	42.0	1378	38.0
	4 #20 + 4 #12	1709	1226	69.0	1274	60.3	1312	53.2	1344	47.4	1370	42.6	1392	38.5
	4 #25 + 0 #0	1963	1249	84.0	1303	74.0	1347	65.7	1383	58.9	1413	53.1	1439	48.2
	4 #20 + 4 #16	2061	1327	69.8	1377	61.0	1417	53.8	1451	47.9	1477	43.0	1499	38.9
	4 #25 + 2 #12	2190	1319	84.0	1372	74.0	1416	65.7	1453	58.9	1483	53.1	1509	48.2
	4 #25 + 2 #16	2366	1372	84.0	1426	74.0	1470	65.7	1507	58.9	1538	53.1	1564	48.2
	4 #25 + 4 #12	2416	1381	85.1	1437	74.8	1482	66.5	1520	59.5	1551	53.7	1577	48.7
	4 #25 + 4 #16	2768	1483	85.9	1540	75.5	1587	67.0	1626	60.0	1658	54.1	1685	49.1
	4 #25 + 2 #25	2945	1549	84.0	1604	74.0	1649	65.7	1686	58.9	1717	53.1	1743	48.2
	4 #32 + 0 #0	3217	1524	112.5	1592	99.7	1648	89.1	1695	80.2	1734	72.7	1768	66.1
	4 #25 + 4 #20	3220	1614	86.9	1674	76.4	1723	67.8	1763	60.6	1796	54.7	1823	49.6
530	4 #16 + 2 #12	1030	1116	62.8	1158	54.5	1192	47.8	1220	42.4	1243	37.9	1262	34.1
	6 #12 + 0 #0	1131	1150	60.7	1193	52.6	1227	46.0	1256	40.7	1278	36.3	1297	32.6
	6 #16 + 0 #0	1206	1130	74.5	1177	65.2	1215	57.5	1246	51.3	1272	46.0	1294	41.6
	4 #16 + 2 #16	1206	1170	62.8	1212	64.5	1246	47.8	1274	42.4	1297	37.9	1316	34.1
	4 #16 + 4 #12	1257	1177	64.0	1222	55.5	1257	48.7	1286	43.1	1310	38.5	1330	34.7
	4 #20 + 0 #0	1257	1142	75.3	1190	65.9	1228	58.2	1259	51.8	1286	46.6	1308	42.1
	6 #16 + 2 #12	1433	1199	74.5	1247	65.2	1285	57.5	1316	51.3	1342	46.0	1364	41.6
	4 #20 + 2 #12	1483	1211	75.3	1259	65.9	1298	58.2	1329	51.8	1356	46.6	1378	42.1
	4 #16 + 4 #16	1608	1278	64.9	1325	56.3	1362	49.4	1393	43.7	1417	39.0	1437	35.1
	4 #20 + 2 #16	1659	1265	75.3	1313	65.9	1352	58.2	1383	51.8	1410	46.6	1432	42.1
	4 #20 + 4 #12	1709	1273	76.5	1323	66.9	1363	59.0	1396	52.6	1423	47.2	1446	42.7
	4 #25 + 0 #0	1963	1295	93.2	1351	82.0	1396	72.9	1434	65.3	1465	58.9	1492	53.5
	4 #20 + 4 #16	2061	1374	77.5	1426	67.7	1468	59.7	1502	53.1	1530	47.7	1553	43.1

For details of arrangement of longitudinal Reinforcement see NOTES at the end of Table G-2B.

Table G-2b **Allowable Combinations of Ultimate axial Compression P_u in *kN* and Ultimate Moment M_u *kN.m* continued . . .**

P_u - M_u
M 20, Fe 415
b = 230 mm

Depth D mm	Steel N1 D1 + N2 D2 mm mm	Area mm²	Neutral Axis Factor $k_u = x_u/D$ 0.4 P_u	0.4 M_u	0.5 P_u	0.5 M_u	0.6 P_u	0.6 M_u	0.7 P_u	0.7 M_u	0.8 P_u	0.8 M_u	0.9 P_u	0.9 M_u
530	4 #25 + 2 #12	2190	288	202.2	431	207.0	587	201.1	815	174.6	1027	146.7	1207	120.0
	4 #25 + 2 #16	2366	257	202.2	431	207.0	607	201.1	849	174.6	1071	146.7	1259	120.0
	4 #25 + 4 #12	2416	265	208.0	430	212.4	613	205.6	855	178.2	1073	149.3	1260	121.8
	4 #25 + 4 #16	2768	215	212.5	429	216.7	653	209.1	919	180.9	1153	151.4	1353	123.2
	4 #25 + 2 #25	2945	156	202.2	431	207.0	672	201.1	961	174.6	1219	146.7	1427	120.0
	4 #32 + 0 #0	3217	310	287.1	426	290.9	587	280.2	881	237.4	1147	196.3	1372	159.4
	4 #25 + 4 #20	3220	152	218.3	427	222.1	704	213.6	1003	184.5	1257	154.0	1471	125.0
550	4 #16 + 2 #12	1030	317	129.3	453	135.2	585	134.1	741	122.0	891	105.8	1026	87.7
	6 #12 + 4 #12	1131	297	126.6	452	132.3	609	130.6	769	119.1	919	103.3	1056	85.4
	6 #16 + 0 #0	1206	353	160.4	451	166.1	566	163.3	726	146.3	885	125.1	1030	103.1
	4 #16 + 2 #16	1206	286	129.3	453	135.2	605	134.1	775	122.0	936	105.8	1078	87.7
	4 #16 + 4 #12	1257	295	135.7	452	141.3	611	139.1	779	126.0	937	108.7	1079	89.7
	4 #20 + 0 #0	1257	352	163.3	451	169.0	567	166.0	732	148.1	894	126.5	1040	104.2
	6 #16 + 2 #12	1433	313	160.4	451	166.1	591	163.3	769	146.3	943	125.1	1096	103.1
	4 #20 + 2 #12	1483	312	163.3	451	169.0	593	166.0	775	148.1	951	126.5	1106	104.2
	4 #16 + 4 #16	1608	247	140.7	451	146.1	650	143.1	843	129.0	1017	111.0	1170	91.3
	4 #20 + 2 #16	1659	281	163.3	451	169.0	612	166.0	809	148.1	996	126.5	1157	104.2
	4 #20 + 4 #12	1709	290	169.7	450	175.0	618	171.0	814	152.0	997	129.4	1158	106.2
	4 #25 + 0 #0	1963	342	214.0	448	219.3	581	213.3	791	186.0	992	156.5	1169	128.0
	4 #20 + 4 #16	2061	241	174.6	449	179.7	658	174.8	878	155.1	1077	131.6	1250	107.7
	4 #25 + 2 #12	2190	302	214.0	448	219.3	606	213.3	835	186.0	1050	156.5	1234	128.0
	4 #25 + 2 #16	2366	272	214.0	448	219.3	626	213.3	869	189.0	1095	156.5	1286	128.0
	4 #25 + 4 #12	2416	279	220.1	446	225.1	632	218.0	874	189.7	1096	159.2	1287	129.9
	4 #25 + 4 #16	2768	230	224.9	445	229.6	671	221.7	938	192.7	1176	161.3	1379	131.4
	4 #25 + 2 #25	2945	170	214.0	448	219.3	691	213.3	981	186.0	1242	156.5	1454	128.0
	4 #32 + 0 #0	3217	325	303.5	442	307.8	605	296.6	898	252.6	1168	209.1	1397	169.9
	4 #25 + 4 #20	3220	167	231.0	444	235.4	722	226.5	1021	196.4	1279	164.1	1498	133.3
600	6 #12 + 4 #12	1131	332	146.3	494	153.0	658	151.5	824	138.9	983	120.8	1128	99.9
	6 #16 + 0 #0	1206	387	184.0	493	190.6	614	188.1	779	169.8	948	145.6	1101	120.1
	4 #16 + 2 #16	1206	320	149.3	495	156.2	654	155.4	830	142.3	1000	123.7	1150	102.6
	4 #16 + 4 #12	1257	330	156.5	493	163.2	660	161.1	834	146.8	1000	127.0	1150	104.9
	4 #20 + 0 #0	1257	386	187.4	493	194.0	615	191.2	785	172.0	956	147.3	1111	121.4
	6 #16 + 2 #12	1433	347	184.0	493	190.6	639	188.1	823	169.8	1005	145.6	1167	120.1
	4 #20 + 2 #12	1483	346	187.4	493	194.0	641	191.2	828	172.0	1013	147.3	1177	121.4
	4 #16 + 4 #16	1608	283	162.2	492	168.6	699	165.6	898	150.3	1080	129.6	1242	106.7
	4 #20 + 2 #16	1659	316	187.4	493	194.0	661	191.2	862	172.0	1058	147.3	1298	121.4
	4 #20 + 4 #12	1709	325	194.5	491	200.9	666	196.9	867	176.4	1058	150.5	1229	123.7
	4 #25 + 0 #0	1963	377	244.4	489	250.8	628	244.6	841	215.3	1051	181.6	1237	148.8

For details of arrangement of longitudinal Reinforcement see NOTES at the end of Table G-2B.

Table G-2b **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u kN.m continued . . .**

P_u - M_u
M 20, Fe 415
b = 230 mm

Depth D mm	Steel N1 D1 + N2 D2 mm mm	Area mm²	Neutral Axis Factor $k_u = x_u/D$ 1.00 P_u	1.00 M_u	1.05 P_u	1.05 M_u	1.10 P_u	1.10 M_u	1.15 P_u	1.15 M_u	1.20 P_u	1.20 M_u	1.30 P_u	1.30 M_u
530	4 #25 − 2 #12	2190	1364	93.2	1420	82.0	1466	72.9	1503	65.3	1535	58.9	1562	53.5
	4 #25 + 2 #16	2366	1418	93.2	1474	82.0	1520	72.9	1558	65.3	1589	58.9	1617	53.5
	4 #25 + 4 #12	2416	1426	94.4	1484	83.0	1531	73.7	1570	66.0	1603	59.5	1630	54.0
	4 #25 + 4 #16	2768	1528	95.3	1588	83.7	1636	74.3	1677	66.5	1710	60.0	1738	54.5
	4 #25 + 2 #25	2945	1595	93.2	1652	82.0	1698	72.9	1737	65.3	1769	58.9	1796	53.5
	4 #32 + 0 #0	3217	1567	124.7	1637	110.5	1695	98.8	1743	88.9	1784	80.6	1819	73.4
	4 #25 + 4 #20	3220	1658	96.4	1721	84.7	1772	75.1	1814	67.2	1847	60.6	1876	55.0
550	4 #16 + 2 #12	1030	1148	67.1	1192	58.3	1227	51.1	1255	45.3	1279	40.5	1298	36.4
	6 #12 + 4 #12	1131	1182	64.9	1227	56.2	1262	49.2	1291	43.5	1314	38.8	1334	34.8
	6 #16 + 0 #0	1206	1162	79.6	1210	69.5	1249	61.4	1281	54.7	1308	49.1	1330	44.4
	4 #16 + 2 #16	1206	1202	67.1	1246	58.3	1281	51.1	1309	45.3	1333	40.5	1353	36.4
	4 #16 + 2 #12	1257	1209	68.4	1255	59.4	1292	52.1	1322	46.1	1346	41.2	1366	37.0
	4 #20 + 0 #0	1257	1174	80.4	1223	70.3	1262	62.1	1294	55.3	1321	49.7	1344	44.9
	6 #16 + 2 #12	1433	1231	79.6	1280	69.5	1319	61.4	1351	54.7	1378	49.1	1400	44.4
	4 #20 + 2 #12	1483	1243	80.4	1292	70.3	1332	62.1	1364	55.3	1391	49.7	1414	44.9
	4 #16 + 4 #16	1608	1310	69.4	1358	60.2	1396	52.8	1428	46.7	1453	41.7	1474	37.5
	4 #20 + 2 #16	1659	1297	80.4	1346	70.3	1386	62.1	1418	55.3	1446	49.7	1469	44.9
	4 #20 + 4 #12	1709	1304	81.7	1355	71.4	1397	63.0	1431	56.1	1459	50.4	1482	45.5
	4 #25 + 0 #0	1963	1326	99.5	1383	87.5	1429	77.8	1468	69.7	1500	62.8	1527	57.0
	4 #20 + 4 #16	2061	1405	82.7	1459	72.2	1502	63.7	1537	56.7	1566	50.9	1589	46.0
	4 #25 + 2 #12	2190	1395	99.5	1452	87.5	1499	77.8	1537	69.7	1570	62.8	1597	57.0
	4 #25 + 2 #16	2366	1448	99.5	1506	87.5	1553	77.8	1592	69.7	1624	62.8	1652	57.0
	4 #25 + 4 #12	2416	1456	100.7	1516	88.6	1564	78.6	1604	70.4	1637	63.5	1665	57.6
	4 #25 + 4 #16	2768	1558	101.7	1619	89.4	1669	79.3	1711	71.0	1744	64.0	1773	58.1
	4 #25 + 2 #25	2945	1625	99.5	1684	87.5	1731	77.8	1771	69.7	1804	62.8	1832	57.0
	4 #32 + 0 #0	3217	1595	133.0	1667	117.9	1726	105.3	1775	94.9	1817	86.0	1853	78.4
	4 #25 + 4 #20	3220	1688	102.9	1752	90.4	1804	80.2	1848	71.7	1882	64.7	1911	58.7
600	6 #12 + 4 #12	1131	1263	76.0	1311	65.8	1349	57.5	1380	50.8	1405	45.3	1426	40.7
	6 #16 + 0 #0	1206	1242	92.7	1293	80.9	1335	71.4	1369	63.6	1397	57.1	1421	51.6
	4 #16 + 2 #16	1206	1283	78.5	1330	68.1	1367	59.8	1398	52.9	1423	47.3	1445	42.5
	4 #16 + 4 #12	1257	1290	80.0	1339	69.4	1378	60.8	1410	53.8	1436	48.0	1458	43.2
	4 #20 + 0 #0	1257	1253	93.7	1306	81.9	1347	72.3	1382	64.4	1410	57.8	1435	52.2
	6 #16 + 2 #12	1433	1311	92.7	1363	80.9	1405	71.4	1439	63.6	1467	57.1	1491	51.6
	4 #20 + 2 #12	1483	1323	93.7	1375	81.9	1417	72.3	1452	64.4	1480	57.8	1505	52.2
	4 #16 + 4 #16	1608	1391	81.2	1442	70.4	1482	61.7	1516	54.5	1543	48.7	1565	43.8
	4 #20 + 2 #16	1659	1376	93.7	1429	81.9	1471	72.3	1506	64.4	1535	57.8	1559	52.2
	4 #20 + 4 #12	1709	1383	95.2	1438	83.1	1482	73.3	1518	65.3	1548	58.6	1573	52.9
	4 #25 + 0 #0	1963	1403	115.7	1463	101.8	1513	90.4	1553	81.0	1587	73.0	1616	66.3

For details of arrangement of longitudinal Reinforcement see NOTES at the end of Table G-2B.

Table G-2b **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u $kN.m$ continued . . .**

P_u - M_u
M 20, Fe 415
b = 230 mm

Depth D mm	Steel N1 D1 + N2 D2 mm mm	Area mm²	0.4 P_u	0.4 M_u	0.5 P_u	0.5 M_u	0.6 P_u	0.6 M_u	0.7 P_u	0.7 M_u	0.8 P_u	0.8 M_u	0.9 P_u	0.9 M_u
			Neutral Axis Factor $k_u = x_u/D$											
600	4 #20 + 4 #16	2061	277	200.1	490	206.2	706	201.3	930	179.8	1138	153.1	1320	125.5
	4 #25 + 2 #12	2190	338	244.4	489	250.8	653	244.6	884	215.3	1109	181.6	1303	148.8
	4 #25 + 2 #16	2366	307	244.4	489	250.8	673	244.6	918	215.3	1153	181.6	1354	148.8
	4 #25 + 4 #12	2416	315	251.3	488	257.4	679	250.2	923	219.6	1154	184.7	1355	150.9
	4 #25 + 4 #16	2768	267	256.7	487	262.5	718	254.3	987	222.9	1234	187.2	1447	152.6
	4 #25 + 2 #25	2945	205	244.4	489	250.8	738	244.6	1030	215.3	1301	181.6	1523	148.8
	4 #32 + 0 #0	3217	362	345.0	484	350.9	650	338.7	941	291.6	1221	242.0	1461	196.9
	4 #25 + 4 #20	3220	205	26.7	485	269.1	769	259.7	1070	227.2	1337	190.3	1565	154.8
650	6 #16 + 0 #0	1206	421	208.7	534	216.4	663	214.2	833	194.5	1011	167.2	1173	138.1
	4 #16 + 2 #16	1206	354	170.3	536	178.4	703	177.9	885	163.9	1064	142.7	1223	118.5
	4 #16 + 4 #12	1257	364	178.4	535	186.3	709	184.4	889	168.8	1064	146.4	1223	121.1
	4 #20 + 0 #0	1257	420	212.5	534	220.3	664	217.7	838	197.1	1018	169.2	1182	139.7
	6 #16 + 2 #12	1433	381	208.7	564	216.4	688	214.2	877	194.5	1068	167.2	1238	138.1
	4 #20 + 2 #12	1483	381	212.5	534	220.3	689	217.7	882	197.1	1076	169.2	1248	139.7
	4 #16 + 4 #16	1608	318	184.7	533	192.3	748	189.4	952	172.7	1143	149.3	1314	123.1
	4 #20 + 2 #16	1659	350	212.5	534	220.3	709	217.7	916	197.1	1121	169.2	1299	139.7
	4 #20 + 4 #12	1709	360	220.5	533	228.0	715	224.0	920	202.0	1121	172.8	1300	142.2
	4 #25 + 0 #0	1963	412	275.9	531	283.2	675	277.1	891	245.9	1111	207.9	1306	170.6
	4 #20 + 4 #16	2061	313	226.7	531	233.9	754	229.0	984	205.8	1200	175.6	1391	144.2
	4 #25 + 2 #12	2190	373	275.9	531	283.2	700	277.1	935	245.9	1169	207.9	1372	170.6
	4 #25 + 2 #16	2366	342	275.9	531	283.2	720	277.1	969	245.9	1214	207.9	1423	170.6
	4 #25 + 4 #12	2416	351	283.6	529	290.7	726	283.3	973	250.7	1214	211.5	1424	173.1
	4 #25 + 4 #16	2768	304	289.7	528	296.5	766	288.1	1037	254.4	1294	214.2	1515	175.0
	4 #25 + 2 #25	2945	240	275.9	531	283.2	785	277.1	1081	245.9	1361	207.9	1592	170.6
	4 #32 + 0 #0	3217	398	387.8	525	394.4	695	382.1	986	332.0	1276	276.2	1526	225.1
	4 #25 + 4 #20	3220	242	297.5	527	303.9	817	294.2	1119	259.2	1396	217.7	1633	177.4
680	4 #16 + 4 #12	1257	385	192.0	560	200.7	738	199.0	922	182.7	1105	158.6	1266	131.3
	4 #20 + 0 #0	1257	440	228.1	559	236.6	693	234.2	871	212.8	1056	182.9	1225	151.1
	6 #16 + 2 #12	1433	401	223.9	559	232.5	717	230.4	910	210.0	1106	180.7	1282	149.4
	4 #20 + 2 #12	1483	401	228.1	559	236.6	718	234.2	915	212.8	1114	182.9	1291	151.1
	4 #16 + 4 #16	1608	338	198.7	558	207.2	778	204.3	985	186.8	1181	161.6	1357	133.4
	4 #20 + 2 #16	1659	370	228.1	559	236.6	738	264.2	949	212.8	1159	182.9	1342	151.1
	4 #20 + 4 #12	1709	380	236.6	558	244.8	744	240.9	952	218.0	1158	186.7	1343	153.8
	4 #25 + 0 #0	1963	433	295.3	556	303.3	704	297.3	922	264.9	1148	224.3	1348	184.2
	4 #20 + 4 #16	2061	334	243.2	556	251.2	784	246.2	1016	222.0	1238	189.7	1434	155.9
	4 #25 + 2 #12	2190	393	294.3	556	303.3	729	297.3	966	264.9	1205	224.3	1414	184.2

For details of arrangement of longitudinal Reinforcement see NOTES at the end of Table G-2B.

Table G-2b **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u kN.m continued . . .**

P_u - M_u
M 20, Fe 415
b = 230 mm

Depth D mm	Steel N1 D1 + N2 D2 mm mm	Area mm²	1.00 P_u	1.00 M_u	1.05 P_u	1.05 M_u	1.10 P_u	1.10 M_u	1.15 P_u	1.15 M_u	1.20 P_u	1.20 M_u	1.30 P_u	1.30 M_u
600	4 #20 + 4 #16	2061	1484	96.3	1541	84.1	1587	74.2	1624	66.0	1655	59.2	1680	53.5
	4 #25 + 2 #12	2190	1472	115.7	1533	101.8	1582	90.4	1623	81.0	1657	73.0	1686	66.3
	4 #25 + 2 #16	2366	1526	115.7	1587	101.8	1636	90.4	1677	81.0	1712	73.0	1741	66.3
	4 #25 + 4 #12	2416	1533	117.1	1596	103.0	1647	91.4	1690	81.8	1725	73.8	1754	66.9
	4 #25 + 4 #16	2768	1634	118.2	1699	103.9	1752	92.2	1796	82.5	1832	74.4	1862	67.5
	4 #25 + 2 #25	2945	1703	115.7	1764	101.8	1815	90.4	1856	81.0	1891	73.0	1921	66.3
	4 #32 + 0 #0	3217	1668	154.4	1744	136.8	1806	122.2	1858	110.1	1902	99.8	1939	90.9
	4 #25 + 4 #20	3220	1765	119.6	1832	105.1	1887	93.2	1932	83.3	1969	75.1	2000	68.2
650	6 #16 + 0 #0	1206	1322	106.5	1377	93.0	1421	82.0	1457	73.0	1487	65.5	1512	59.1
	4 #16 + 2 #16	1206	1364	90.7	1414	78.6	1454	68.9	1487	61.0	1514	54.4	1536	48.9
	4 #16 + 2 #12	1257	1371	92.3	1423	80.0	1465	70.1	1499	62.0	1527	55.3	1550	49.8
	4 #20 + 0 #0	1257	1334	107.8	1389	94.1	1433	83.0	1470	73.9	1500	66.3	1526	59.9
	6 #16 + 2 #12	1433	1391	106.5	1446	93.0	1491	82.0	1527	73.0	1557	65.5	1582	59.1
	4 #20 + 2 #12	1483	1403	107.8	1458	94.1	1503	83.0	1540	73.9	1570	66.3	1596	59.9
	4 #16 + 4 #16	1608	1471	93.7	1526	81.1	1569	71.1	1604	62.8	1633	56.0	1657	50.4
	4 #20 + 2 #16	1659	1456	107.8	1512	94.1	1557	83.0	1594	73.9	1625	66.3	1650	59.9
	4 #20 + 4 #12	1709	1463	109.4	1521	95.5	1568	84.2	1606	74.9	1637	67.2	1664	60.7
	4 #25 + 0 #0	1963	1481	132.7	1545	116.7	1597	103.6	1640	92.7	1676	86.6	1706	75.9
	4 #20 + 4 #16	2061	1564	110.7	1624	96.6	1672	85.1	1712	75.7	1744	67.9	1771	61.4
	4 #25 + 2 #12	2190	1550	132.7	1614	116.7	1666	103.6	1709	92.7	1746	83.6	1776	75.9
	4 #25 + 2 #16	2366	1604	132.7	1668	116.7	1721	103.6	1764	92.7	1800	83.6	1831	75.9
	4 #25 + 4 #12	2416	1611	134.3	1677	118.0	1731	104.7	1776	93.7	1813	84.5	1844	76.6
	4 #25 + 4 #16	2768	1712	135.5	1780	119.0	1836	105.6	1882	94.4	1920	85.1	1951	77.2
	4 #25 + 2 #25	2945	1781	132.7	1846	116.7	1899	103.6	1943	92.7	1979	83.6	2010	75.9
	4 #32 + 0 #0	3217	1743	176.6	1822	156.4	1887	139.7	1941	125.8	1987	114.0	2026	103.9
	4 #25 + 4 #20	3220	1842	137.1	1913	120.4	1970	106.7	2018	95.4	2057	86.0	2089	78.0
680	4 #16 + 4 #12	1257	1419	100.1	1474	86.7	1517	75.9	1552	67.2	1581	59.9	1605	53.8
	4 #20 + 0 #0	1257	1382	116.5	1439	101.7	1485	89.7	1523	79.9	1554	71.6	1580	64.7
	6 #16 + 2 #12	1433	1440	115.2	1497	100.5	1542	88.6	1580	78.8	1611	70.7	1637	63.8
	4 #20 + 2 #12	1483	1451	116.5	1509	101.7	1555	89.7	1593	79.9	1624	71.6	1651	64.7
	4 #16 + 4 #16	1608	1520	101.5	1576	87.9	1621	77.0	1658	68.0	1688	60.6	1712	54.5
	4 #20 + 2 #16	1659	1505	116.5	1562	101.7	1609	89.7	1647	79.9	1679	71.6	1705	64.7
	4 #20 + 4 #12	1709	1511	118.3	1571	103.2	1619	91.0	1659	80.9	1691	72.6	1718	65.6
	4 #25 + 0 #0	1963	1528	143.2	1594	125.9	1648	111.7	1692	100.0	1729	90.2	1760	81.8
	4 #20 + 4 #16	2061	1612	119.7	1674	104.4	1724	92.0	1764	81.8	1798	73.3	1826	66.2
	4 #25 + 2 #12	2190	1597	143.2	1664	125.9	1717	111.7	1762	100.0	1799	90.2	1830	81.8

For details of arrangement of longitudinal Reinforcement see NOTES at the end of Table G-2B.

Table G-2b **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u in kN continued . . .**

P_u - M_u
M 20, Fe 415
b = 230 mm

Depth D mm	Steel N1 D1 + N2 D2 mm	Area mm^2	0.4 P_u	0.4 M_u	0.5 P_u	0.5 M_u	0.6 P_u	0.6 M_u	0.7 P_u	0.7 M_u	0.8 P_u	0.8 M_u	0.9 P_u	0.9 M_u
			Neutral Axis Factor $k_u = x_u/D$											
680	4 #25 + 2 #16	2366	363	295.3	556	303.3	749	297.3	1000	264.9	1250	224.3	1465	184.2
	4 #25 + 4 #12	2416	372	303.6	554	311.2	755	303.9	1004	270.0	1250	228.0	1465	186.8
	4 #25 + 4 #16	2768	325	310.0	553	317.4	794	309.0	1068	274.0	1330	231.0	1557	188.8
	4 #25 + 2 #25	2945	261	295.3	556	303.3	814	297.3	1112	264.9	1398	224.3	1634	184.2
	4 #32 + 0 #0	3217	419	413.9	550	421.1	723	408.8	1015	356.9	1310	297.3	1566	242.5
	4 #25 + 4 #20	3220	264	318.3	552	325.4	845	315.5	1150	279.0	1432	234.7	1674	191.5
700	6 #16 + 2 #12	1433	415	234.3	576	243.5	737	241.5	932	220.5	1132	189.9	1310	157.1
	4 #20 + 2 #12	1483	414	238.6	576	247.7	738	245.4	936	223.5	1139	192.2	1320	158.9
	4 #16 + 4 #16	1608	352	208.3	575	217.3	797	214.5	1008	196.4	1207	170.1	1387	140.4
	4 #20 + 2 #16	1659	384	238.6	576	247.7	758	245.4	971	223.5	1184	192.2	1371	158.9
	4 #20 + 4 #12	1709	394	247.5	574	256.2	763	252.5	974	228.9	1184	196.2	1371	161.7
	4 #25 + 0 #0	1963	447	308.5	572	316.9	723	311.0	943	277.8	1172	235.4	1376	193.4
	4 #20 + 4 #16	2061	348	254.4	573	262.9	803	258.0	1037	233.1	1263	199.3	1462	163.9
	4 #25 + 2 #12	2190	407	308.5	572	316.9	748	311.0	987	277.8	1230	235.4	1442	193.4
	4 #25 + 2 #16	2366	377	308.5	572	316.9	768	311.0	1021	277.8	1274	235.4	1493	193.4
	4 #25 + 4 #12	2416	386	317.1	571	325.2	774	317.8	1025	283.1	1274	239.3	1493	196.2
	4 #25 + 4 #16	2768	339	323.8	570	331.6	813	323.2	1089	287.2	1354	242.4	1585	198.3
	4 #25 + 2 #25	2945	275	308.5	572	316.9	833	311.0	1133	277.8	1422	235.4	1662	193.4
	4 #32 + 0 #0	3217	434	431.6	567	439.2	741	426.8	1034	373.8	1333	311.6	1592	254.3
	4 #25 + 4 #20	3220	279	332.4	568	339.9	864	330.0	1170	292.5	1456	246.3	1702	201.0

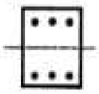
Notes : Arrangement of Longitudinal Steel.

N1 - D1 Represent Total Number of bars in first and Last raw having Same diameter.

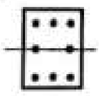
N2 - D2 Represent Total Number of bars BETWEEN first and Last row

For Example

6 #20 + 0 -#0 (i.e.N1 = 6 and *N2* = 0), means 3 No of 20*mm* diameter bars in first row and 3 No. in the last row.
Thus, total no of rows = 2



6 #20 + 2 -#12 (i.e.N1 = 6 and *N2* = 2), means 3 No of 20*mm* diameter bars in first and row and 3 No. in the last row and 2 no of 12*mm* bars at mid-depth
Thus, total no of rows = 3



6 #20 + 4 -#12 (i.e.N1 = 6 and *N2* = 4), means 3 No of 20*mm* diameter bars in first row and 3 No. in the last row and 2 no of 12*mm* bars in 2nd row and 2 No of 12*mm* in3rd row equispaced.
Thus, total no of rows = 4

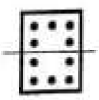


Table G-2b **Allowable Combinations of Ultimate axial Compression P_u in kN and Ultimate Moment M_u in kN continued . . .**

P_u - M_u
M 20, Fe 415
b = 230 mm

Depth D mm	Steel N1 D1 + N2 D2 mm mm	Area mm²	1.00 P_u	1.00 M_u	1.05 P_u	1.05 M_u	1.10 P_u	1.10 M_u	1.15 P_u	1.15 M_u	1.20 P_u	1.20 M_u	1.30 P_u	1.30 M_u
			Neutral Axis Factor $k_u = x_u/D$											
680	4 #25 + 2 #16	2366	1651	143.2	1718	125.9	1771	111.7	1816	100.0	1853	90.2	1885	81.8
	4 #25 + 4 #12	2416	1658	144.9	1726	127.3	1782	113.0	1828	101.0	1866	91.1	1898	82.6
	4 #25 + 4 #16	2768	1759	146.3	1829	128.5	1886	113.9	1934	101.9	1973	91.8	2005	83.3
	4 #25 + 2 #25	2945	1828	143.2	1895	125.9	1950	111.7	1995	100.0	2032	90.2	2064	81.8
	4 #32 + 0 #0	3217	1788	190.3	1870	168.5	1936	150.5	1992	135.5	2039	122.8	2079	111.9
	4 #25 + 4 #20	3220	1889	148.0	1961	129.9	2021	115.1	2070	102.9	2110	92.7	2143	84.1
700	6 #16 + 2 #12	1433	1472	121.1	1530	105.6	1577	93.1	1615	82.8	1647	74.3	1674	67.0
	4 #20 + 2 #12	1483	1483	122.5	1542	106.9	1589	94.3	1628	83.9	1660	75.3	1687	68.0
	4 #16 + 4 #16	1608	1553	106.8	1610	92.5	1656	81.0	1693	71.6	1724	63.8	1749	57.3
	4 #20 + 2 #16	1659	1537	122.5	1596	106.9	1643	94.3	1682	83.9	1715	75.3	1742	68.0
	4 #20 + 4 #12	1709	1543	124.4	1605	108.5	1654	95.6	1694	85.0	1727	76.2	1755	68.8
	4 #25 + 0 #0	1963	1560	150.4	1627	132.2	1682	117.3	1727	105.0	1764	94.6	1796	85.8
	4 #20 + 4 #16	2061	1644	125.8	1707	109.7	1758	96.7	1800	85.9	1834	77.0	1862	69.5
	4 #25 + 2 #12	2190	1629	150.4	1697	132.2	1751	117.3	1796	105.0	1834	94.6	1866	85.8
	4 #25 + 2 #16	2366	1683	150.4	1750	132.2	1805	117.3	1851	105.0	1889	94.6	1921	85.8
	4 #25 + 4 #12	2416	1689	152.2	1759	133.7	1816	118.6	1863	106.0	1902	95.6	1934	86.7
	4 #25 + 4 #16	2768	1790	153.6	1862	134.8	1920	119.6	1968	106.9	2008	96.3	2041	87.4
	4 #25 + 2 #25	2945	1860	150.4	1928	132.2	1984	117.3	2030	105.0	2068	94.6	2101	85.8
	4 #32 + 0 #0	3217	1819	199.5	1902	176.6	1969	157.8	2026	142.0	2073	128.7	2114	117.3
	4 #25 + 4 #20	3220	1920	155.4	1994	136.4	2054	120.8	2104	108.0	2146	97.2	2179	88.2

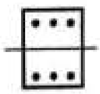
Notes : Arrangement of Longitudinal Steel.

N1 - D1 Represent Total Number of bars in first and Last raw having Same diameter.

N2 - D2 Represent Total Number of bars BETWEEN first and Last row

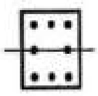
For Example

6 #20 + 0 -#0 (i.e.N1 = 6 and *N2* = 0), means 3 No of 20*mm* diameter bars in first row and 3 No. in the last row.



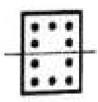
Thus, total no of rows = 2

6 #20 + 2 -#12 (i.e.N1 = 6 and *N2* = 2), means 3 No of 20mm diameter bars in first arow and 3 No. in the last row and 2 no of 12*mm* bars at mid-depth



Thus, total no of rows = 3

6 #20 + 4 -#12 (i.e.N1 = 6 and *N2* = 4), means 3 No of 20*mm* diameter bars in fir row and 3 No. in the last row and 2 no of 12*mm* bars in 2nd row and 2 No of 12*mm* in3rd row equispaced.



Thus, total no of rows = 4

Table G-3 Values of K_1, k_2 for calculation of P_{ub} for Slender Columns.

For Rectangular Section : $P_{ub} = (k_1 + k_2 p / f_{ck}) f_{ck} bD$

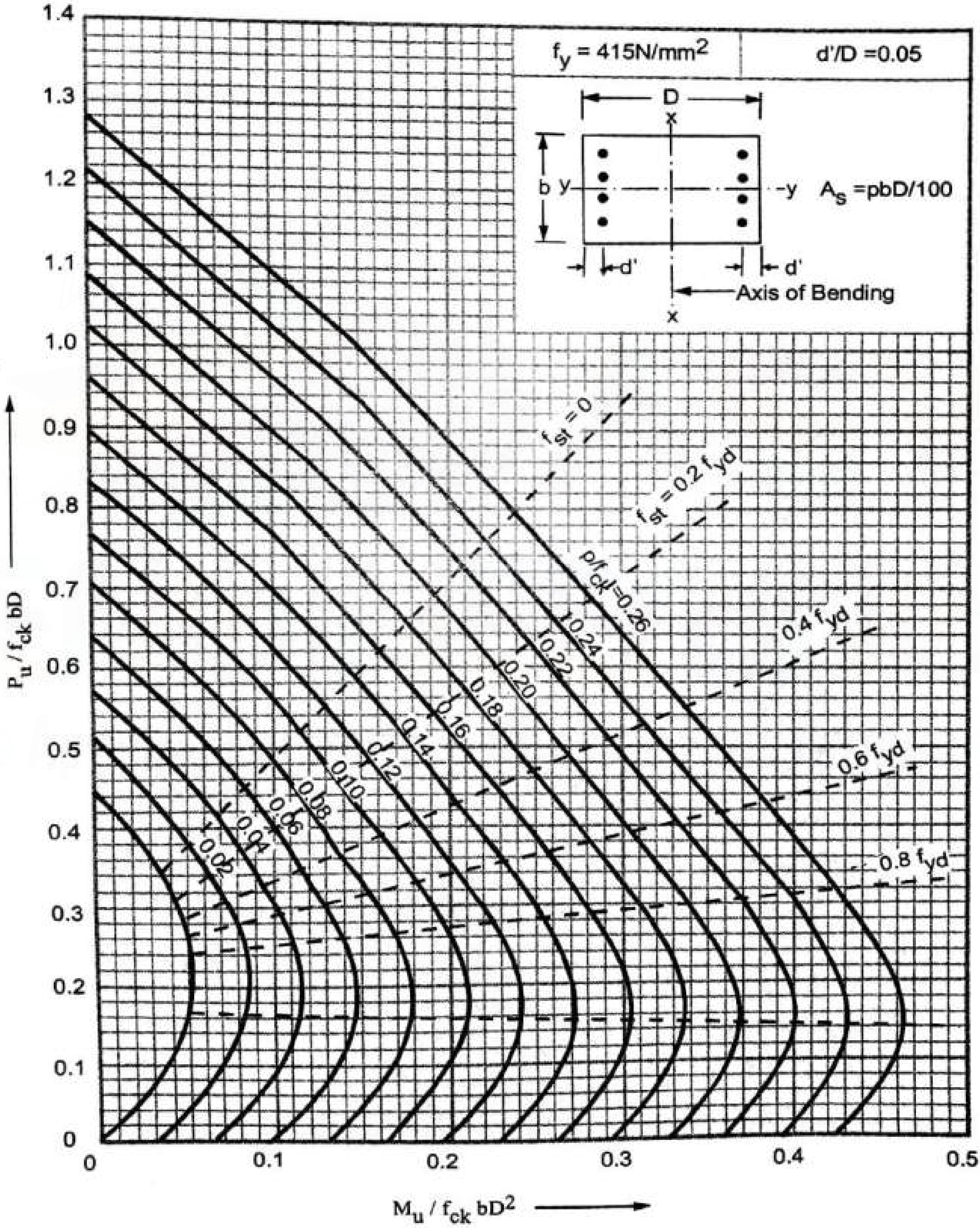
Section	Values of k_1			
	d'/D			
	0.05	0.10	0.15	0.2
Rectangular	0.219	0.207	0.196	0.184

Section	f_y	Values of k_2			
		d'/D			
	N/mm²	0.05	0.10	0.15	0.2
Rectangular					
~Equal Reinforcement on two opposite sides	250	–0.045	–0.045	–0.045	–0.045
	415	0.096	0.082	0.046	–0.022
	500	0.213	0.173	0.104	–0.001
~Equal Reinforcement on all four sides.	250	0.215	0.146	0.061	–0.011
	415	0.424	0.328	0.203	0.028
	500	0.545	0.425	0.256	0.040

Table G-4 Maximum Pitch of Lateral Ties.

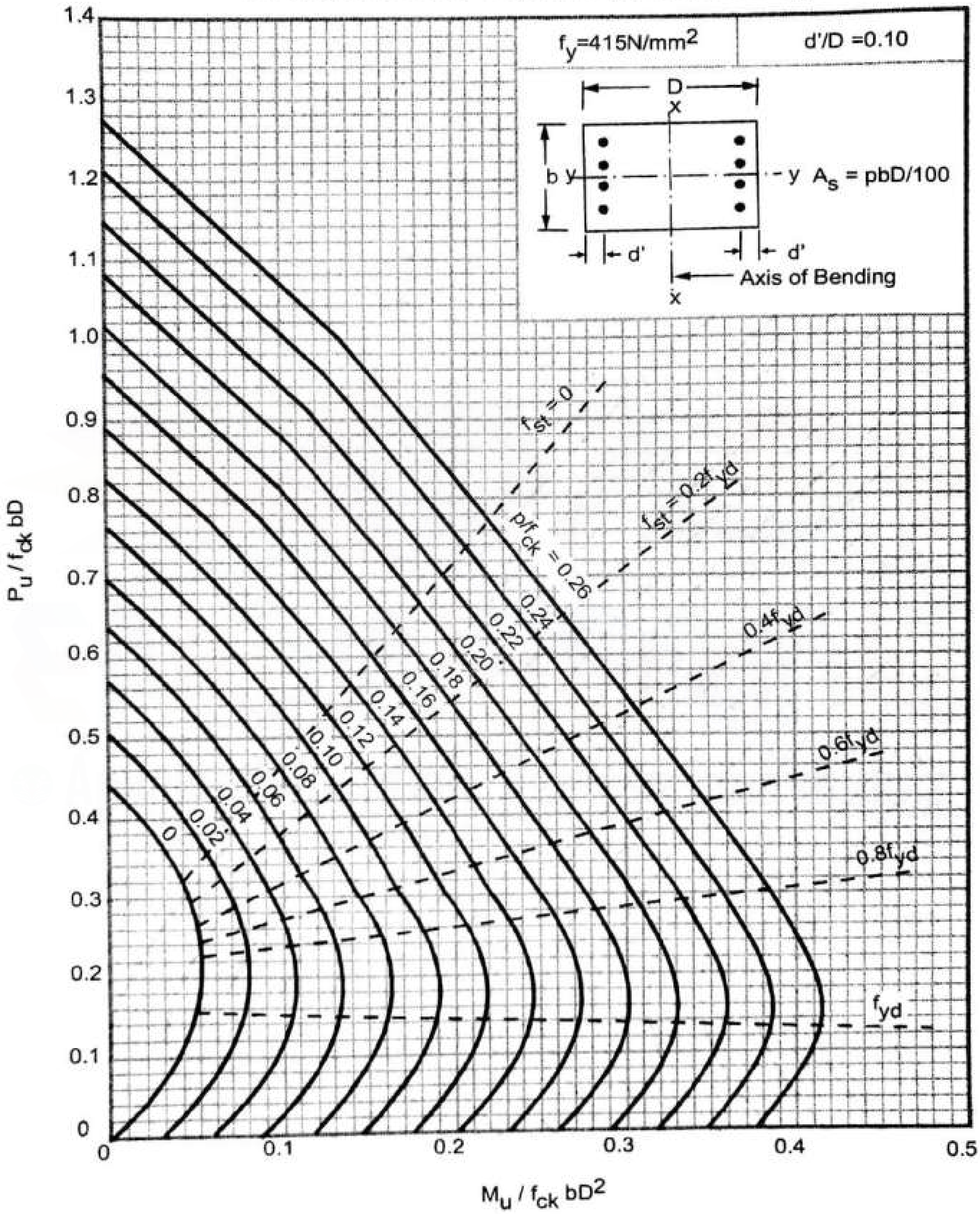
Smallest Dia. of Longitudinal bar	Diameter of Lateral Ties in mm				Remarks
	5	6	8	10	
12 mm	190	190	190	190	but ≯ b
16 mm	250	250	250	250	but ≯ b
20 mm	300	300	300	300	but ≯ b
25 mm	–	–	300	300	but ≯ b
30 mm	–	–	300	300	but ≯ b
32 mm	–	–	300	300	but ≯ b
36 mm	–	–	–	300	but ≯ b

Chart - 1G Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on Opposite Sides



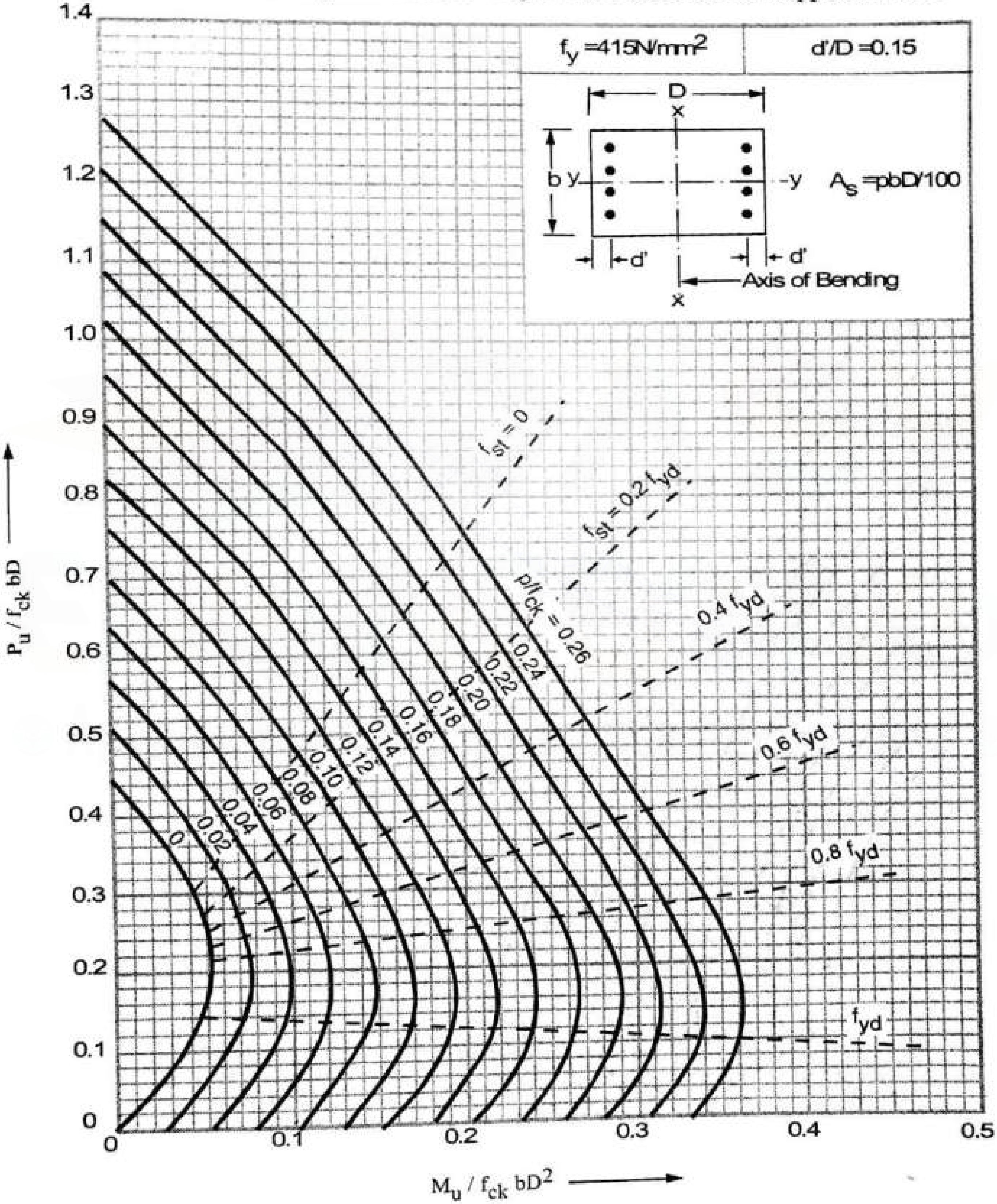
Note: *For bending about y-axis, refer to the same Chart but select the appropriate Chart for the ratio of d'/b and the coefficient obtained on x-axis will be for $M_{uy}/f_{ck}b^2D$*

Chart - 2G Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on Opposite Sides



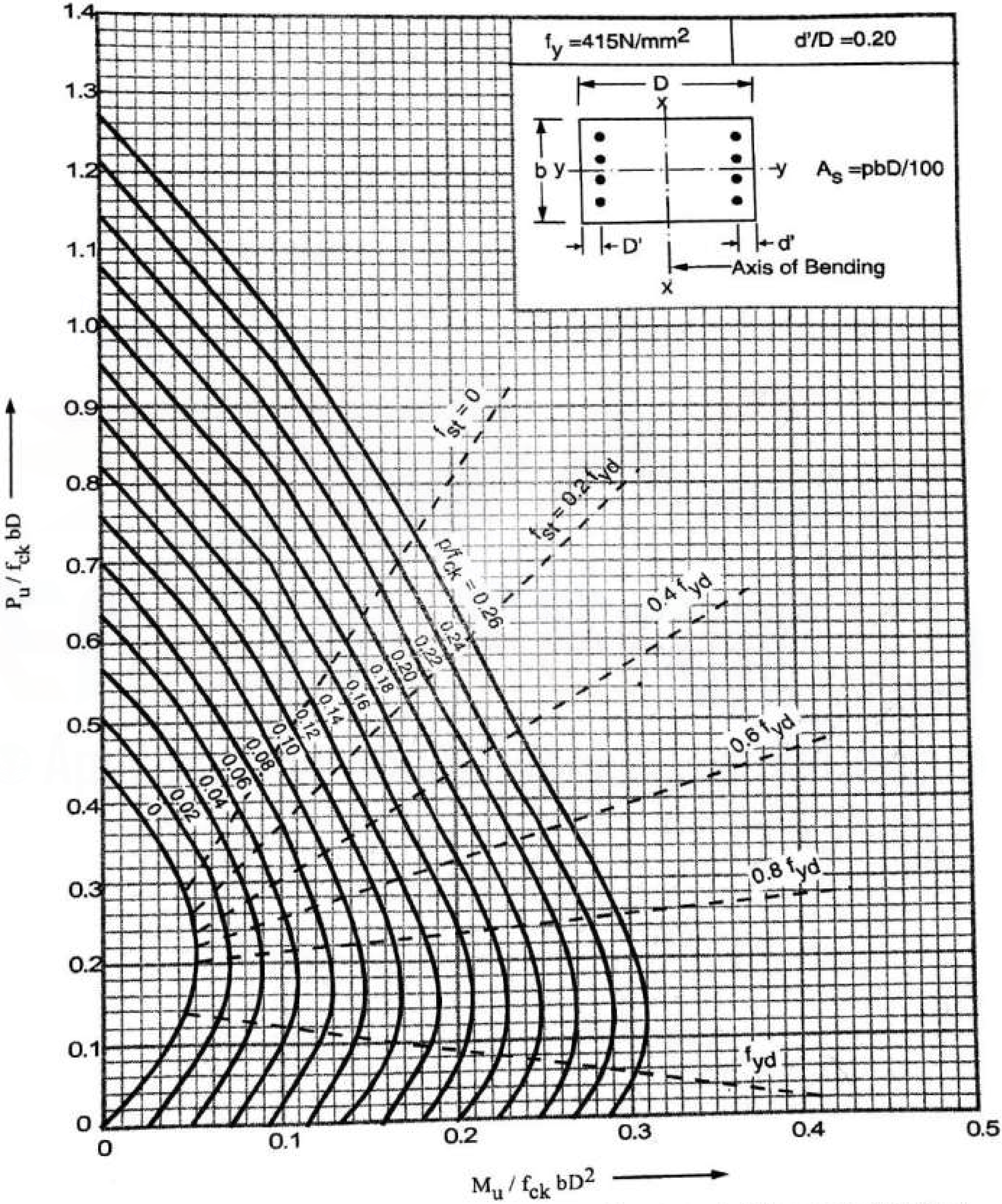
Note: *For bending about y-axis, refer to the same Chart but select the appropriate Chart for the ratio of d'/b and the coefficient obtained on x-axis will be for* $M_{uy}/f_{ck}b^2D$

Chart - 3G Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on Opposite Sides



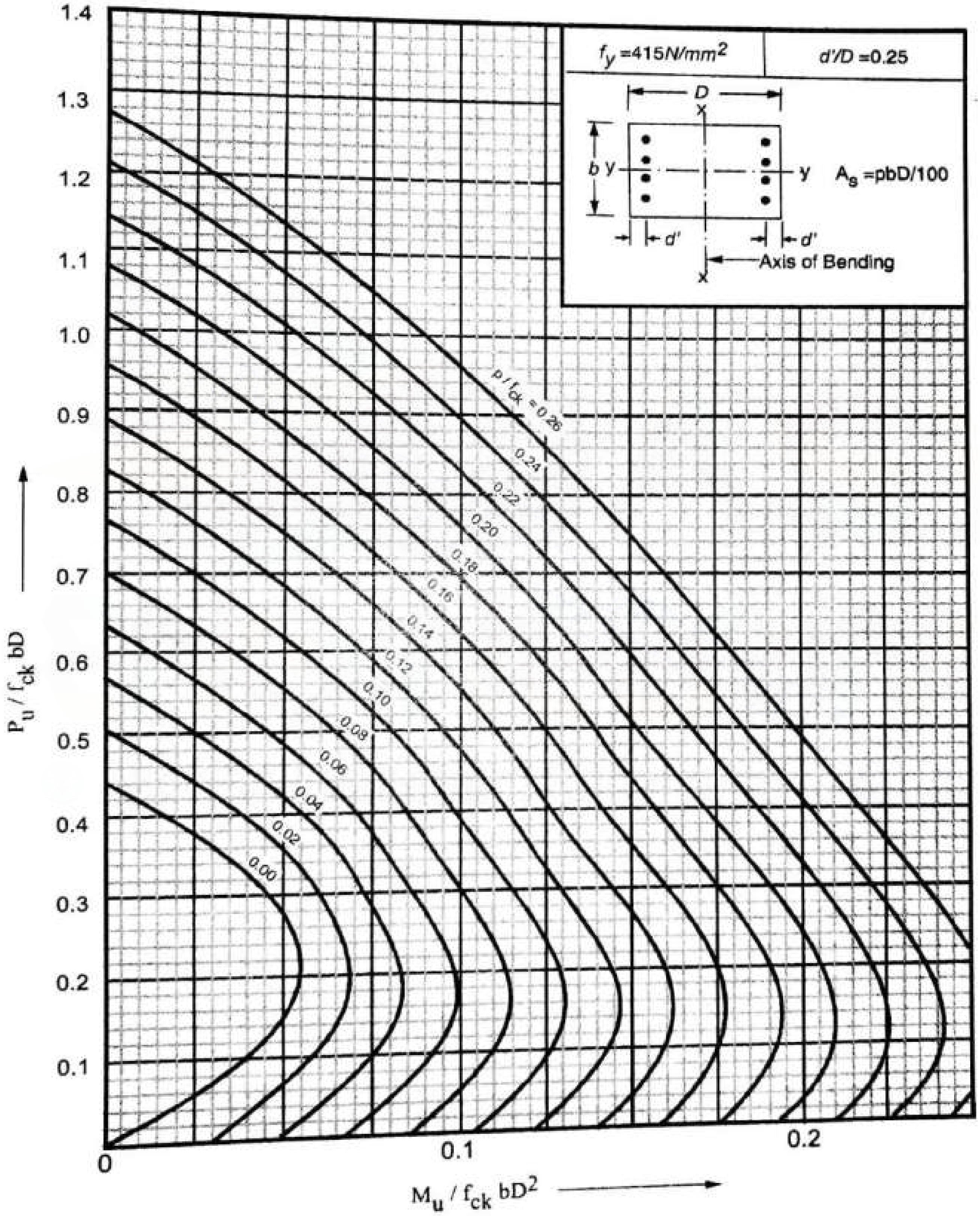
Note: *For bending about y-axis, refer to the same Chart but select the appropriate Chart for the ratio of d'/b and the coefficient obtained on x-axis will be for $M_{uy} / f_{ck}b^2D$*

Chart - 4G Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on Opposite Sides



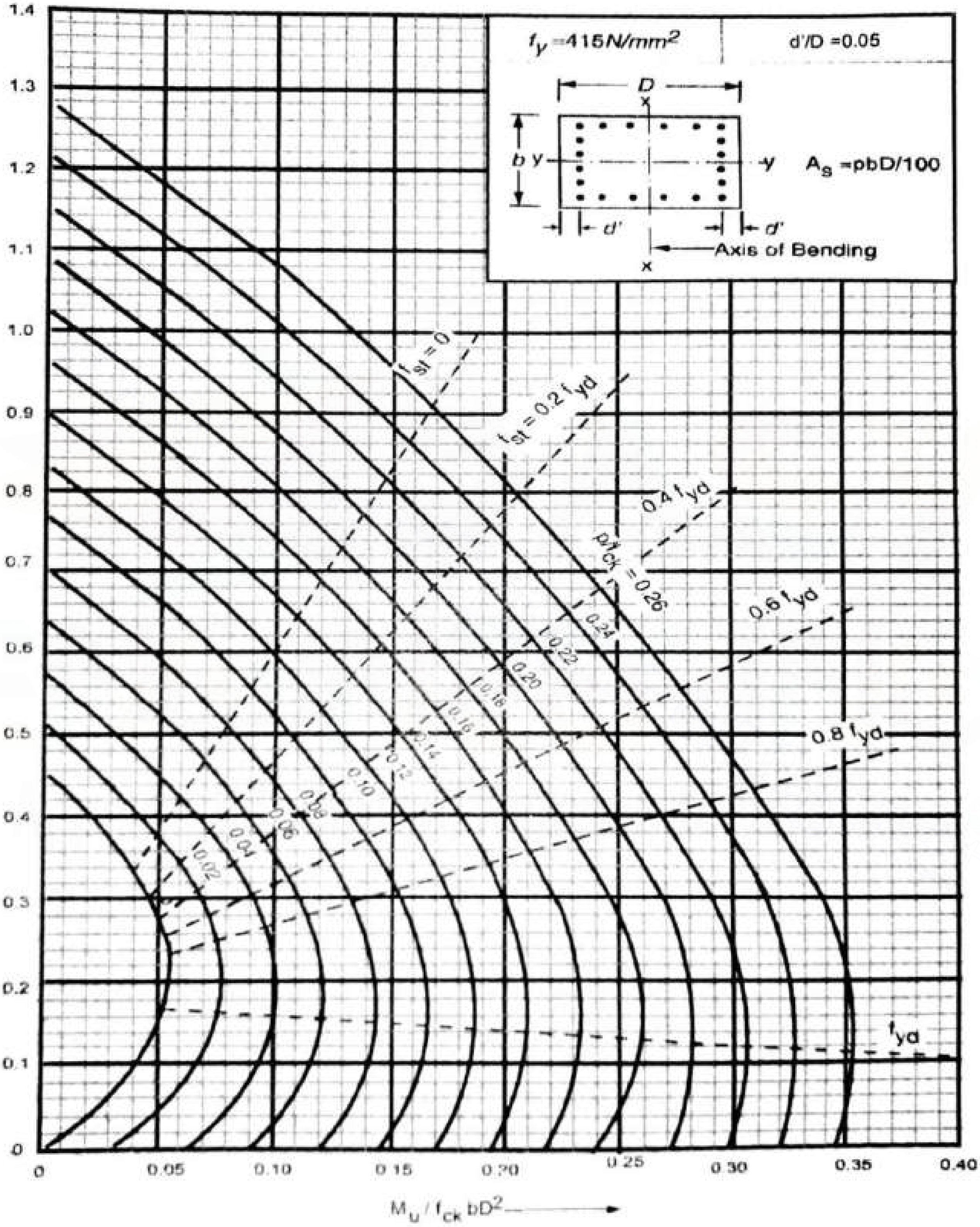
Note : *For bending about y-axis, refer to the same Chart but select the appropriate Chart for the ratio of d'/b and the coefficient obtained on x-axis will be for* $M_{uy}/f_{ck}b^2D$

Chart - 5G Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on Opposite Sides.



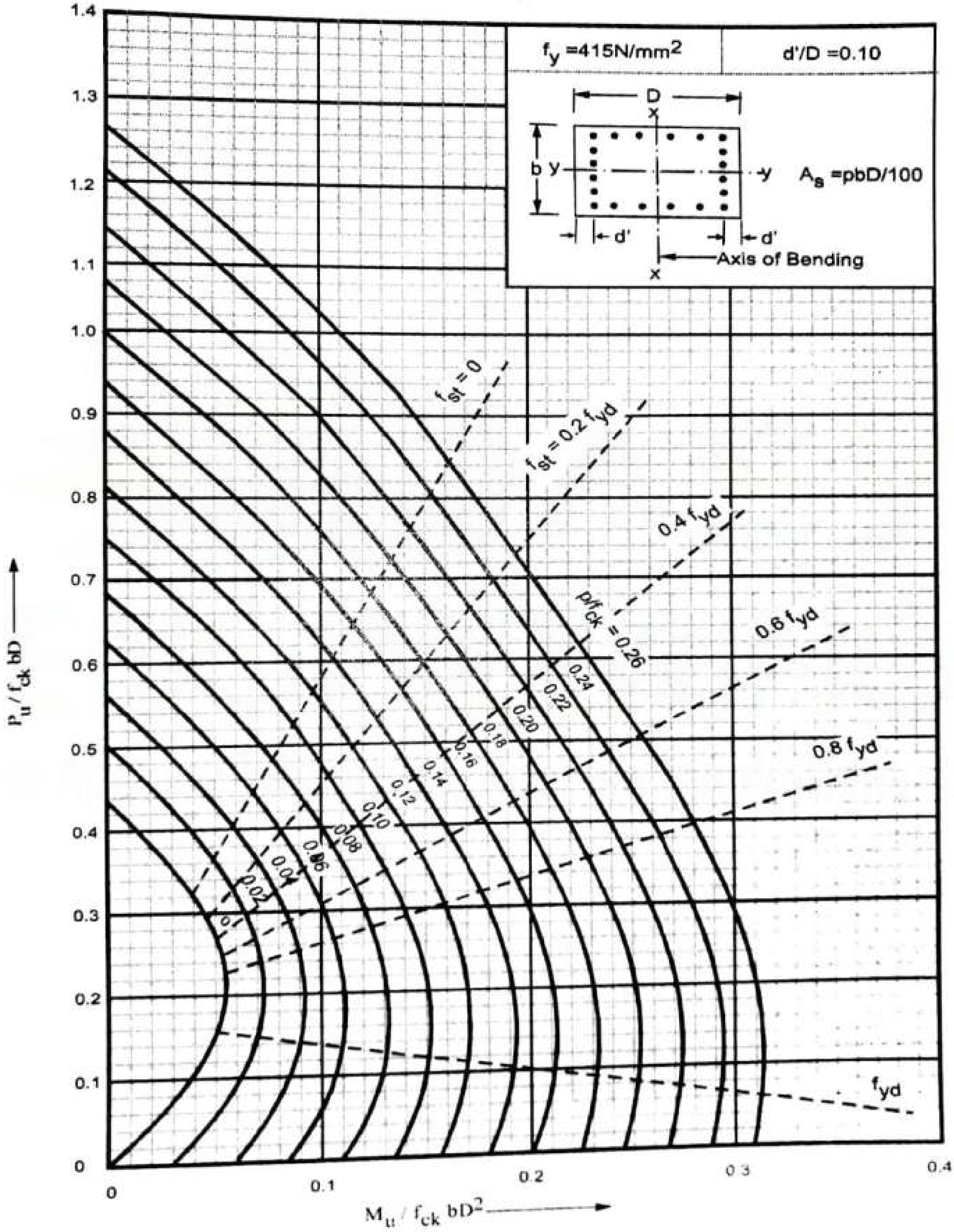
Note: *For bending about y-axis, refer to the same Chart but select the appropriate Chart for the ratio of d'/b and the coefficient obtained on x-axis will be for* $M_{uy}/f_{ck}b^2D$

Chart - 6G Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on Four Sides



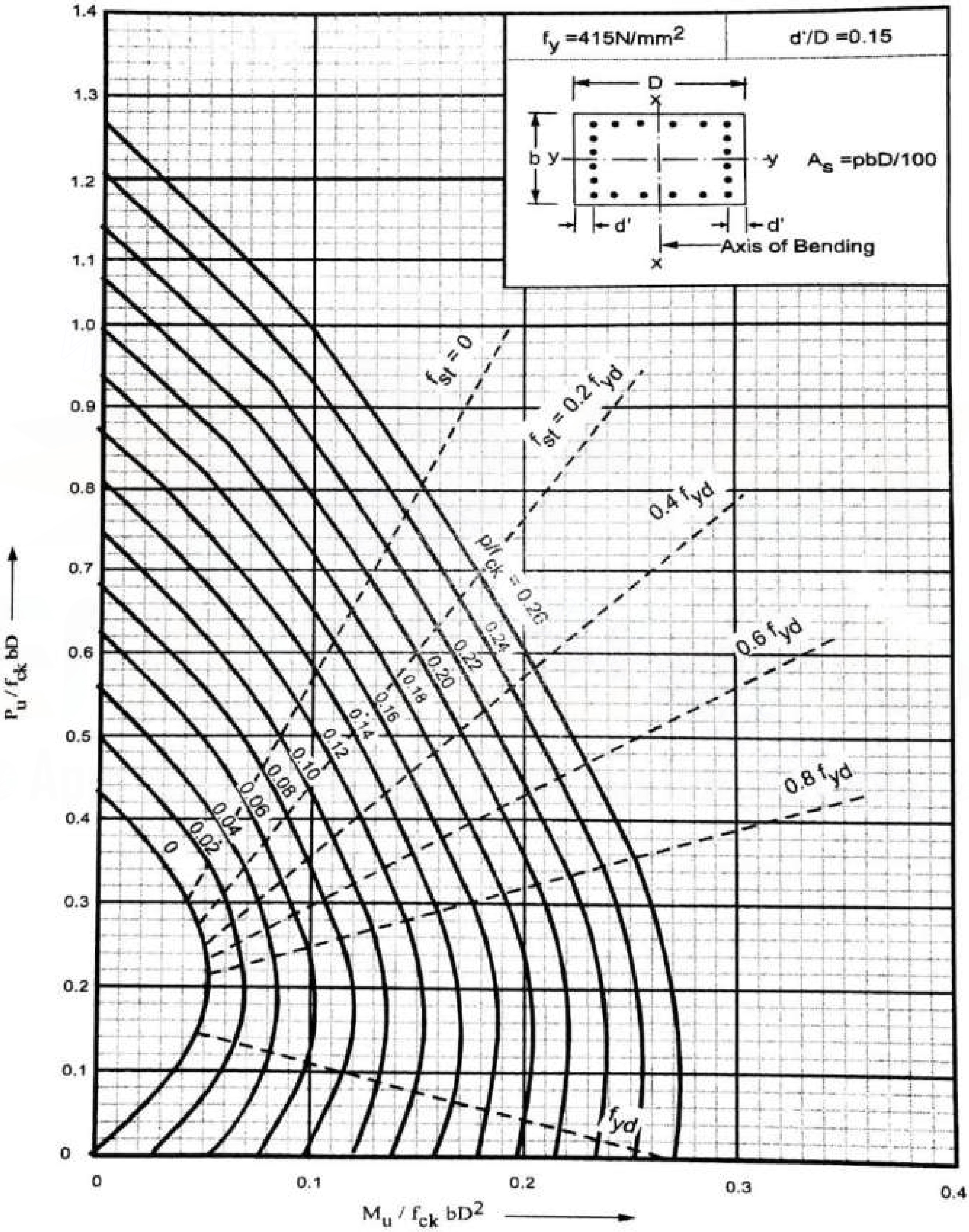
Note: For bending about y-axis, refer to the same Chart but select the appropriate Chart for the ratio of d'/b and the coefficient obtained on x-axis will be for $M_{uy}/f_{ck}b^2D$

Chart - 7G Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on Four Sides



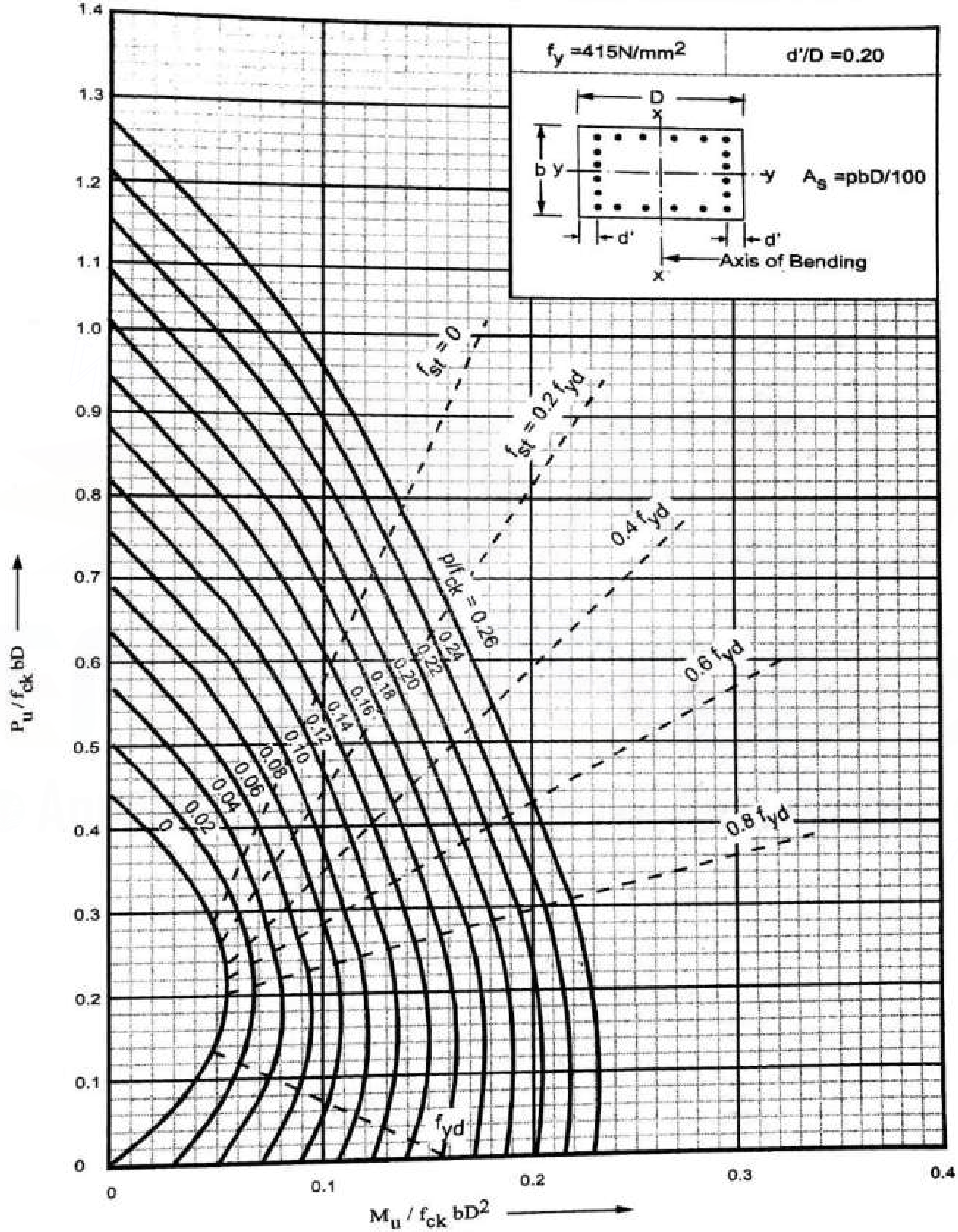
Note: For bending about y-axis, refer to the same Chart but select the appropriate Chart for the ratio of d'/b and the coefficient obtained on x-axis will be for $M_{uy}/f_{ck}b^2D$

Chart - 8G Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on Four Sides



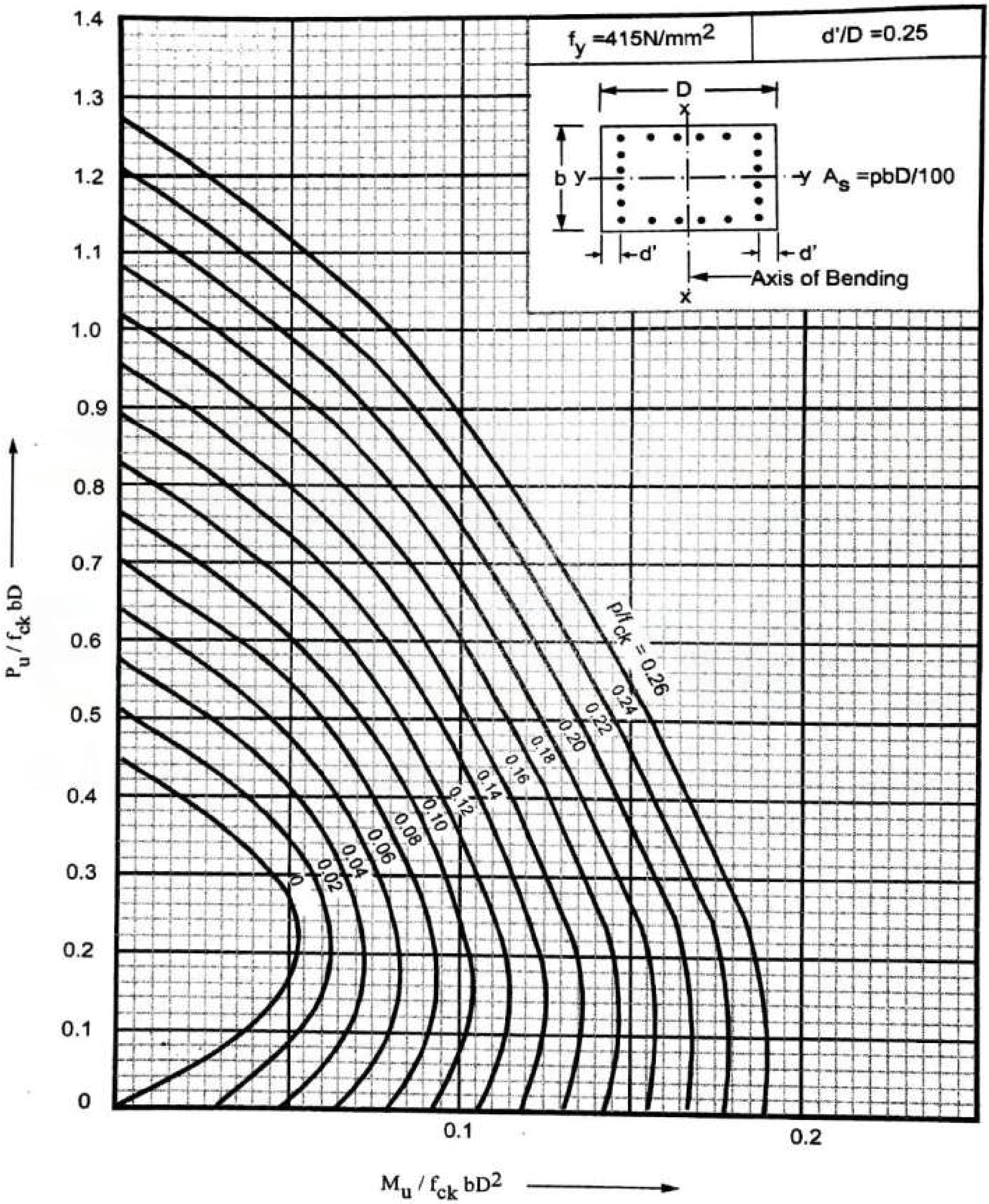
Note: *For bending about y-axis, refer to the same Chart but select the appropriate Chart for the ratio of d'/ b and the coefficient obtained on x-axis will be for* $M_{uy}/f_{ck}b^2D$

Chart - 9G Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on Four Sides



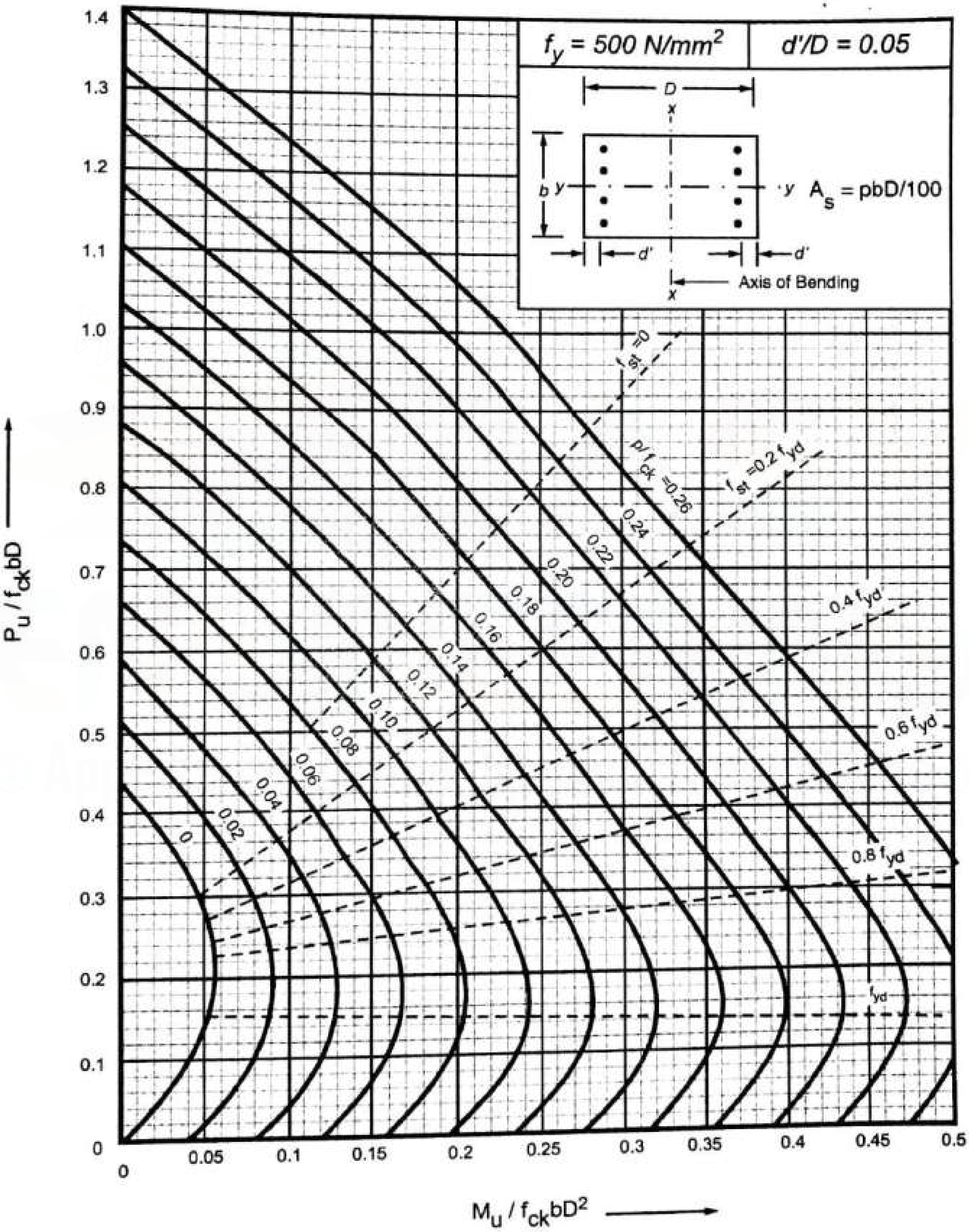
Note: For bending about y-axis, refer to the same Chart but select the appropriate Chart for the ratio of d'/b and the coefficient obtained on x-axis will be for $M_{uy}/f_{ck}b^2D$

Chart - 10G Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on Four Sides



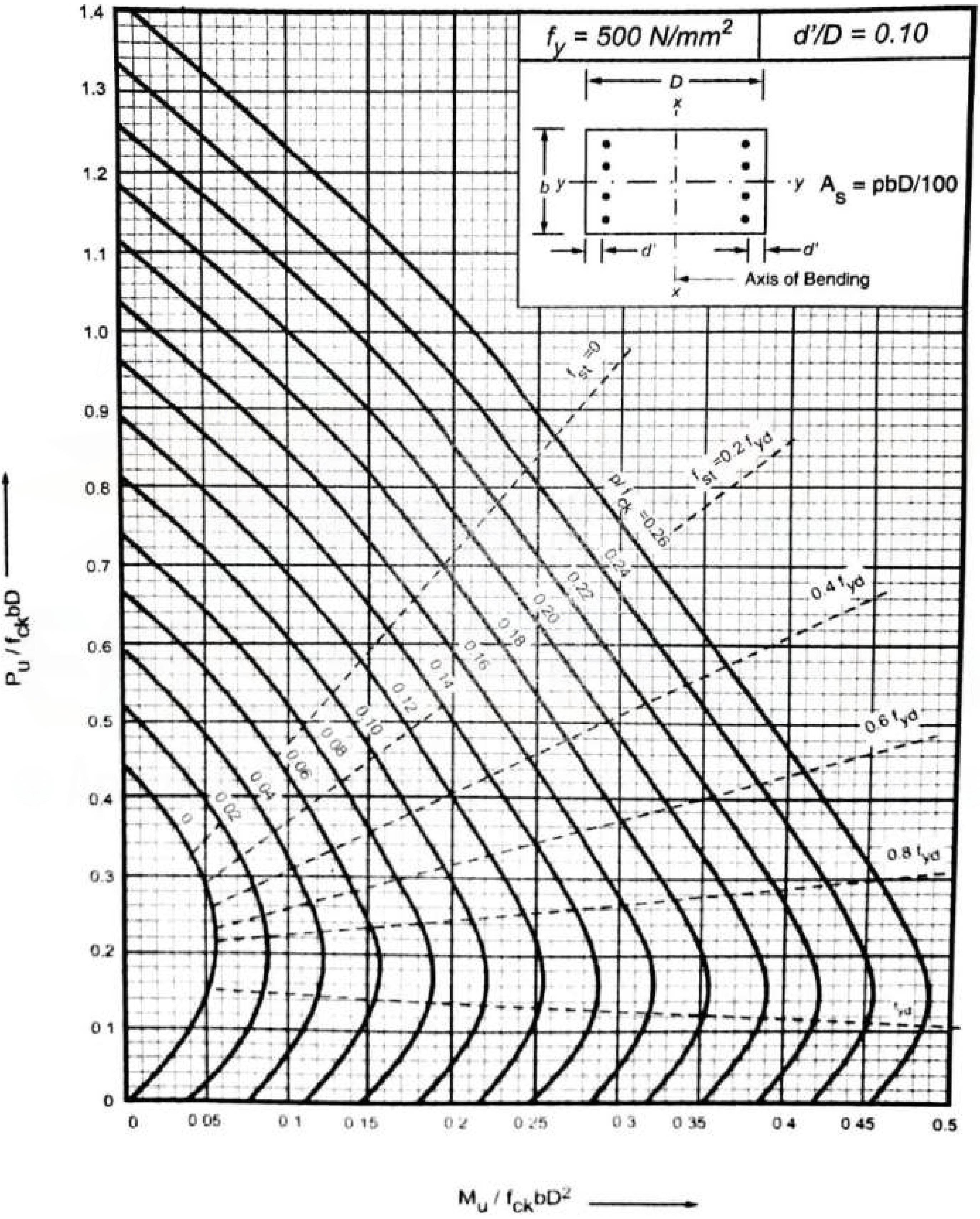
Note: For bending about y-axis, refer to the same Chart but select the appropriate Chart for the ratio of d'/b and the coefficient obtained on x-axis will be for $M_{uy} / f_{ck} b^2 D$

Chart - 11 G Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on Opposite Sides.



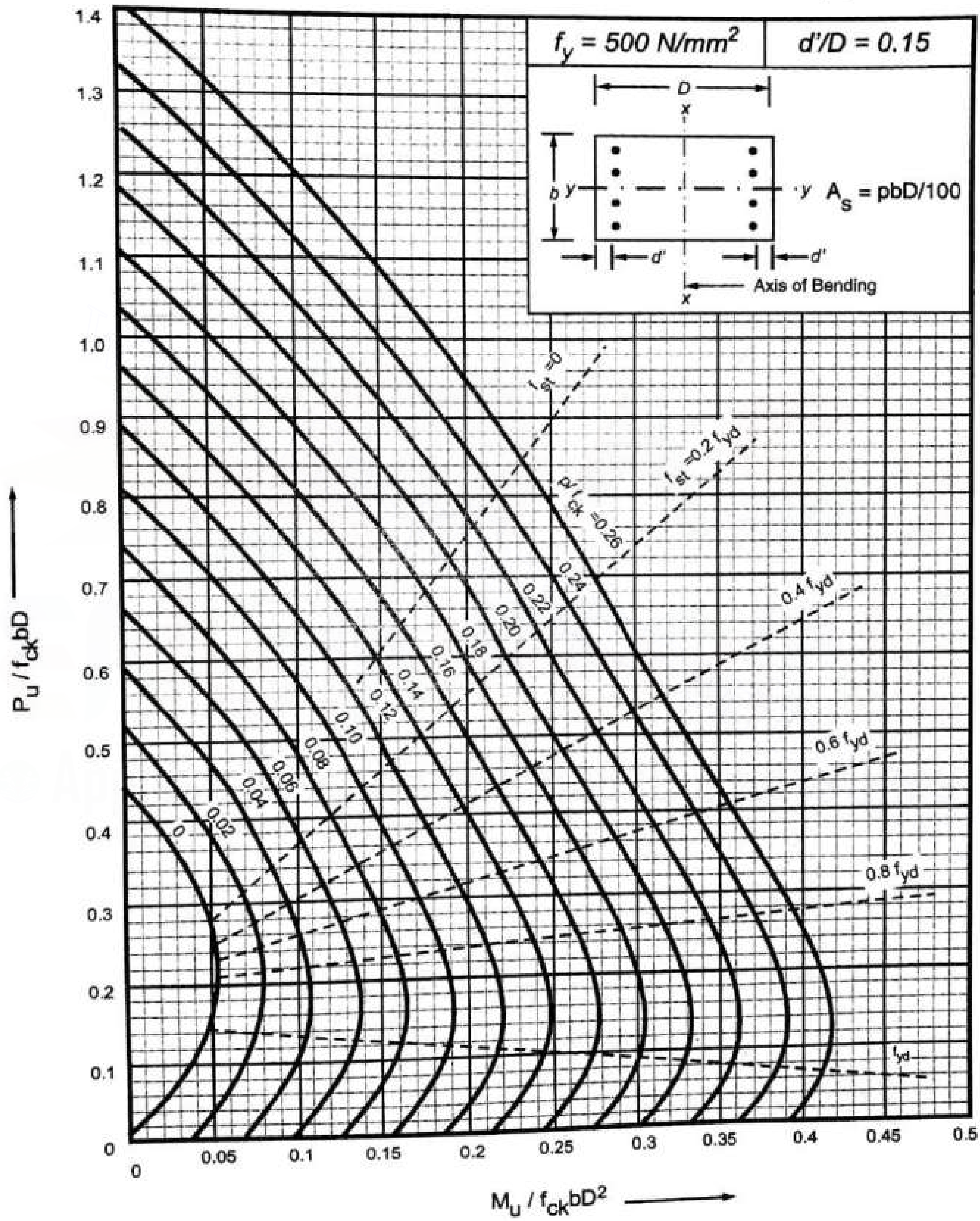
Note : For Bending about y - axis, refer to the same chart but select the appropriate chart for the ratio of d' / b and the coefficient obtained on x- axis will be for $M_{uy} / f_{ck} b^2D$

Chart - 12 G Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on Opposite Sides.



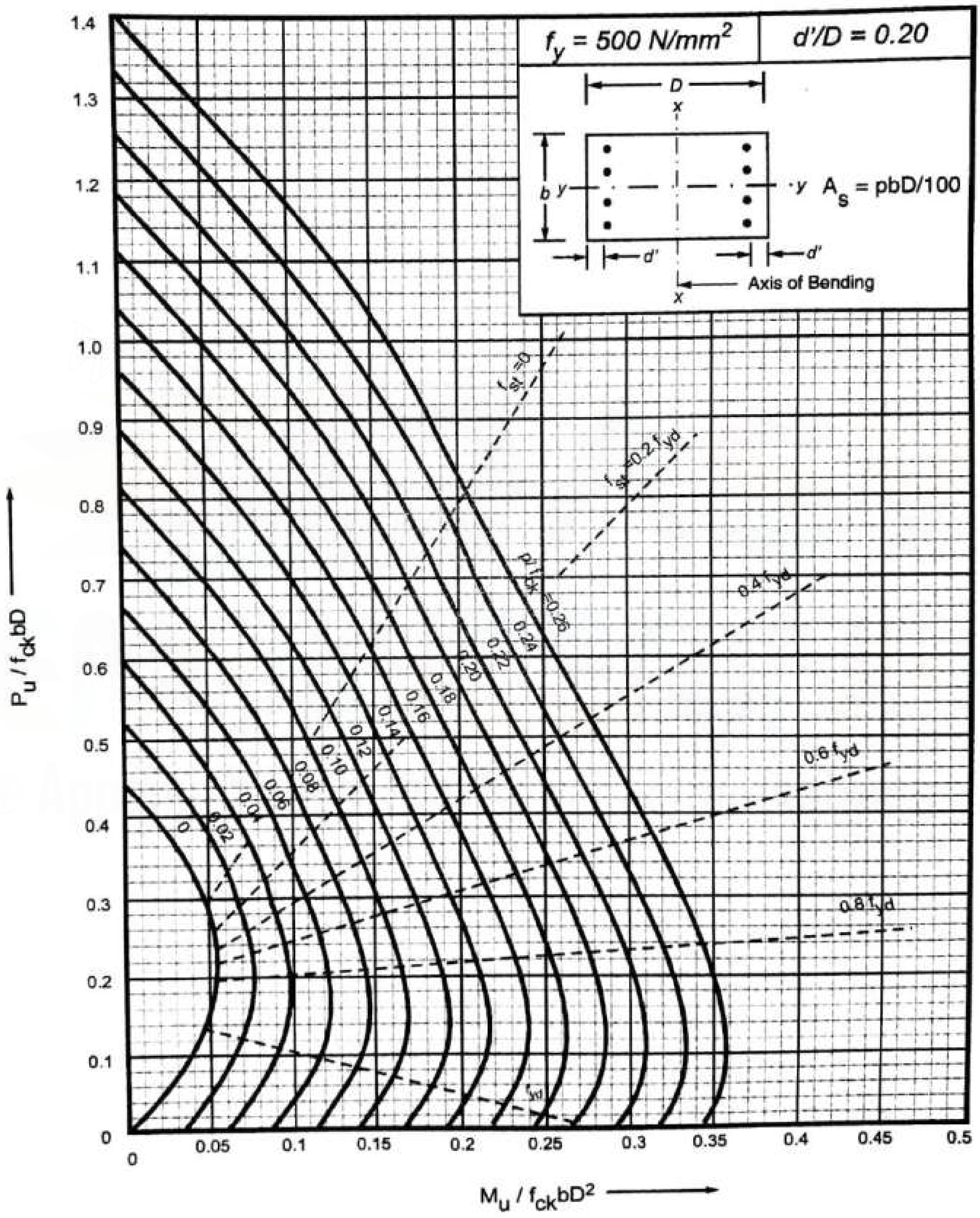
Note : For Bending about y - axis, refer to the same chart but select the appropriate chart for the ratio of d' / b and the coefficient obtained on x- axis will be for $M_{uy} / f_{ck}\, b^2D$

Chart - 13 G Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on Opposite Sides.



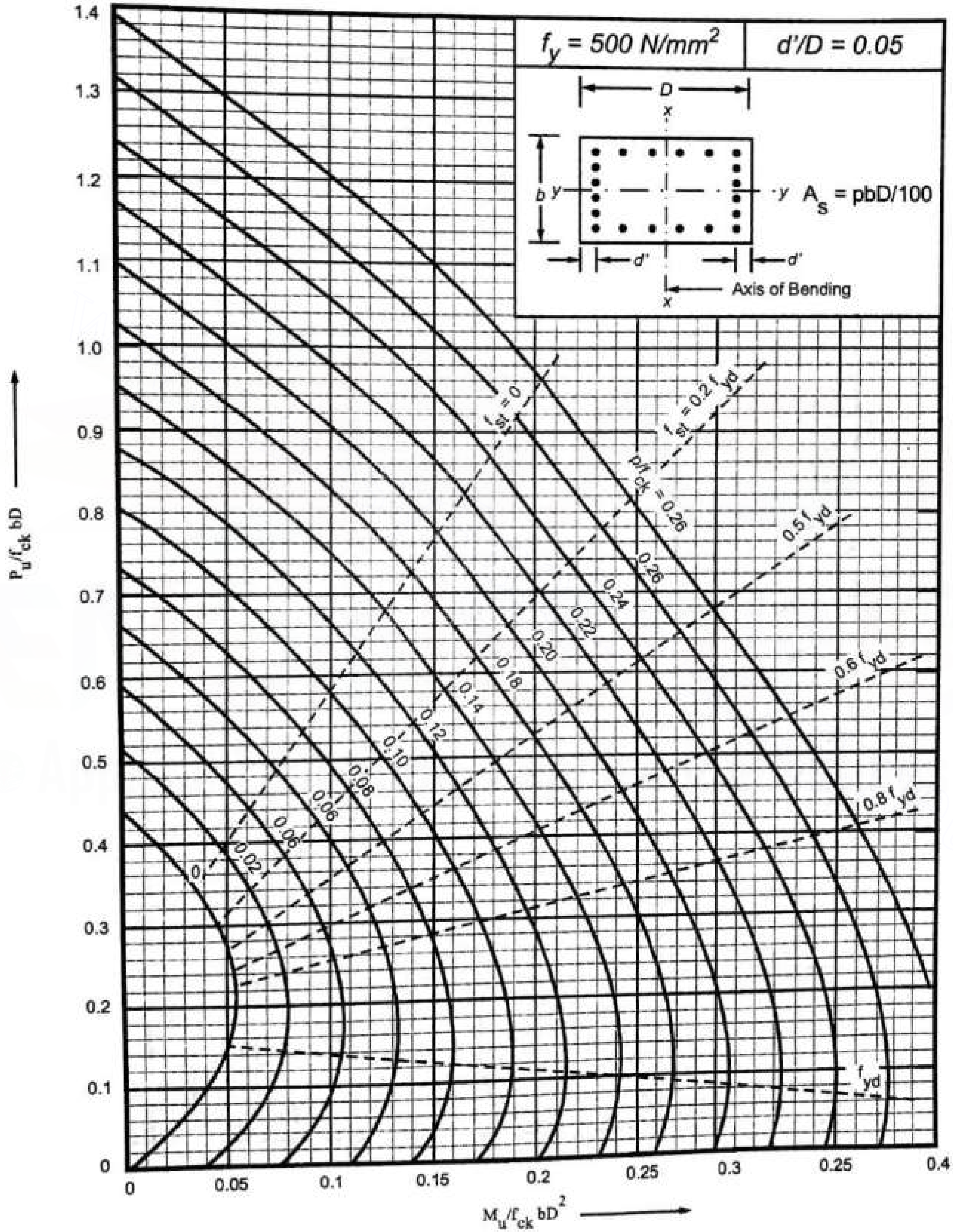
Note : For Bending about y - axis, refer to the same chart but select the appropriate chart for the ratio of d' / b and the coefficient obtained on x- axis will be for $M_{uy} / f_{ck}\, b^2D$

Chart - 14 G Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on Opposite Sides.



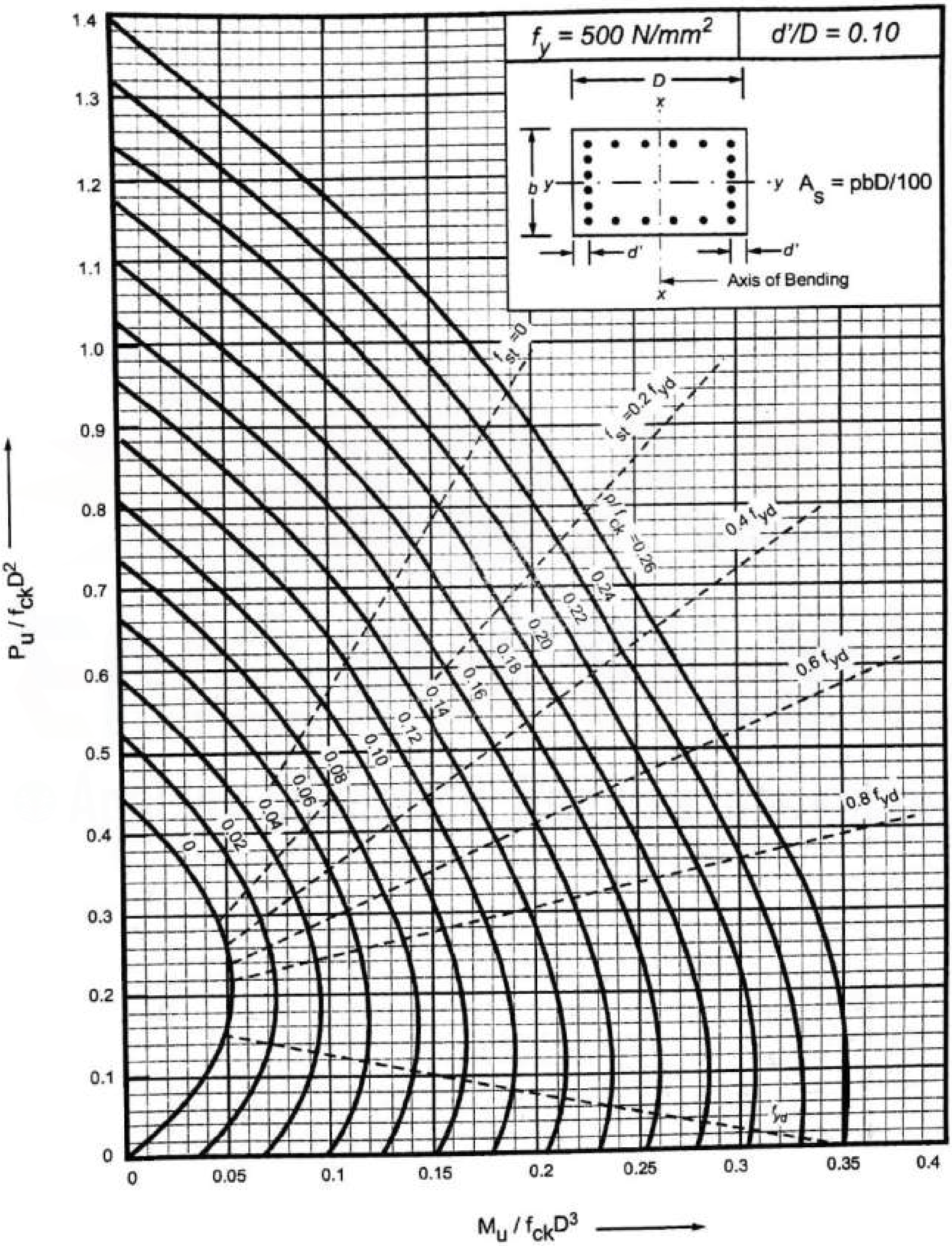
Note : For Bending about y - axis, refer to the same chart but select the appropriate chart for the ratio of d' / b and the coefficient obtained on x- axis will be for M_{uy} / f_{ck} b^2D

Chart - 15 G Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on Four Sides.



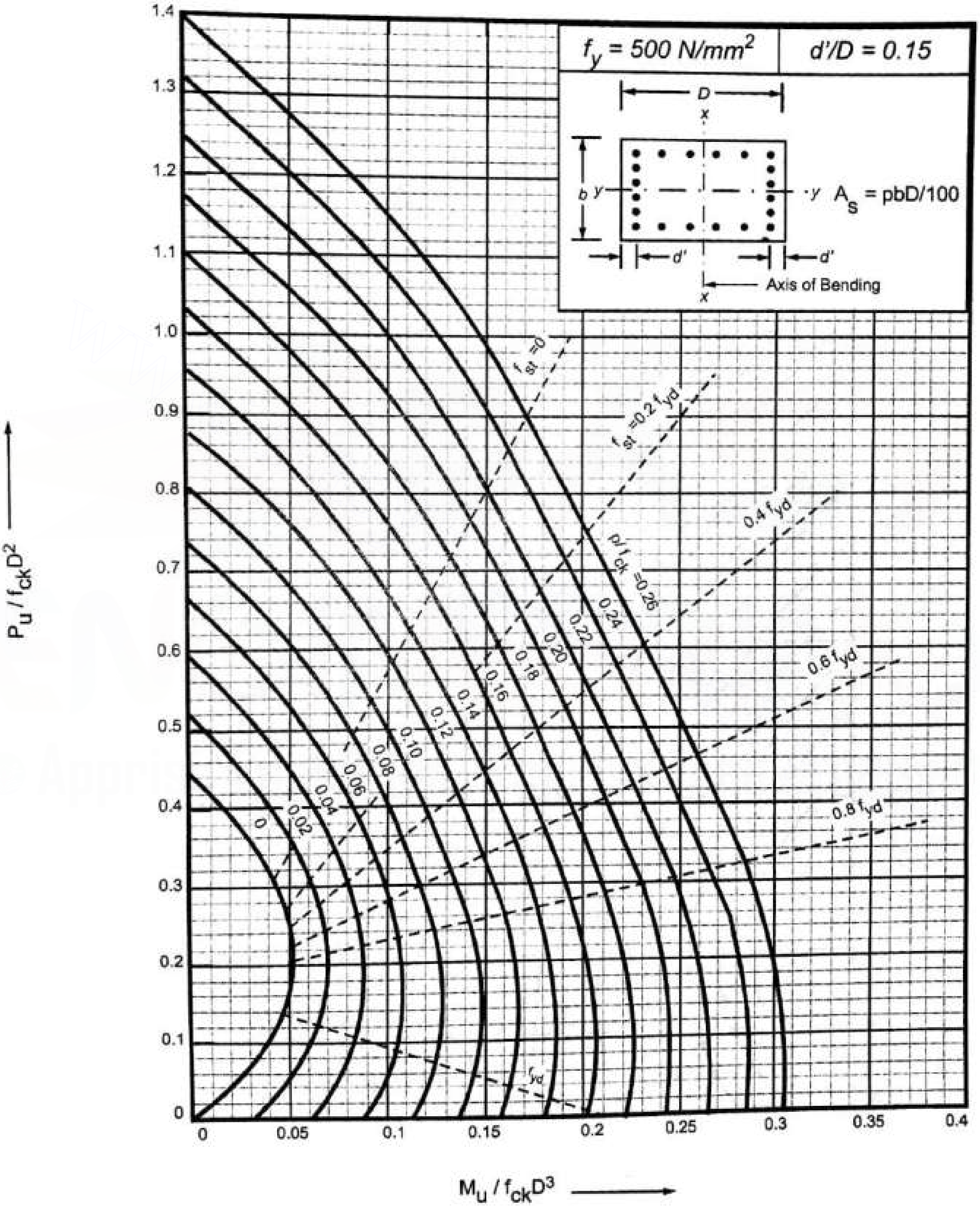
Note : For Bending about y - axis, refer to the same chart but select the appropriate chart for the ratio of d' / b and the coefficient obtained on x- axis will be for $M_{uy} / f_{ck}\, b^2D$

Chart - 16 G Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on Four Sides.



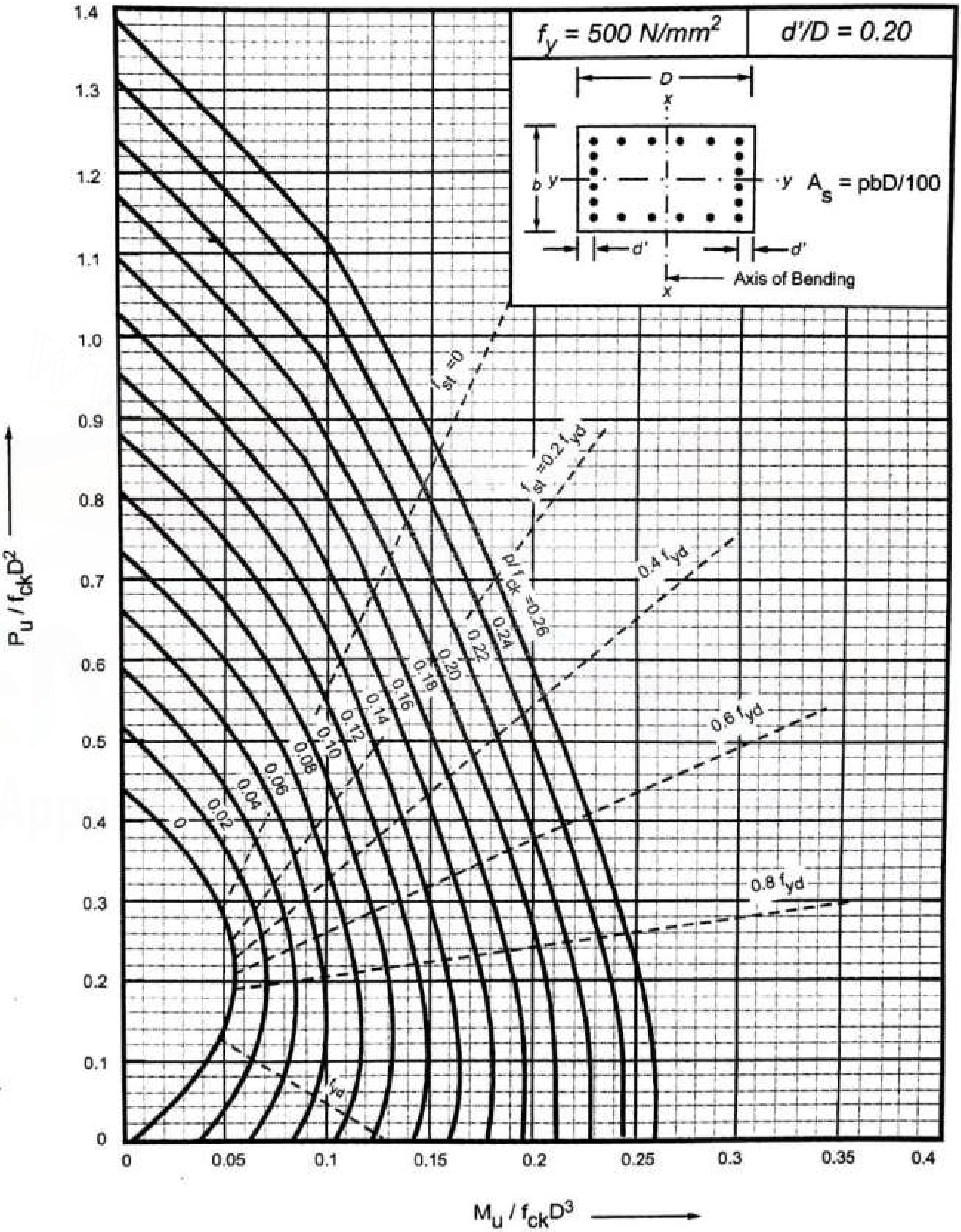
Note : For Bending about y - axis, refer to the same chart but select the appropriate chart for the ratio of d' / b and the coefficient obtained on x- axis will be for $M_{uy} / f_{ck}\ b^2D$

Chart - 17 G Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on Four Sides.



Note : For Bending about y - axis, refer to the same chart but select the appropriate chart for the ratio of d' / b and the coefficient obtained on x- axis will be for $M_{uy} / f_{ck}\, b^2 D$

Chart - 18 G Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on Four Sides.



Note : For Bending about y - axis, refer to the same chart but select the appropriate chart for the ratio of d' / b and the coefficient obtained on x- axis will be for $M_{uy} / f_{ck}\, b^2 D$

APPENDIX - H REINFORCEMENT DATA

Table H-1 Properties of Round Bars

Diam *mm*	*Area* *mm²*	*Peri-meter* *mm*	*Weight* *kg/m*	*Length per tonne* *m*	*Diam* *mm*	*Area* *mm²*	*Peri-meter* *mm*	*Weight* *kg/m*	*Length per tone* *m*
6	28.3	18.8	0.222	4310	**20**	314.2	62.8	2.466	405
8	50.3	25.1	0.395	2332	22	380.1	69.1	2.984	336
10	78.5	31.4	0.616	1621	**25**	490.9	78.5	3.853	260
12	113.1	37.7	0.888	1125	28	615.7	88.0	4.834	207
16	201.1	50.3	10578	633	**32**	804.2	100.5	6313	159

Note :- (1) Basic Weight = 0.00785 *kg/mm²* = 0.006165(F^2) or (F^2/162)*kg/m* where, F is in *mm*.
(2) The maximum Length of bars available ex. stock is 13 m.

Table H-2 Spacing of Distribution Steel

Thickness of slab (mm)	**Fe250 Diameter of bar (mm)**			**Fe415 or Fe500 Diameter of bar (mm)**		
	6	**8**	**10**	**6**	**8**	**10**
	Area 28.27	**50.26**	**78.54**	**28.27**	**50.26**	**78.54**
	Spacing in mm					
100	180	300	--	230	--	--
110	170	300	--	210	300	--
120	150	270	300	190	300	--
130	140	250	300	180	300	--
140	130	230	300	160	290	--
150	120	220	300	150	270	300
160	110	200	300	140	260	300
170	110	190	300	130	240	200
180	100	180	300	130	230	300
190	90	170	300	120	220	300
200	90	160	300	110	200	300
220	80	150	300	100	190	290
250	75	130	300	90	160	260

H - 3 Overall Approximate Consumption of Steel (HYSD bars) in Buildings.

(1) Residenses constructed with Load bearing walls	10 - 20 kg/m²
(2) Flats costructed with columns	30 - 50 kg/m²
(3) Office and residenses multistoreied buildings	40 - 70 kg/m²

H - 4 Approximate Quantities of Other Materials per square meter of Plinth Area

(1) Cement Residential Building	150 kg
(2) Commertial Buildings	200 kg
(3) Bricks	250 - 300 Nos
(4) Sand	0.5 to 0.7 m³
(5) Aggregates for Mass Concrete	0.2 m³
(6) Aggregates for RCC work	0.2 m³

H - 5 Approximate Quantities of Steel in Buildings

(1) Slab	25 - 50 kg/m³
(2) Beam (longitudial reinforcement)	25 - 100 kg/m³
(3) Stirrups	8 - 30 kg/m³
(4) Column	60 - 120 kg/m³
(5) Footing	20 - 50 kg/m³

Table H-6 Area of Steel for Combinations of Bar Diameter and Number.

No. of Bars	DIAMETERS OF BARS in mm											
	5	6	8	10	12	16	18	20	22	25	28	32
						Area in mm^2						
1	19	28	50	78	113	201	254	314	380	491	615	804
2	39	56	100	157	226	402	508	628	760	981	1231	1608
3	59	84	150	235	339	603	763	942	1140	1472	1847	2412
4	78	113	201	314	452	804	1017	1256	1520	1963	2463	3217
5	98	141	251	392	565	1005	1272	1570	1900	2454	3078	4021
6	117	169	301	471	678	1206	1526	1885	2280	2945	3694	4825
7	137	197	351	549	791	1407	1781	2199	2660	3436	4310	5629
8	157	226	402	628	904	1608	2035	2513	3041	3927	4926	6434
9	176	254	452	706	1017	1809	2290	2827	3421	4417	5541	7238
10	196	282	502	785	1131	2010	2544	3141	3801	4908	6157	8042
11	216	311	552	863	1244	2211	2799	3455	4181	5399	6773	8846
12	235	339	603	942	1357	2412	3053	3769	4561	5890	7389	9651
13	255	367	653	1021	1470	2613	3308	4084	4941	6381	8004	10455
14	275	395	703	1099	1583	2814	3562	4398	5321	6872	8620	11259
15	294	424	754	1178	1696	3015	3815	4712	5702	7363	9236	12063
16	314	452	804	1256	1809	3217	4071	5026	6082	7854	9852	12868

Table H -7 Area of Steel for Combinations of Diameter - Spacing of Bars.

Spacing	Diameter of Bars in *mm*											
mm	5	6	8	10	12	16	18	20	22	25	28	32
	Area in mm²											
50	392	565	1005	1570	2262	4021	-	-	-	-	-	-
60	327	471	837	1309	1885	3351	4241	5236	-	-	-	-
70	280	404	718	1122	1615	2872	3635	4488	5430	-	-	-
75	261	377	670	1047	1508	2680	3393	4188	5068	6545	-	-
80	245	353	628	981	1413	2513	3180	3927	4751	6136	-	-
90	218	314	558	872	1256	2234	2827	3490	4223	5454	6841	-
100	196	282	503	785	1131	2010	2544	3141	3801	4908	6157	8042
110	178	257	457	714	1028	1827	2313	2856	3455	4462	5597	7311
120	163	235	419	654	942	1675	2121	2618	3167	4090	5131	6702
125	157	226	402	628	904	1608	2035	2513	3041	3926	4926	6434
130	151	217	387	604	870	1546	1957	2416	2924	3776	4736	6186
140	140	202	359	561	807	1436	1817	2244	2715	3506	4398	5744
150	130	188	335	523	754	1340	1696	2094	2534	3272	4105	5361
160	122	176	314	490	706	1256	1590	1963	2375	3068	3843	5026
170	115	166	296	462	665	1182	1496	1848	2236	2887	3622	4730
175	112	161	287	448	646	1149	1451	1795	2172	2805	3518	4595
180	109	157	279	436	628	1117	1413	1745	2111	2727	3420	4468
190	103	148	265	413	595	1058	1339	1653	2000	2583	3240	4232
200	98	141	251	392	565	1005	1272	1570	1900	2454	3078	4021
210	93	134	239	374	538	957	1211	1496	1810	2337	2932	3829
220	89	128	228	357	514	914	1156	1428	1727	2231	2798	3655
225	87	125	223	349	502	893	1131	1396	1689	2181	2786	3574
230	85	123	218	341	491	874	1106	1365	1652	2134	2677	3496
240	81	117	209	327	471	837	1060	1309	1583	2045	2565	3351
250	78	113	201	314	452	804	1017	1256	1520	1963	2463	3217
260	75	108	193	302	435	773	978	1208	1462	1888	2368	3093
270	72	104	186	290	418	744	942	1163	1407	1818	2280	2978
275	71	102	182	285	411	731	925	1142	1382	1785	2239	2924
280	70	101	179	280	404	718	908	1122	1357	1753	2199	2872
290	67	97	173	270	390	693	877	1083	1310	1692	2123	2773
300	65	94	167	261	377	670	848	1047	1267	1636	2052	2680
350	56	80	143	224	323	574	727	897	1085	1402	1759	2297
400	49	70	125	196	282	502	636	785	950	1227	1539	2010
450	43	62	111	174	251	446	565	698	844	1090	1368	1787

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