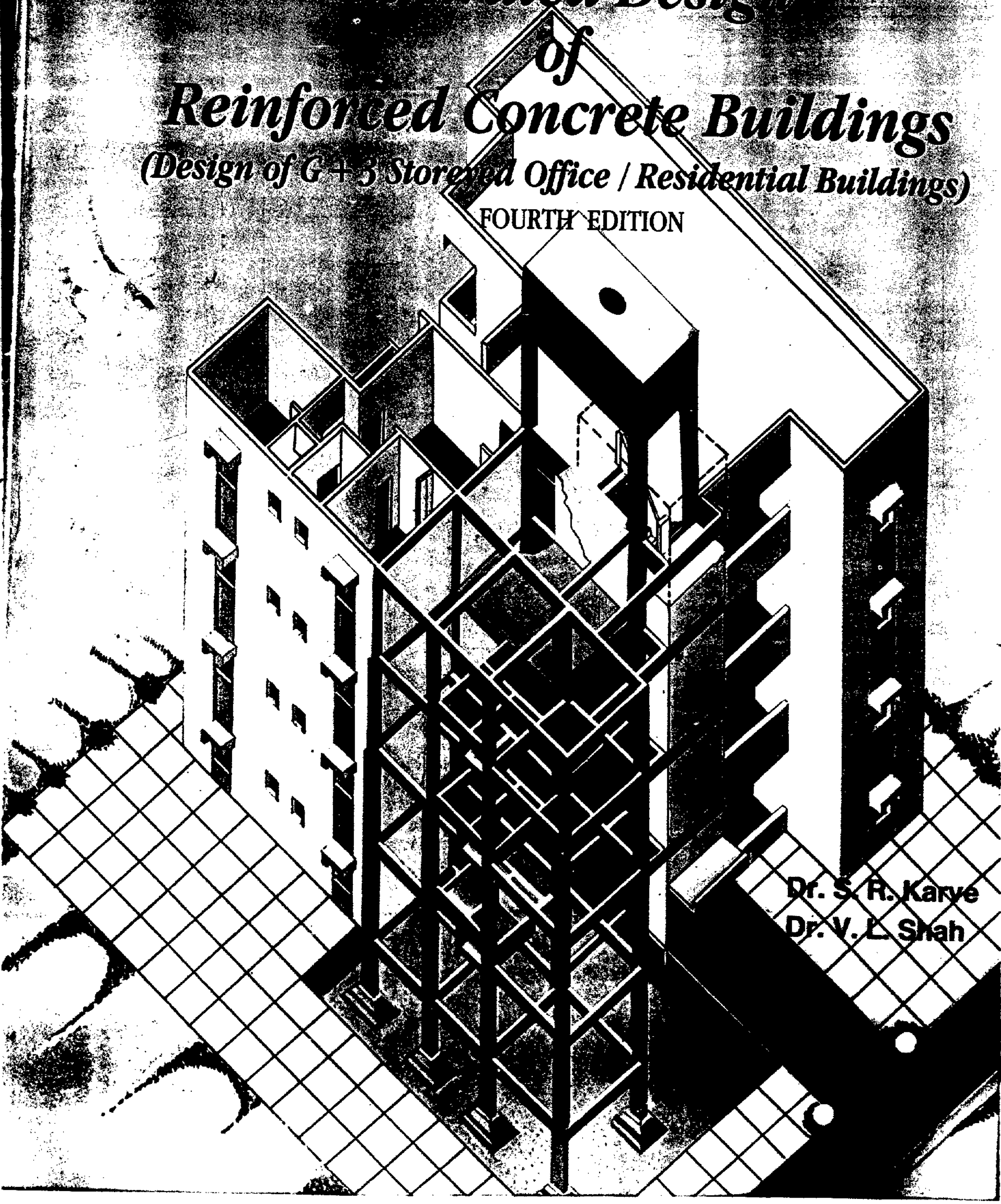
Illustrated Design of Reinforced Concrete Buildings

(Design of G + 3 Storeyed Office / Residential Buildings)

FOURTH EDITION



Dr. S. R. Karve
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Preface to the First Edition

It gives the authors great pleasure and deep sence of satisfaction in presenting this book dealing with "Illustrated Design of Reinforced Concrete Buildings". This Book is an outcome of presistant 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 Theoty & 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 Multistoryed 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 the 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 detail 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 the Second Edition

It is very encouraging that the first edition of the book has been very well received by the Engineers. The book is widely used by consulting Engineers and Civil engineering students of the Engineering Colleges and Technical Institutions. The second edition is revised incorporating some more comments, revised text and many of the printing errors have been corrected.

The author .ecord his gratitude to Dr. L. S. Sane for his continued helpful suggestions. The author further welcomes constructive criticisms and suggestions for its improvement.

Oct. 1990 Dr. V. L. Shah

Preface to the Third Edition

In the third Edition Chapter – IX on Design of R.C. Portal Frame with fixed base and hinged base, along with the detailed drawing, has been added. The footings of these portal frames are good examples of eccentric footing subjected to uniform soil pressure distribution and a concentric footing with linearly varying soil pressure distribtion. The axonometric view of a four storeyed residential building designed in Chapter-VIII has been printed in multicolour on the coverpage to bring out the structural details of reinforced concrete elements and interior details of the building. It is hoped that the New Edition will be more useful to the students, design engineers and field engineers.

April. 1994 Dr. V. L. Shah

Preface to the Fourth Edition

In the fourth edition, the different layouts used for the construction of porches have been discussed and their detailed design and drawings have been included in Chapter-X.

It is hoped that the new edition will be very useful to students, teachers, and the field/ practising engineers. Further suggestions will be greatly appreciated.

November. 1995 Dr. V. L. Shah

Contents

Introduction to Structural Design

1.1 THE DESIGN PROCESS :

Structural design is an art and science of designing, with economy and elegance, a safe, serviceable, and a durable structure.

The entire process of structural planning and design requires not only imagination and conceptual thinking (which form art of designing) but also sound knowledge of science of structural engineering besides knowledge of practical aspects, such as relevant design codes and bye-laws, backed up by ample experience, intuition and judgement.

The process of design commences with planning of a structure, primarily to meet the functional requirements of the user or the client. The requirements proposed by the client may not be well defined. They may be vague and may also be impracticable because he is not aware of the various implications involved in the process of planning and design, and about the limitations and intricacies of structural science. The functional requirements and the aspect of asthetics are looked into normally by an architect while the aspect of safety, serviceability, durability and economy of the structure for its intended use over the life span of the structure are attended to by the structural designer.(Many times,a structural engineer is required to act in capacities of both - the architect and the structural designer).

1.2 STAGES IN STRUCTURAL DESIGN :

The process of structural design involves the following stages.

I Structural Planning   II Estimation of Loads.
III Analysis of Structure.   IV Member Design.
V Drawing, Detailing and Preparation of Schedules.

The first stage of structural planning has been discussed in this chapter and the remaining in the subsequent chapters.

1.3 STRUCTURAL PLANNING :

This involves determination of the form of the structure, the material for the same,the structural system,the layout of its components,the method of analysis, and the philosophy of structural design.

For example, if a large area is to be provided with a cover,the designer is required to decide first the appropriate form and/or system of the covering (roof) structure. He has to fix up whether the roof shall consist of steel roof trusses and girders, or R.C. folded plates, or R. C. shell or a cable-stayed tension structure or a beam-slab grid system, or a prestressed hanging roof, or combination of above. The form and the system will have to be decided from the considerations of functional requirements such as unobstructed area,head-room and also from considerations of economy and asthetics. After deciding the form and the system, the designer is required to select material appropriate to the form. Ofcourse, the choice of material will also be governed by the requirements of asthetics, economy and the availability of the material. Once the form and the material to be used are finalised, the layout of the component members (e.g. positioning of columns, spacing of trusses, or beams, configuration of trusses etc.) will be required to be determined. And finally, the designer will have to choose the realistic design philosophy and the method of analysis appropriate to the structural system and the material used.

Since the scope of this book is restricted to R.C.Buildings, the discussion is limited to the structural planning of R.C.frames.

The principle elements of a R.C.building frame are as follows:
(i) Slabs to cover large area, (ii) Beams to support slabs and walls,
(iii) Columns to support beams,and (iv) Footings to distribute concentrated column loads over a large area of the supporting soil.

After getting an architectural plan of the building,the structural planning of the building frame is done. This involves determination of the following:
(a) Column positions, (b) Beam locations, (c) Spanning of slabs.
(d) Layout and planning of stair, (e) Type of footing.

(a) Positioning of Columns :

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

(1) Columns should preferably be located at or near the corners of a building, and at the intersections of walls, because basically the function of the column is to support beams which are normally placed under the walls to support them. There can, however, be an exception in case of columns in walls on the property line. 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.1.1*. Brackets may be taken out from the column in continuation of cross beams to support walls along the boundary line. Alternatively, a combined footing or a strap footing may be provided.

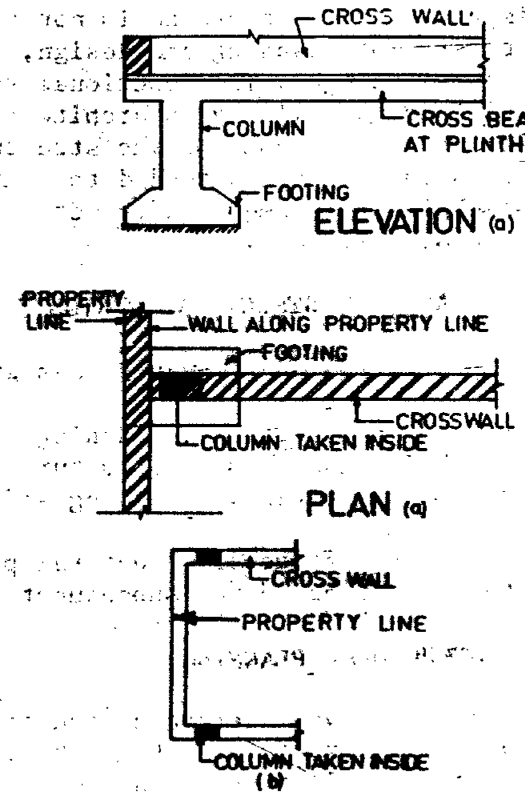


Fig.1.1 Positioning of Columns on property line

(2) When the centre to centre distance between the intersections 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,larger spans of beams should preferably be avoided for economy reasons.This aspect is illustrated in *Fig.1.2*. 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, spans AC and CB will be larger 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 span is reduced and it is required to carry only one concentrated load and that too on central span which further reduces the moment 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 case of multistorey building frames.

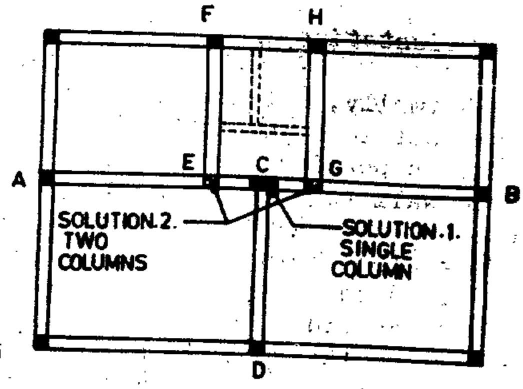


Fig.1.2 Positioning of Columns avoiding large spans for beams

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

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

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

(3) Larger spans of beams shall also be avoided from the consideration of controlling the deflection and cracking. It is well known that the deflection varies directly with the cube of the span and inversely with the cube of the depth D (since the rigidity EI is a function of bD^3). However, for large spans, normally higher L/d ratio is taken to restrict the depth from considerations of economy, headroom, asthetics and psychological effect (a long, heavy, deep beam creates a psychological feeling of a crushing load leading to a fear of collapse). Consequently, increase in D is less than increase in span which results in greater deflection for large span. Therefore, spans of beams (and hence spacing of columns) which require the depth of beam greater than one metre should as far as possible be avoided.

(4) Column should be avoided inside a big hall as it mars the functional utility and the appearance, and obstructs the clear view and the usable space.

(5) 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 lead to offsets from walls and obstruct the floor area.

(6) 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 should be provided instead of two at such a position so as to reduce the beam moment.

Orientation of Columns :

Normally, columns provided in a building are rectangular with width of column not less than the width of the supported beam for effctive load transfer. As far as possible, the width of column should also not exceed the thickness of of the wall to avoid offsets. Restriction on the width of column necessitates the other side (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 guide lines can be useful.

(1) According to requirements of asthetics and utility, projections of columns outside the wall, in the room (and especially at the corner, see *Fig.1.3(a)*) should be avoided as they not only give bad appearance but also obstruct the use of corners, and create problems in placing furniture flush with the wall. The depth of the column shall be 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 usually thinner. Now a days use of 150 mm thick block walls allowed by the construction authorities for outer walls also to get more floor space, has posed this problem for external walls too, because the width of column is required to be kept not less than 225mm to prevent the column from being slender. In such cases 150 mm thick columns may be provided only at the intersection of two walls at right angles where 150 mm side of the column could be matched with one of the walls. See *Fig.1.3.(b)* Such columns should be laterally braced at the lintel level by connecting the columns to lintel in the cross wall. This will reduce the effective length of the column and keep the column short. See *Fig.1.3(c)*. Besides, this solution is possible only for upper two floors since 150 mm thickness may become inadequate for the lower storey columns carrying heavier loads. For this, only alternative is either to use *L* shaped columns at the corners or *T*-shaped columns at the intersection of intermediate cross walls as shown in Fig.1.3(d). Alternatively, spacing of the columns should be considerably reduced so that the load on column at each floor is less and the necessity of large sections for columns does not arise.

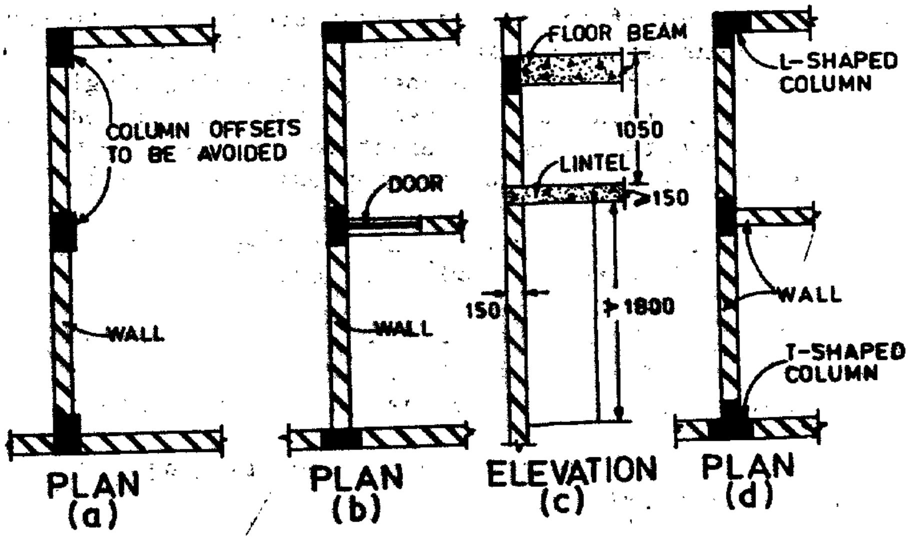


Fig.1.3 Orientation of Columns from Architectural Criteria a) avoiding offsets, b) matching of column width with wall width, c) Bracing of 150 mm thick columns at right angles, d) Use of L & T section columns at corners and at cross wall junctions respectively.

(2) When a column is rigidly connected to beams at right angles, it is required to carry 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 so as to get larger moment resisting capacity (i.e. the depth of the column shall be in the plane of bending). 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. This can be avoided to some extent by limiting the depth of the column but increasing its moment of resistance by increasing the percentage of steel. See *Fig.1.4(a)*.

(3) Also, when the effective length of the column in one plane is greater than that in other plane at right angles (e.g. L_{eff} of a column in a plane frame free to sway is more in the plane of the frame than across it when all frames are laterally braced at the top), the greater dimension shall be in the plane (of the frame) having larger effective length so as to reduce the governing L_{eff}/D ratio and to increase the load carrying capacity of the column. *Fig.1.4(b)*

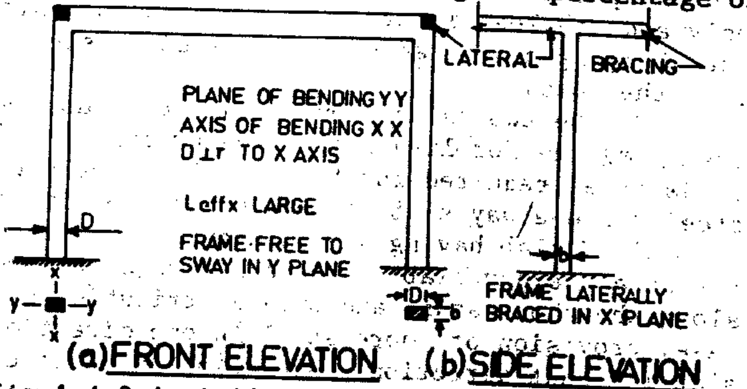


Fig.1.4 Orientation of Columns from stiffness and effective length criteria.

(b) 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. Basic principle in deciding the layout of component members is that heavy loads should be transferred to the foundation along the shortest path.

(2) Since beams are primarily provided to support slabs, its 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 high compared to other components). Therefore, the thickness of slab is required to be kept minimum. The maximum practical thickness 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 case of buildings, with live loads less than 5 kN/sq.m. (i.e. other than warehouses, godowns and heavy duty floors), 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*
Max.Span in metres	1.5	2.0	3.5	4.5	4.5	6.0

(c) Spanning of Slabs :

This is decided by the positions of supporting beams or walls. When the supports are only on opposite sides or only in one direction, then the slab acts as a one-way supported slab. When the slab is supported in two perpendicular directions, it acts as a two-way supported slab. However, the two-way action of slab does not depend only on the manner in which it is supported but also on the aspect ratio L_y/L_x (the ratio of Long span L_y to Short span L_x), the ratio of reinforcement in the two directions (A_{stx}/A_{sty} or m_{ux}/m_{uy}) and the boundary conditions. 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 acts as a two-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 predominent. In practice, however, a slab is designed as two-way only when $L_y/L_x<1.5$.

(2) A two-way slab is generally economical compared to one-way slab because steel along both the spans acts as main steel and transfers the load to all the four supports, while 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.

(3) The two-way action is advantageous essentially for large spans (greater than 3 m) and for live loads greater than 3 kN/sq.m. 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.

(4) A slab having supports on all sides but having $L_y/L_x<2$ can be 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 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 supported over the short edge also, there is a tendancy of the load on the slab by the side of support to get transferred to the nearer support causing tension at top along the supporting edge. Since there does not exist any steel at top across this edge in a one-way slab interconnecting the slab and the side beam, a crack develops at top along that edge. The crack 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.

(5) Spanning of slab is also decided by the necessity of continuity to adjacent slab. For illustration, in *Fig.1.5*, if the slab *S1*, is to be designed as a slab continuous over the support *AB*, then it is necessary that slab *S2* also spans across *AB*. If it is designed as one-way slab spanning only in the direction parallel to *AB*, then the slab *S1* will not get the desired fixity or structural continuity over *AB*. In such case, eventhough 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 *S2*, and *S1* simply acts as a slab freely supported on *AB*.

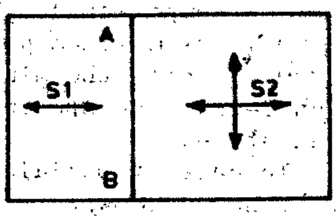


Fig.1.5 Spanning of Slabs Necessity of continuity over Edge.

(6) While deciding the type of slab, whether a cantilever, a simply supported or a continuous slab, it should be borne in mind that (for uniform loading) the maximum bending moment in a cantilever $(m = wL^2/2)$ 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.

Similarly, deflection of a cantilever $(a = (wL^4/8EI) = (48/5)(5wL^4/384EI))$ is 9.6 times that of simply supported slab $(5wL^4/384EI)$ for the same span and load (Besides, additional reduction in deflection is obtained in simply supported slab due to partial fixity at supports). In 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 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.

Thus, in case of an isolated single balcony, S1 in *Fig.1.6(a)* 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 slab as simply supported spanning longitudinally across the transverse end beams instead of as a cantilever slab. 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 (*Lx*) 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 *S1* could be made to span across the floor beam *DG*. (This principle is adopted in counterfort retaining walls by making the vertical stem to span longitudinally across the counterforts instead of transversely (i.e. vertically as cantilever) when the height is large.)

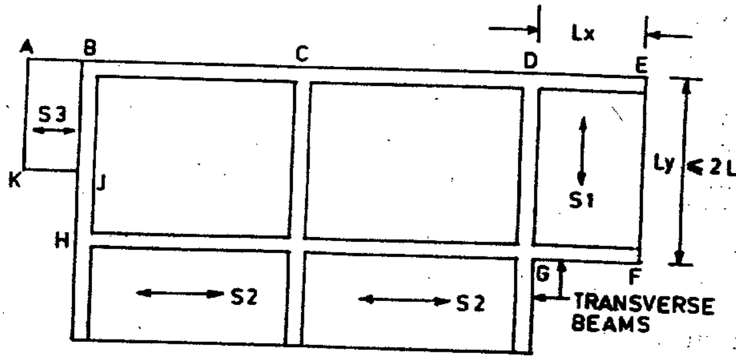


Fig.1.6(a) Spanning of Balcony Slabs Longitudinal and Transverse

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. See *Fig.1.6(a)*. In case of balcony *S3* in this figure which does not extend over the complete length of the room, transverse beam could be made availbale 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 will have to be designed as cantilever because provision of a separate supporting beam at *JK* would induce large twisting moment in beam *BH*.

The presence of a vertical parapet wall at the edge of a balcony makes the cantilever spanning further uneconomical because of additional moments induced by the weight of the parapet acting at the free end as point load and due to horizontal loads acting on edges of vertical wall. See *Fig.1.6(b)*.

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 ofcourse it is supported transversely at top by either a transverse parapet wall or a hand rail.

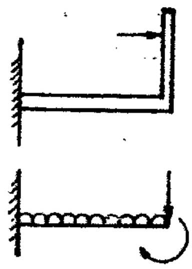


Fig.1.6(b) Parapet Load on Cantilever Balconies.

(7) 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 *S1* in *Fig.1.6(c)* (page 8) is to be provided outside the entrance instead of a column supported porch and that too at a level different (lower) than that of the floor beam *AB*, then adequate anchorage will not be available because slab *S1* cannot be extended inside the hall due to level difference betwen *S1* and *S2*. In such case, the beam *AB* will either be required to be made very deep, with depth equal to level difference between *S1* & *S2*, and canopy slab connected to its bottom, or a seperate beam will have to be provided below *AB* at the level of *S1* if the projection of the canopy is large. In both the cases, these supporting beams will be subjected to very large torsional moment.

An alternative better solution for this would be to provide columns at *C* and *D* from which brackets *CE* and *DF* could be taken out across which *S1* could be spanned longitudinally.

(8) Another common problem in case of balconies is that of a corner balcony *S3*. See ***Fig.1.6(d)***. If balconies *S1* and *S2* are both spanning longitudinally across transverse beams *AB* and *AD*, corner slab *S3* can just be overhanging extensions of slabs *S1* and *S2* with 50 % load transferred in each direction. This is economical. On the other hand, if both *S1* and *S2* are cantilever balconies with no beams at *AB* and *AD*, corner slab *S3* does not get any support except from *S1* and *S2* which themselves are elastic cantilevers. Since the transfer of load of *S3* on to *S1* and *S2* makes the design of *S1* & *S2* further uneconomical and complicated, slab *S3* should be supported by radial bars of minimum 12 mm diameter, and anchored backwards in slab *S4* 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 *B1* and *B2* to prevent lifting of the radial bars as shown in ***Fig.1.6(d)***

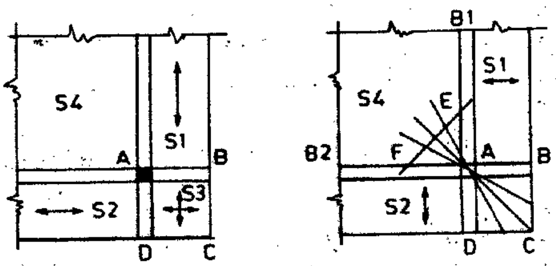


Fig.1.6(d) Corner Balconies.

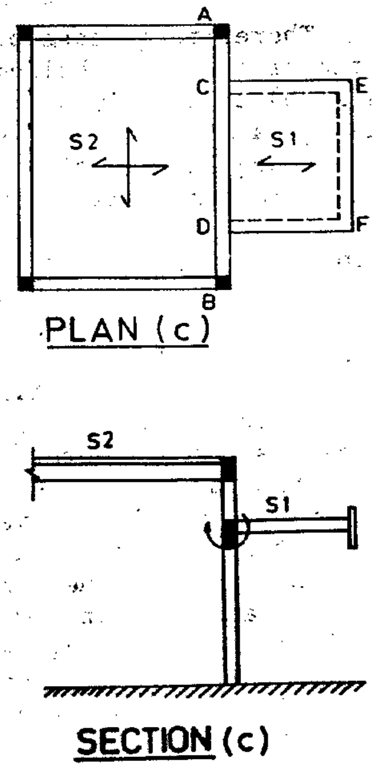


Fig.1.6(c) Anchoring Canopy Slabs

(d) Layout of Stairs :

The type of stair 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 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) Wherever possible, landing beams may be provided at the end of flight to reduce the span. For example, in ***Fig.1.7(a)*** 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* acts as cantilevers which reduce the design moment at midspan giving double benefit and hence this arragement is most economical. Supporting stair slabs along *AB* and *CD* is uneconomical. When the provision of a midlanding beam, say at *EF*, is not possible due to nonavailability of adequate headroom under the landing, the flight may be supported on landing slab itself. The landing slab may be made to span transversly across *AE* and *BF* on walls or on bracket beam taken out from the columns as shown in ***Fig.1.7(b)***.

(4) If the span of stair flight is greater than 4.5 metre, the flight may be supported on a central stringer beam spanning across *AB* & *CD* and the steps of the stair flight cantilevering out from the stringer beam on both sides. This arrangement is asthetically excellent for public buildings like hotels, theatres, banks, etc. See *Fig.1.7(c)*.

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

(e) Choice of Footing Type :

The type of footing depends upon the load carried by the column and the bearing capacity of the supporting soil. For framed structures under study, isolated column footings are normally preferred except in case of soils with very low bearing capacities. If such soil or black cotton soil exists for greater depths, pile foundations can be an appropriate choice. If columns are very closely spaced and the bearing capacity of the soil is low, raft foundation can also be an alternative solution. For a column on a boundary line, a combined footing or a strap footing may be provided.

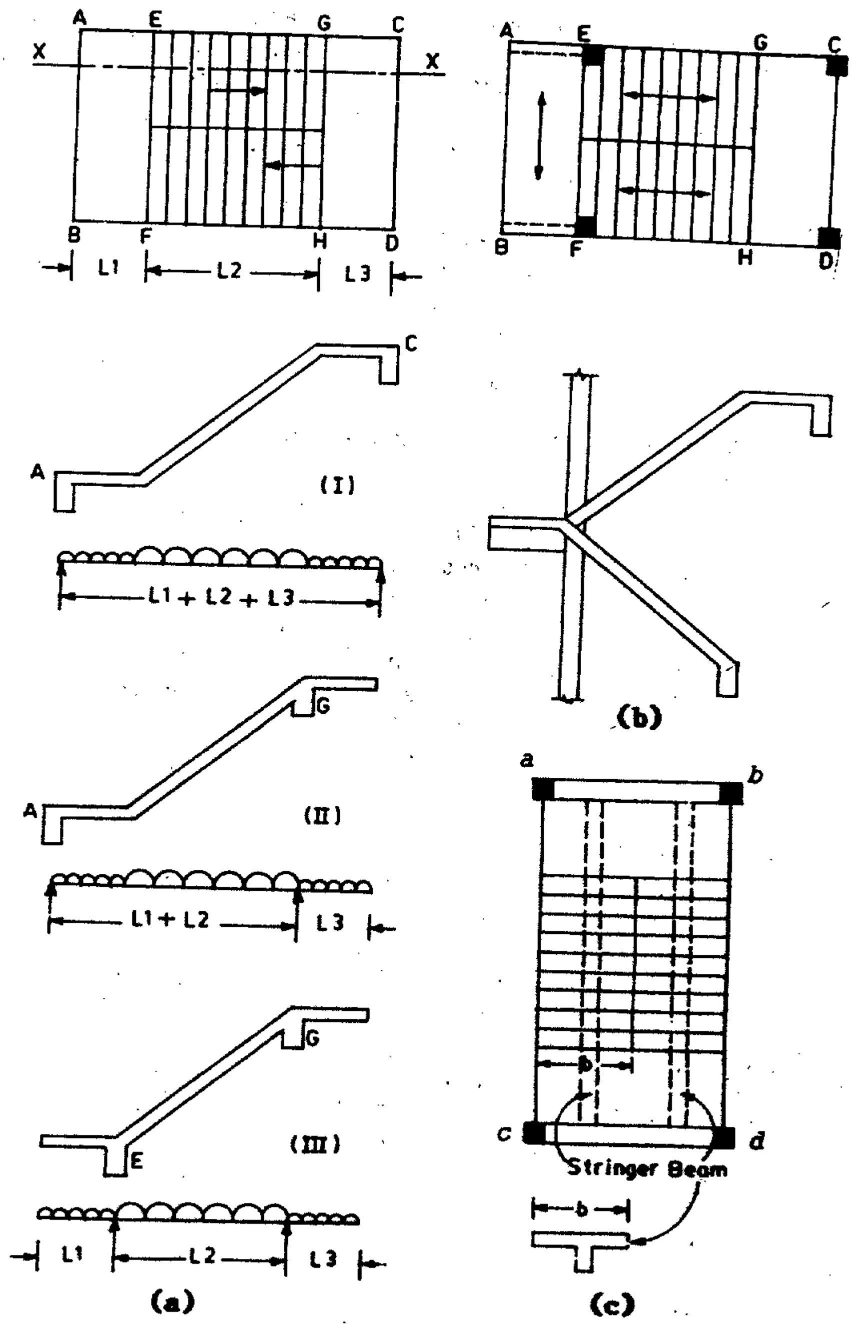


Fig.1.7 Layout and Spanning of Stairs

1.4 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) Scheme as recommended by I.S.696:Code of Practice for General Engineering Drawings. This scheme of marking will be called as *Column Reference Scheme*.

(b) Scheme as recommended by I.S.5525 : Recommendations for detailing of Reinforced Concrete Works. This scheme of marking will be called as a *Grid Reference Scheme*.

(c) Scheme followed by the private sector.

1.4.1 Column Reference Scheme :

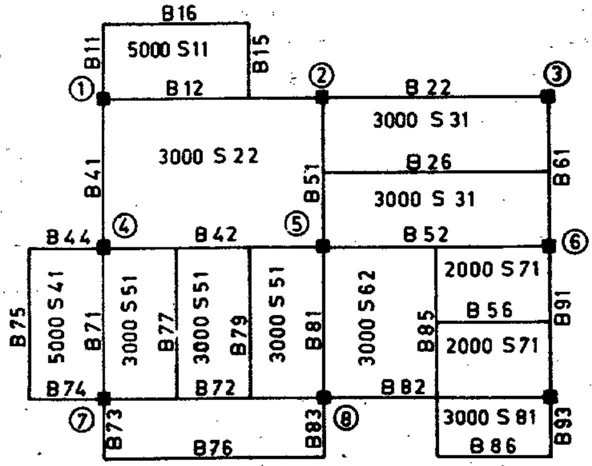


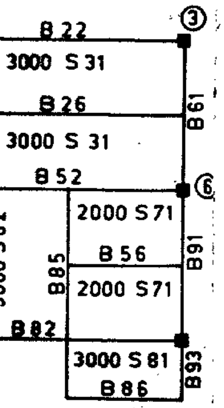
Fig.1.8 Column Reference Scheme

In this scheme, columns are first of all numbered serially starting from the column at top left corner proceeding rightwards and then downwards as shown in Fig.1.8. 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 nothwards 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 *B51* is a beam starting from column No.5 and runing in *Y* direction, while beam *B52* is a beam starting from the same column 5 but running eastwards in *X*-direction. (See ***Fig.1.8***). 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 *Bi5*, *Bi7*, *Bi9* etc. while those in *X* direction are designated by even numberes such as *Bi6*, *Bi8* etc. See ***Fig.1.8***.

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 private sector. The typical marking plan given by the Code (I.S.696) deals with steel frame work and, therefore, does not specify marking for slabs. The Government Departments which adopt this marking scheme, designate slabs as wSij 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, *200S32* indicates slab of category No.3 designed as two-way for a live load of 200 kg/m² . 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. See also Fig.7.1.

1.4.2 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***. 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*** 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 rightwards (baywise) sequentially. Slabs are designated serially as S_{b1}, S_{b2}, starting from panel in top left corner, proceeding vertically downwards baywise and then rightwards. See Fig. 1.9. 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 favoured (especially writing suffixes *m* and *b* to mark beam and slab respectively, is considered to be superfluous).



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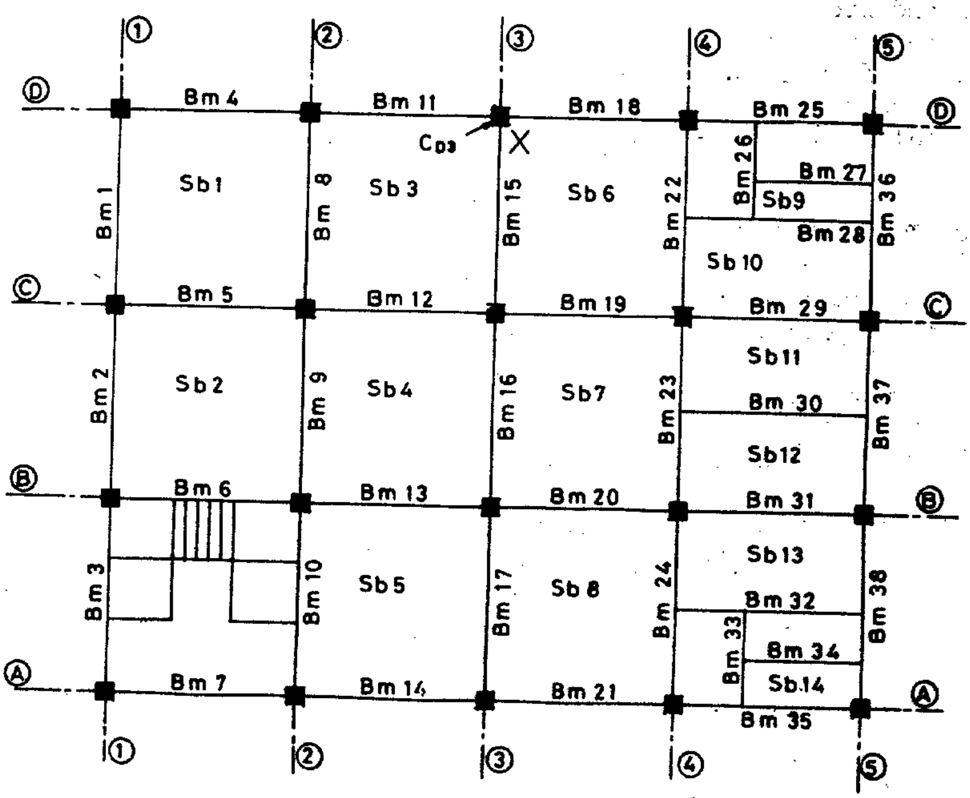
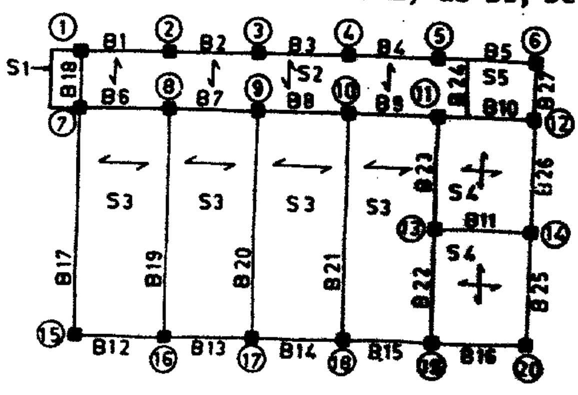


Fig.1.9 Grid Reference Scheme

1.4.3 Scheme used in Private Sector :

In this scheme, the columns are marked serially as $C1$,$C2$,$C3$ etc. or by cled numbers such as 1 , 2 , 3 etc. by the side of the column starting r from top (or bottom) left corner and moving rightwards and downwards (or ds as the case may be). See *Fig.1.10*. Beams are marked serially as *B1*, *B2*, tarting from first column and moving wards first and then downwards (or ds as the case may be), thus numbe- first all the beams in horizontal or ection and then numbering upwards in ection starting from leftmost beams own in *Fig.1.10*. However, the slabs ot marked serially but are marked ding to their categories based on n specifications (namely the thick- diameter, and spacing of reinforce- along two perpendicular spans). facilitates scheduling of slabs. theless, it requires grouping of panel first having nearly equal ,end conditions and the load so that



Scheme of Marking Frame Components used in Private Sector.
Fig.1.10 Marking Scheme used in Private Sector

ories of slabs required to be designed are reduced to a minimum. The span- of slabs is shown, by arrows on the plan and besides specifying seperately e schedules under remarks column.

At present, the loads for which the slabs are designed are many times not on the drawings. However, since these drawings form a permanent record 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.)

1.5 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)*

Working stress method used over decades is now practically outdated. It is not used at all in many advanced countries of the world because of its inherent drawbacks. The latest I.S.Code gives emphasis on Limit State method which is the modified version of Ultimate load method. It is a judicious amalgamation of *WSM* and *ULM* removing all drawbacks of both methods but maintaining their good points. It is also based on sound scientific principles backed up by 25 years of research. The limit state method has proved to have an edge over the working stress design from the view point of economy. Consequently, there is no point in sticking to Working stress method of design hereafter. For detailed comparison of the three methods, refer to author's text book.

In this book, therefore, the limit state philosophy of design has been followed throughout using SI units. The Government departments and large consulting firms are switching over to this method. It is high time that even small practicing firms and engineers in the private sector should switchover to this philosophy even in routine design of R.C. buildings. This will enable them freely use the computer programs now readily available for design based on limit state method.

Chapter – 2
Loads and Materials

2.1. INTRODUCTION : DEFINITIONS

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. For such a quantity of varying nature, it is necessary to arrive at a single representative value. Such a value is known as Characteristic value. The value to be taken in design which provides appropriate or desired margin of safety is known as Design value. Ratio of the two greater than unity is known as Partial Factor of Safety.

2.1.1 Characteristic Load (F_k)

It is defined as that value of the load which has 95% probability of not being exceeded during the life time of the structure. It can be determined using statistical probabilistic principles from the mean value and standard deviation. However,this requires large amount of statistical data. But since such data are not available at present to express the load in statistical form,Code recommends to take working loads or service loads, decided in the past using the principle of equivalent load giving the same maximum effect and which are based on past experience and judgement, as the characteristic loads..

2.1.2 Design Load (F_d)

It is given by $F_d = \gamma_f . F_k$ where F_k = characteristic load,

and γ_f = partial safety factor for load (>1)

For values of γ_f, please refer to Table 12 of Code .

2.2 TYPES OF LOADS

The various types of loads acting on the structure which need consideration in building design are as follows:

(a) Dead load, (b) Live load, (c) Other loads.

2.2.1 Dead Loads

It includes (a) Self weight, (b) Weights of finishes
(c) Weights of partitions, walls, grills etc.

The unit weight of materials,weight of structural components such as slabs, beams,columns,walls,grills, etc. and weight of floor and roof finishes are given in Table 2.1. Weight of members of common sizes are given in Table 2.2. For details, refer to I.S.1911 or Table 1.1 and 1.2 of Author's Handbook.

Table-2.1 : Dead Loads

(a) Unit Weight of Materials :

Material	Unit Weight	:	Material	Unit Weight
Asbestos sheets	140-160 kN/sq.m	:	Earth	16-18 kN/cu.m
Brickwork	20 kN/cu.m	:	Mortar, plaster	20 kN/cu.m
Concrete-Plain	24 kN/cu.m	:	Steel	78.5 kN/cu.m
Concrete-Reinforced	25 kN/cu.m	:	Water	10 kN/cu.m

(b) Unit Weight of Building Components :

Component	Formula	
Beams and Columns:	25bD kN/m (b & D in metre);	b = width of beam/column
Slabs	25D kN/m/m (D in metre)	; D = Depth of beam/column or
Brick walls	20B kN/m/m (B in metre)	; B = width of wall [slab
R.C.Grills,parapets	25t kN/m/m (t in metre)	; t = thickness of parapet

(c) Weights of Finishes :

Floor finish including weight of tile,morter bed,underneath ceiling plaster
-for rooms 0.75 to 1 kN/sq.m
-for sanitary blocks (Indian type) 1.5 to 2.5 kN/sq.m
Roof finish including weight of waterproofing course- 1 to 2.5 kN/sq.m

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Note :1) Appropriate reduction may be made in weight of grill for openings.
2) Some designers allow appropriate reduction in wall load for large openings in external walls also. However, no such reduction may be made in internal walls because, normally,there are no windows in internal walls and reduction in load due to door openings is offset by increase in wall loads due to wall mountings, fixtures, cupboards,shelves etc.
Separate load shall be taken for lofts in rooms projecting outside the walls, and for lofts over sanitary blocks.

Table-2.2 : Weights of Members of Common Sizes

Slab Depth in mm	100	110	120	130	140	150	160	180	200
Weight in kN/m	2.50	2.75	3.00	3.25	3.50	3.75	4.00	4.50	5.00

Wall	Thickness	Height-1 metre	3 metres	
of Solid Concrete Blocks	- 150+25 mm	4.5	13.5	kN/m
including plaster	- 200+25 mm	5	15	kN/m
of Hollow Concrete Blocks	- 150+25 mm	3	9	kN/m
including plaster	- 200+25 mm	4	12	kN/m
of Brick including plaster	- 150 mm (4½+1½")	3	9	kN/m
	- 225+25 mm (9 +1 ")	5	15	kN/m
Beams and Columns 150 mm wide	- Depth 300 to 600 mm	1 to 2.5 kN/m apprpx.		
225 mm wide	- Depth 300 to 900 mm	2 to 5 kN/m approx.		
300 mm wide	- Depth 300 to 900 mm	2.5 to 7 kN/m approx.		

Note : For residential buildings with spans less than 3.5 metres,the total loads are roughly taken as follows:
-for rooms = 6 kN/sq.m (600 kg/sq.m) which consists of--
weight of 125 mm (5") thick slab = 3.125 kN/sq.m (312.5 kg/sq.m)
+ floor finish= .825 kN/sq.m (82.5 kg/sq.m)
+ live load = 2.0 kN/sq.m (200 kg/sq.m).
-for cantilever balconies = 8 kN/sq.m (800 kg/sq.m)
-for stairs = 10 kN/sq.m (1000 kg/sq.m).

2.2.2 Live Loads

Live loads on roofs and on floors are taken according to IS:875. These are given in Table 2.3 below.

Table-2.3 : Live Loads on Floors

(according to IS:875-1964)

Loading Class (1)	Type of Floors (2)	Minimum Live Load - N/m² (3)	Alternative Minimum Live Load (4)
2000	Floors in dwelling houses, tenements, hospital wards, bed-rooms and private sitting rooms in hostels & dormitories	2000	Subject to a minimum total load of 2.5 times the values in column 3 for any given slab panel and 6 times the values in column 3 for any given beam. This total load shall be assumed uniformly distributed on the entire area of the slab panel or the entire length of beam.
2500	Office floors other than entrance halls, floors of light workrooms.	*2500 *4000	
3000	Floors of banking halls, office entrance halls and reading rooms.	3000	
4000	Shop floors used for the display and sale of merchandise; floors of workrooms generally; floors of class rooms in schools, floors or places of assembly with fixed seating, restaurants; circulation space in machineray halls, power stations etc. which are not occupied by plant or equipment.	4000	
5000	Floors of warehouses, workshops, factories, and other buildings or parts of buildings of similar category for light weight loads; office floors for storage and filing purposes; floors of places of assembly without fixed seating; public rooms in hotels, dance halls, waiting halls etc.	5000	**Note :** **The Live Loads as per IS : 875 (Part 2) - 1987 are given in Appendix - G**
7500	Floors of warehouses, workshops, factories and other buildings or parts of buildings of similar category for medium weight loads.	7500	
10000	Floors of warehouses, workshops, factories and other buildings or parts of buildings of similar category for heavy weight loads, floors of book stores and libraries, roofs of pavement lights over basements projecting under the public footpath.	10000	
Garage Light	Floors used for garages for vehicles not exceeding 25 kN gross weight : Slabs Beams	4000 2500	or the worst combination of actual wheel loads whichever is greater.
Garage Heavy	Floors used for garages for vehicles not exceeding 40 kN gross weight.	7500	Subject to a minimum of one and a half times maximum wheel load but not less than 9 kN, considered to be distributed over 750 mm².
Stairs	Stairs, landings and corridors for Class 2000 loading but not liable to overcrowding.	3000	Subject to a minimum of 1.30 kN concentrated load at the unsupported end of each step for stairs constructed out of structurally independent cantilever steps.
	Stairs, landing and corridors for Class 2000 loading but liable to overcrowing, and for all other classes.	5000	
Balconies	Balconies not liable to overcrowding		
	For class 2000 loading	3000	
	For all other classes	5000	
	Balconies liable to overcrowding	5000	

*The lower value of 2500 N/m² should be taken where storage facilities are provided and the higher value of 4000 N/m² should be taken where such provisons are lacking.

2.2.3 Other Loads

Besides dead load and live load, the wind load and earthquake load are the main loads required to be considered in building design especially when the height of building exceeds two times the dimensions transverse to exposed wind face. Such cases occur, normally, when the number of storeys exceed 4. Wind load is not critical in buildings of height upto 4 storeys and which have walls in both directions in between columns and beams. Since, the scope of this book is restricted to design of such buildings only,the action of wind is not considered. Design of building for earthquake is also outside the scope of this book.

Note : The attention of readers is invited to the fact that because of purely transitory nature of wind, code allows 33% increase in permissible stresses in working stress design when effect of wind is considered.In limit state design the factor for design load is reduced to 1.2 (DL+LL+WL) when wind load is considered, as against 1.5(DL+LL) when wind load is not considered. For buildings of less than 4 storeys, the dead load of a concrete structure is so large compared to wind load that the actual increase in stresses due to wind is less than the allowance given in design by the Code.

2.3 CRITICAL LOAD COMBINATIONS, ARRANGEMENTS AND PARTIAL SAFETY FACTORS

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 upto 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 and minimum loads.For this,Code prescribes different load factors as given below.

Maximum load w = 1.5 (DL+LL)
Minimum load w = 0.9 DL (minimum LL being zero)

Critical Loading Arrangements :

In case of design of beams and slabs, we are normally interested in determination of (a) maximum support moment, (b) maximum span moment and, (c) maximum support shear.

For maximum support moment and maximum support shear, two spans adjacent to support under consideration shall be fully loaded (i.e., they shall carry maximum design load 1.5(DL+LL) in limit state method), and there will be minimum load (.9DL) on all other spans. See Fig.2.1.

For maximum span moment, the span under consideration and all other alternate spans shall be fully loaded (i.e. they shall carry maximum design load 1.5(DL+LL) in limit state design) and all other spans will carry minimum dead load (.9DL) only. See Fig.2.2.

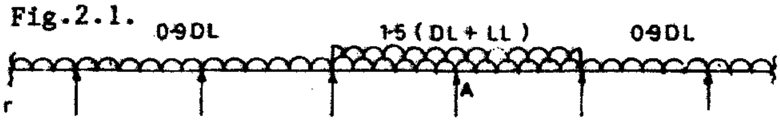


Fig.2.1 Loading for maximum negative moment at support A according to Code

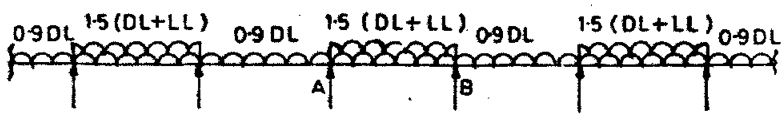


Fig.2.2 Loading arrangement for maximum positive span moment (say span AB)

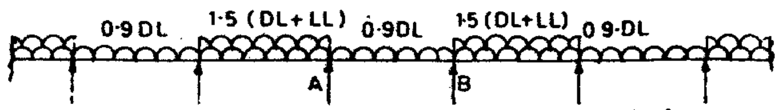


Fig.2.3 Loading arrangement for maximum negative moment in span (say span AB)

For getting maximum region of negative moment in any span,that span and all other alternate spans shall carry minimum load (i.e. 0.9DL) and all remaining spans shall carry maximum design loads (i.e.1.5(DL+LL)).See Fig.2.3. For further details, refer to Sec.3.5.4 of Author's Textbook.

2.4 BEARING CAPACITY OF SOIL

For designing the foundations,bearing capacity of supporting soil is required. The values of bearing capacity of different soils, according to IS:1904 are given in Table 2.4 below.

Table-2.4 : Safe Bearing Capacity of Different Soils

Non-Cohesive Soils		# Cohesive Soils	
Type of Soil	Values in kN/sq.m	Type of Soil	Values in kN/sq.m
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 moderate 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 material removed from above the bearing level, that is, the base of the foundation.

(b) For non-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. If the water table is below the bearing surface of the soil at a distance at least equal to the width of the foundation, 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 50X50X700mm with a sharp point. The picket shall be pushed vertically into the soil by the full weight of person weighing at least 70kg. 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 and expert advice shall be sought.

2.5 PROPERTIES OF CONCRETE

2.5.1 Grade of Concrete

Concrete is known by its grade which is designated as M15, M20 etc in which letter M refers to concrete Mix and the number 15, 20 etc. denotes the charac- teristic compressive strength of concrete (f_{ck}) in N/mm^2. Thus,concrete is known by its compressive strength. In R.C.work, following three grades of concrete are common.

New nomenclature	M 15,	M 20,	M 25 for f_{ck}	in N/sq.mm.
Old nomenclature	M 150,	M 200,	M 250 for f_{ck}	in kg/sq.cm.

2.5.2 Compressive Strength

Like load,the strength of concrete is also a quantity which varies conside- rably for the same concrete mix.Therefore,a single representative value,known as characteristic strength,is arrived at using statistical probabilistic principles.

(a) Characteristic Strength (f_k) :

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). For the procedure for determination of characteristic strength refer to Sect.2.8.

(b) Characteristic Strength of Concrete Test-Specimen (f_{ck}) :

The strength of concrete is obtained by testing a concrete specimen of specified shape and size under specified condition. Test specimens are,normally, cubes or cylinders. In India, cube of size 150mm is taken as the test specimen.

Thus,characteristic strength of concrete is defined as the test strength of

- a cube (having h/d ratio = 1 where h is height, and d is transverse dimension, of the test specimen)
- of size 150mm,
- under uniaxial compression,
- at 28 days of age (28 days of curing after casting).

It may be noted from above specifications that the test strength is a func- tion of shape (h/d ratio) and size of test specimen, the state of stress in the specimen,and the age of curing of the test specimen.For further details of obta- ining the test strength under different conditions,refer to Sect.2.2 of Authors' Textbook.For sampling,testing and acceptance criteria,see Sect.2.8 of this book.

(c) Characteristic Strength of Concrete in Member :

It may be noted that the strength of concrete cube does not truly represent the strength of concrete in member because other factors noted earlier, namely, the shape effect, the size effect, the prism effect (ratio h/d 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 member is taken as 0.67 times the strength of concrete cube.

(d) Design Strength (f_d) & Partial Safety Factor (γ_m) for Material Strength

The strength to be taken for the purpose of design is known as design strength and is given by

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

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,

γ_m = 1.5 for concrete, and = 1.15 for steel.

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

2.5.3 Tensile Strength (f_{cr})

The flexural tensile strength or the modulus of rupture or the cracking strength of concrete as given by the Code is as follows:

$$f_{cr} = 0.7\sqrt{f_{ck}} \quad \text{N/sq.mm}$$

2.5.4 Short-term Modulus of Elasticity (E_c)

According to the Code, short-term modulus of elasticity of concrete at 28 days age is given by

$$E_c = 5700\sqrt{f_{ck}} \quad \text{N/sq.mm}$$

2.5.5 Creep

It is defined as the continued deformation under sustained load. It is given by a creep coefficient defined by

$$\theta = \text{creep strain/Elastic strain} = \varepsilon_{cc}/\varepsilon_i$$

Creep strain ε_{cc} depends primarily on the duration of sustained loading. Practically,75% of the total creep occurs in first 6 months and 100% in 5 years. According to the Code, value of ultimate creep coefficient is 1.6, which means that the total creep strain after 5 years is 1.6 times the initial elastic strain at 28 days.

Since, the effect of creep is to increase the total strain ($\varepsilon_i+\varepsilon_{cc}$) keeping the stress unchanged, the modulus of elasticity of concrete is reduced (and thus the modulur ratio E_s/E_c increases with time.)

2.5.6 Shrinkage

It is defined as the volumetric contraction under zero stress. It depends mainly on the duration of exposure. Just like creep,it also occurs to the extent of 75% of total or ultimate shrinkage strain within first 6 months and remaining 25% in remaining 4½ years. 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} The I.S.Code gives ultimate shrinkage strain ε_{cs}= 0.0003.

2.5.7 Longterm Modulus of Elasticity (E_{ce})

As seen earlier, 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

$$E_{ce} = \sigma_i/(\varepsilon_i+\varepsilon_{cc}) = (\sigma_i/\varepsilon_i)/(1+\varepsilon_{cc}/\varepsilon_i) = E_c/(1+\theta)$$

Effect of the this reduction in E_c 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 changes with time. Thus, working stress method which takes single value of modular ratio m does not represent the true strength and behaviour of concrete members.

2.5.8 Modular Ratio (m)

It is the ratio $m = E_s / E_c$
where E_s = modulus of elasticity of steel = $2X10^5$ N/mm² ($2X10^6$ kg/cm²)
E_c = modulus of elasticity of concrete.
For working stress design, I.S.Code recommends the value of $m = 280/(3X\sigma_{cbc})$
where σ_{cbc} = permissible stress in Concrete in Bending Compression.
This value of m takes into account only partially the effect of creep and shrinkage. The short-term modular ratio is based on $E_c = 5700\sqrt{f_{ck}}$ and long-term modular ratio is based on $E_{ce} = E_c/(1+\theta)$.

Table-2.5 : Modulus of Elasticity of Concrete and Modular Ratio

Concrete Grade	σ_{cbc} N/mm²	*Modulus of Elasticity of Concrete* Short-term E_c	Long-term E_{ce}	*Modular Ratio* Short-term	Long-term	*WSM*
M15	5	22075	8490	9.06	23.56	18.67
M20	7	25490	9805	7.85	20.40	13.33

2.5.9 Stress-Strain Curve

The typical stress-strain curve for concrete is shown in Fig.2.4.Curve 1 is actual curve for characteristic cube strength, Curve 2 is idealised curve for characteristic cube strength, Curve 3 is idealised curve for characteristic strength of concrete in members. Curve 4 is idealised curve for design strength of concrete in member.

Idealised curves are parabolic upto a strain =0.002 and straight line line beyond with a constant value of stress upto an ultimate (crushing) strain of ϵ_{cu} = .0035.

The equation for the parabolic part is given by
$f_c = 0.45f_{ck}[(2\epsilon_c/\epsilon_{cy})-(\epsilon_c/\epsilon_{cy})^2]$ where
$\epsilon_{cy} = 0.002$..'.$f_c = 450\ \epsilon_c(1-250\times\epsilon_c)*f_{ck}$

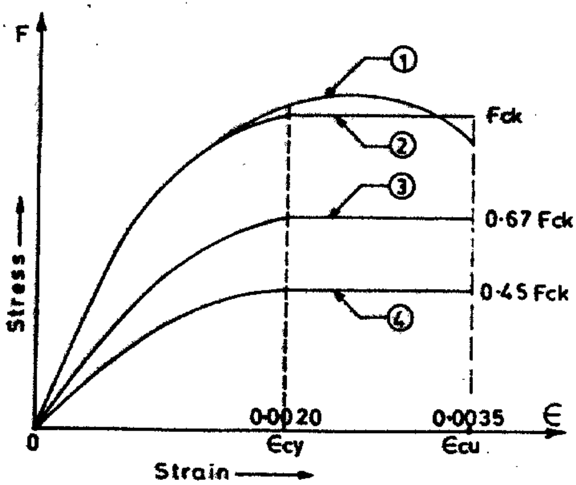


Fig. 2.4 Stress-Strain Curve for Concrete

2.6 CONCRETE MIX PROPORTIONING

The structural designer specifies the strength and properties of concrete assumed in design.The engineer is required to proportion the various ingradients making concrete (namely,cement,sand,coarse aggregate,and water) so that the resulting mix has proper workability for placing and gives the desired strength.

The proportioning is done by any of the following ways. (a) By designing concrete mix; such a concrete is called Design mix concrete. (b) By adopting nominal concrete mix; such a concrete is called Nominal mix concrete .

The former is used for large and important works while the later is used for medium type routine concrete construction with concrete grade lower than M20. The method of of concrete mix design is given in IS:10262 -1982. Normally, it is desirable to proportion the ingradients by weight.However,for small routine jobs, it is conveniently being done by volume. For general guidance, the quantities of concrete constituents for 1 cu.m of compact concrete are given in Table 2.6.

Table-2.6 : Approximate Quantities of Concrete Constituents by Volume per Cu.m of Compacted Concrete

Mix	Water/cement Ratio	Water in litres/bag	Cement in bags	Sand dry in cu.m	Sand moist in cu.m	Coarse Aggregate broken stone cu.m
M15	0.55	27.5	6.2	0.425	0.51	0.854
M20	0.42	21.0	8.1	0.410	0.50	0.824

Notes: 1) Weight of one bag of cement = 0.5 kN (50 kg).
2) Assumptions :(a) Voids in sand 40%, broken stone 45%, specific gravity of cement = 3.1. (b) Bulking of moist sand = 20%.

2.7 CURING PERIOD AND STRIPPING TIME FOR STRIKING OF FORMWORK

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 2.7. 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-2.7 : Stripping Time for Striking of Formwork :

	Member	*Period*
1	Walls, columns and vertical faces of all structural members.	24 to 48 hours as decided by the engineer incharge
2	Slabs (props left under)	3 days
3	Beam soffits (props left under)	7 days
4	Removal of props under slabs :	
	i) spanning upto 4.5m	7 days
	ii) spanning over 4.5m	14 days
5	Removal of props under beams and arches.	
	i) spanning upto 6m	14 days
	ii) spanning over 6m	21 days

2.8 DETERMINATION OF CHARACTERISTIC STRENGTH

For determining the characteristic strength of concrete, the Code lays down certain requirements for sampling,number of test specimens to be taken,procedure for determining the standard deviation (representing workmanship of concrete manufacture, and the formula relating characteristic strength with mean strength and standard deviation.)

(a) Sampling : The samples may be taken so that each concrete batch has reasonable chance of being tested. The sampling shall be spread over the entire period of concreting and may be taken at random in case of continuous concreting with the same mix.The frequency of sampling will depend upon the nature of work, the volume of concrete, and importance of the location of the component member from the viewpoint of stress condition. Also, it will be appropriate to have higher rate of sampling and testing at the earlier part of the work to establish 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 volume of concrete as shown in Table 2.8.

Table-2.8 : Number of Samples for Testing

Quantity of Concrete work in cu.m.	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 one additional sample for each additional volume of 50 cu.m. or part there of.

(b) Test-Specimen : 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 individual variation shall not be more than ±15 % of the average.

(c) Standard Deviation : For determination of standard deviation, at least 30 test results are required from the same mix. The standard deviaion will be obtained from the following relation:

$$s = \sqrt{\Sigma\Delta^2 / (n-1)}$$

where Δ = deviation of test result from mean = $(f-f_m)$

f_m = mean value of n number of test results.

In absence of sufficient test results, the code prescribes the following values for standard deviation.

Table-2.9 : Standard Deviation for Concrete Strength

Concrete Grade	M15	M20
Standard deviation in N/sq.mm	3.5	4.6

When past records of similar mix or grade exists and consistancy in workmanship is observed, the standard deviation obtained from these records may be allowed.

(d) Characteristic Strength : It is given by the relation

$f_{ck} = f_m - 1.64$ S where f_m = mean strength, S = standard deviation, and 1.64 is the value corresponding to acceptable probability of 5 % for non-achievement of the value of f_{ck}.

Thus,average or mean strength f_m required to get the desired characteristic strength f_{ck} is given by $f_m = f_{ck} + 1.64$ S

For example, for getting f_{ck} = 15 N/sq.mm for M15 grade,

required f_m = 15+1.64x3.5 = 20.7 N/sq.mm.

However, if past records assure a standard deviation of say 2.5 N/sq.mm,

the required f_m = 15+1.64x2.5 = 19.1 N/sq.mm, and it can be accepted as concrete of grade M15.

Exact acceptance criteria as prescribed by the Code are given in subsequent section.

2.9 ACCEPTANCE CRITERIA FOR CONCRETE

(a) The concrete shall be deemed to comply with the strength requirements if:

(i) every sample has a test strength not less than the characteristic value,or

(ii)the strength of one or more samples, though less than the characteristic value is, in each case, but shall not be less than the greater of

$f_{ck} - 1.35$ S and $0.8f_{ck}$;

and the average value of all the samples is not less than

$f_{ck} + (1.65-1.65/\sqrt{n})$ x S

(b) The concrete shall be deemed not to comply with the strength requirements if:

(i) the strength of any sample is less than the greater of $(f_{ck} - 1.35$ S) and $0.8f_{ck}$; or

(ii)the average strength of all the samples is less than $f_{ck} + (1.65-3/\sqrt{n})$.S

(c) Concrete which does not meet the requirement as specified in (a) above but has a strength greater than that required by (b) above, may be accepted as being structurally adequate without further testing, at the discretion of the engineer in charge.
(d) If the concrete is deemed not to comply the requirements of (b) above, the structural adequacy of the affected part shall be investigated by core test or load test and consequential action, as needed, shall be taken.

2.10 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).

(a) Grade of Steel:

Grade of steel is known by its characteristic yield strength and is designated as Fe250, Fe415, and Fe500 where 'Fe' stands for Ferrous metal and the number following it represents guaranteed yield strength in N/sq.mm. At present, steel is commercially available in the above three grades.

(b) Type of Bars :

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 (of grades Fe415 and Fe500). HYSD bars have ribs, lugs on their surfaces. They are available in two types.
- Hot rolled bars conforming to I.S.1139,and
- Cold twisted bars, commercially manufactured under trade name "TORsteel" and conforming to IS:1786.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 a 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. See Fig.2.5.

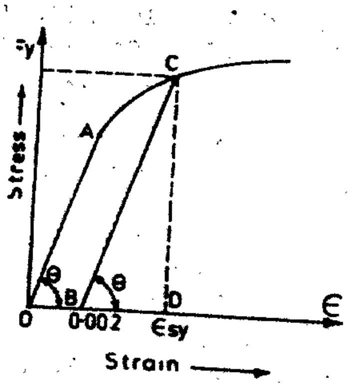


Fig.2.5 Stress-Strain of HYSD bars.

(c) 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-2.10(a) for ready reference.

Table-2.10(a) : Specifications of Reinforcing Bars .

Bar Type	*Grade of Steel*	*Yield Stress *N/sq.mm*	*Min.% elongation at failure*	*Relevant I.S. Code*
M.S. Round	Fe250	250	23	IS : 432 - 1982
HYSD-ToR40	Fe415	415	14.5	IS : 1786 - 1985
HYSD-ToR50	Fe500	500	12	IS : 1786 - 1985

(d) Stress Strain Relation :

For steel of grade Fe 250, $f_s = \varepsilon_s . E_s$.
For steel of grades Fe 415 and Fe 500,
$f_s = \varepsilon_s . E_s$ upto a strain of .00144.
For strains above .00144, refer to Table 2.10(b) below.

Table-2.10(b) : Values of Stress f_s (N/sq.mm) for different values of Strains * for Steel of 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	348.46	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	301.96	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	258	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

Chapter – 3
Analysis

This chapter deals 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).

However, 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 behaviour of the members and structures at working loads. It is based on the following assumptions.

(i) Relation between force and displacement is linear. (i.e. Hooke'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 Method.

(ii) Relaxation /Iterative Methods:
 (a) Moment Distribution Method, (b) Kani's Method.

(iii) Matrix Methods:
 (a) Stiffness Method, (b) Flexibility Method, (c) Finite Element Method

(iv) Computer Methods:
 (a) Matrix Method, (b) Finite Element Method, (c) Finite Difference Method.

(v) Approximate Methods:
 (a) Substitute Frame Method, (b) Cantilever Method, (c) Portal Method

(vi) Coefficient Method

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 when the computer is not available.

Method of Consistent Deformations, Moment Distribution Method, Kani's Method, Approximate Methods, and Coefficient Method come under this group. 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. Use of Approximate methods using stiffness approach on Personal Computer is considered best choice in design of R.C.Buildings. 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.

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 moments, shear, 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, fixed, and continuous beams and slabs, and single bay-single storeyed rectangular portal frames. Their coefficients are presented in Appendix B-1.

Since in many cases, the coefficients make use of redistribution of moments, it is necessary to know the limit analysis or the ultimate load analysis of R.C. structures, and redistribution of moments. This is ,therefore, cursorily reviewed in the next sub-section.

3.1.2 Limit Analysis

It is an analysis dealing with the study of strength and behaviour 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 behaviour of which is characterised 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 behaviour at collapse is absolutely necessary to know the exact margin of safety.

Consider the behaviour of a simply supported beam subjected to a gradually increasing uniformly distributed load See Fig.3.1.1. When, at a particular load, the maximum moment M_u at the centre reaches the ultimate moment Capacity of the beam, plastic hinge is developed at this section due to plastification of concrete in compression, and cracking of concrete accompanied by yielding of steel in tension. This divides the beam into two segments and causes the beam to collapse. The load w_u causing collapse is known as collapse load or ultimate load. Thus, $M_u = w_u L^2/8$. The ultimate load w_u is taken equal to 1.5 times the working load considering 1.5 as the load safety factor as prescribed by the Code. The beam section is so designed that it develops an ultimate moment capacity M_{ur} equal to M_u.

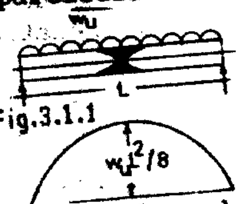


Fig.3.1.1

Now consider the behaviour of a statically indeterminate beam -a beam fixed at both ends A,B, and carrying a uniformly distributed load, which is gradually increased till collapse occurs. When the maximum moment at fixed ends reaches the ultimate moment capacity M_{ur} of the section (say at Stage-I) plastic hinges develop at fixed ends A and B. But this does not cause the beam to collapse. It simply converts a fixed beam into a hinged beam (i.e. a statically indeterminate beam into a statically determinate beam). Let the load at this stage be w_1 and $M_A = M_B = w_1 L^2/12 = M_{ur}$. The corresponding moment at midspan at this stage 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). A stage is reached when the bending moment at midspan reaches a value $M_{uc}=M_{ur}$ when a third hinge is developed at C. This causes division of beam into two segments AC and CB with rotations occurring at A, B and C. A mechanism is said to have formed which leads to collapse of the beam. This load causing collapse is known as ultimate load w_u. At this stage, $M_{uc}=M_{ur}=w_uL^2/8-M_{uA}$. But $M_{uA} = M_{ur}$. Therefore, $M_{ur} = w_uL^2/8 - M_{ur}$ or $M_{ur} = w_uL2/16$. It will be observed that at collapse since $M_{uA} = M_{uC} = M_{ur} = w_uL^2/16$, M_{uA} is also equal to $w_1L^2/12$. Therefore, $w_1L^2/12 = w_uL^2/16$ or $w_u = 1.33w_1$. Thus, there is an increase in load carrying capacity of a statically indeterminate beam by 33 % .

This analysis is known as Limit analysis which involves rearrangement called redistribution of moments along the span of the beam from Diagram I to Diagram II prior to collapse as shown in Fig.3.1.2. The redistribution of moment allows us to design the section for a single value equal to $w_uL^2/16$ only. On the other hand, if the limit analysis is not used and the redistribution of moments is not to be allowed, the load required to be carried by the beam is still w_u, the maximum moment at supports will be $w_uL^2/12$ according to elastic analysis and the beam will be required to be designed for $M_{ur} = w_uL^2/12$ at support instead of $w_uL^2/16$ while the moment at midspan will be just $w_uL^2/24$ instead of $w_uL^2/16$.

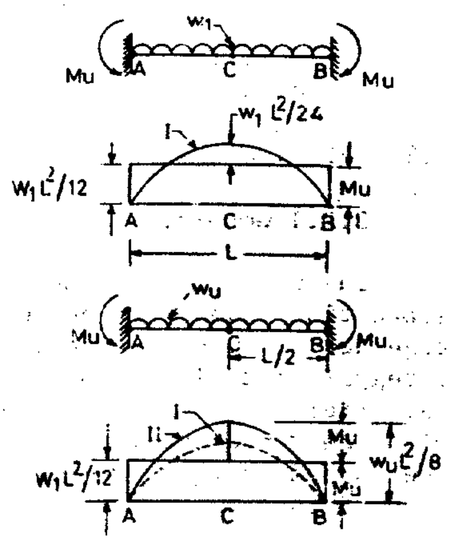


Fig.3.1.2 Redistribution of Moments Statically Interminate Beam.

This would obviously be uneconomical because of higher value of absolute maximum moment. Thus, limit analysis involves the process of redistribution of moments which allows the designer to take design moment at support less than that obtained by elastic analysis with corresponding increase in span moment to maintain equilibrium.

Redistribution of moments has the following advantages:

(1) Reduction in absolute maximum design moment and hence reduction in cross-section (depth in case of slabs). Design is, therefore, economical.

(2) Equalisation of span moment and support moment gives better distribution of moments and reinforcement detailing across the length of the beam.

(3) It reduces steel required at supports, and therefore, avoids congestion of steel at supports. This leads to better concreting at the beam-column junction and increases the reliability of the joint.

(4) Increase in the span moment helps to take advantage of the flange action of the beam at midspan.

(5) It not only reduces the moment at support but, many times, it also does not increase the design moment at midspan. This can be seen in 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.

(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 never be less than the maximum span moment for a simply supported beam. Thus, for beam subjected to uniformly distributed load,

$$M_{sup} + M_{span} => w_uL^2/8$$

In brief, a reduction in support moment by dM must be accompanied by corresponding increase (= dM/2) in the midspan moment.

(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
 - 15 % in Working stress design.
 - 10 % in case of multistoreyed frames in which design is governed by horizontal loads.

(c) Nowhere, the design moment M_{DU} shall be less than 0.7 times the elastic moment M_{EU} at ultimate state. Since M_{EU} = 1.5xthe Working moment M_{EW}, it means that $M_{DU} > 0.7 \times 1.5\ M_{EW}$ i.e. $M_{DU} > 1.05\ M_{EW}$, or nearly, $M_{DU} > M_{EW}$.

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

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

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

3.2 ELASTIC 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 large 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 localised 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.3.2.1.The frame consists of a continuous oneway slab S1 cast monolithically with secondary beams B1,B2,and main beams B3,B4,B5. 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 inderminate. 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 consideration of large number of unknown displacements and will require 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 such buildings does not demand the use of rigorous classical analysis. The approximate methods are more than adequate.

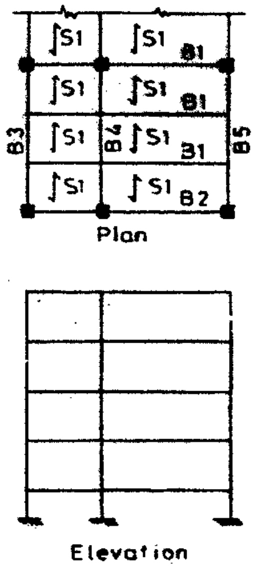


Fig.3.2.1
Typical Building Frame.

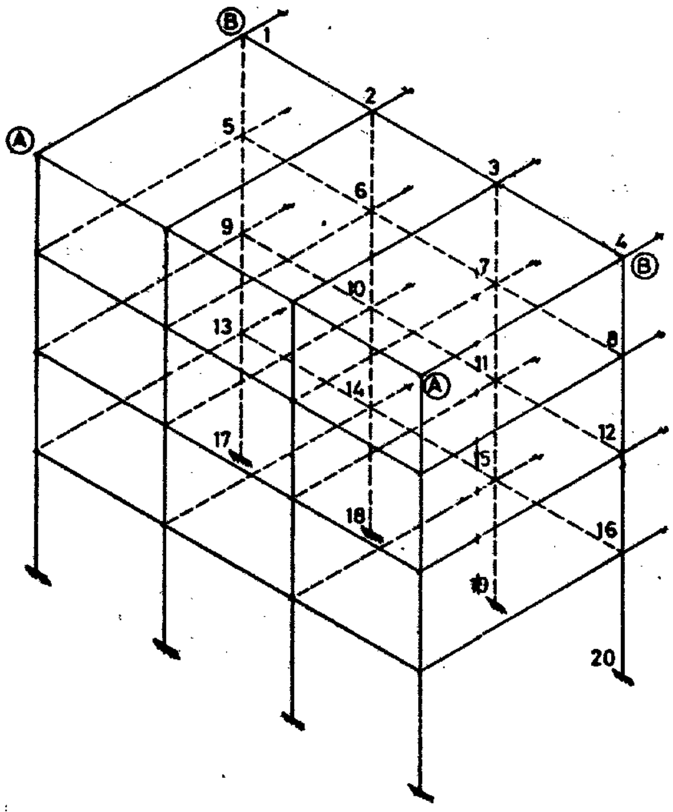
The approximate methods are based on the principle of dividing the structure into parts and analysing only the parts of interest, disregarding the effect of loads and resistances of members far away from the member of interest. The above simplification is based on the fact that a load on any member and its stiffness hardly affect 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 TableB-1).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.

The approximate method,therefore,adopts some standardised small portions of the whole frame, known as Substitute Frames or Subframes principally consisting of the members of interest and other adjacent members connected to it.The method of analysis is known is Substitute Frame Method. Since the scope of this book is restricted to buidings of upto 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

A building frame, in fact, is a three dimensional frame i.e. a space frame. See Fig.3.2.2(a). The analysis of a space frame is complex, laborious, and also



Elevation

Fig.3.2.2(a) Building Space Frame

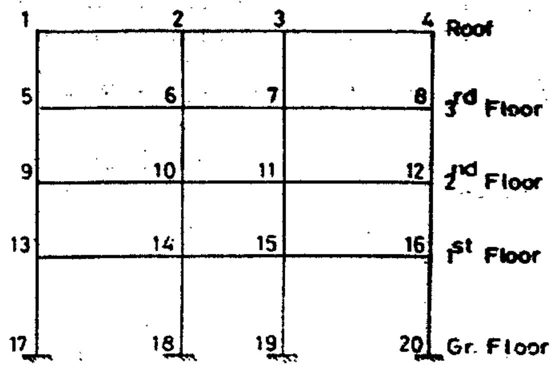


Fig.3.2.2(b) Frame Elevation

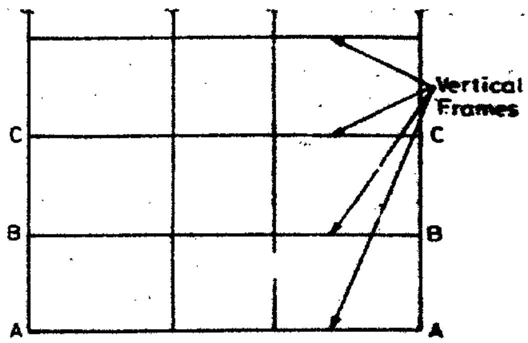


Fig.3.2.2(c) Plan

timeconsuming. 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.3.2.2b) is analysed for the loads (vertical or horizontal) in the plane of the frame and is

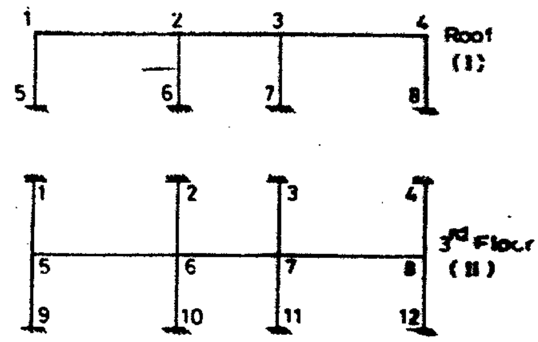


Fig.3.2.2(d) Substitute Floor Frame-
Top Storey & Middle Floor

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, C-C etc in Fig.3.2.2(c), 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.3.2.2(b) shows the front elevation of vertical frame marked B-B in Fig.3.2.2(c). This plane frame is further subdivided into substitute frames in different manners discussed below making second degree (or level) of approximation. As stated eariler, the description of substitute frames given is restricted to those used in analysis for vertical loads only.

(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 substitute frames for the top floor and intermediate floors are shown in Fig.3.2.2(d). The frame at any floor consists of beams at that floor level together with all connected columns in adjacent upper and lower storeys, assumed to to be fixed at their far ends. This frame can be analysed by any method for different loading cases shown in Fig.3.2.2(e) to determine 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.

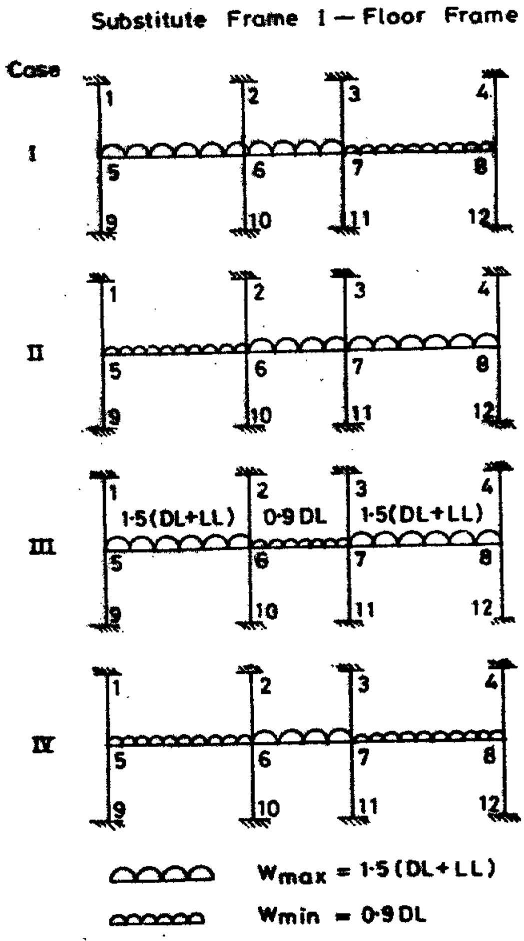
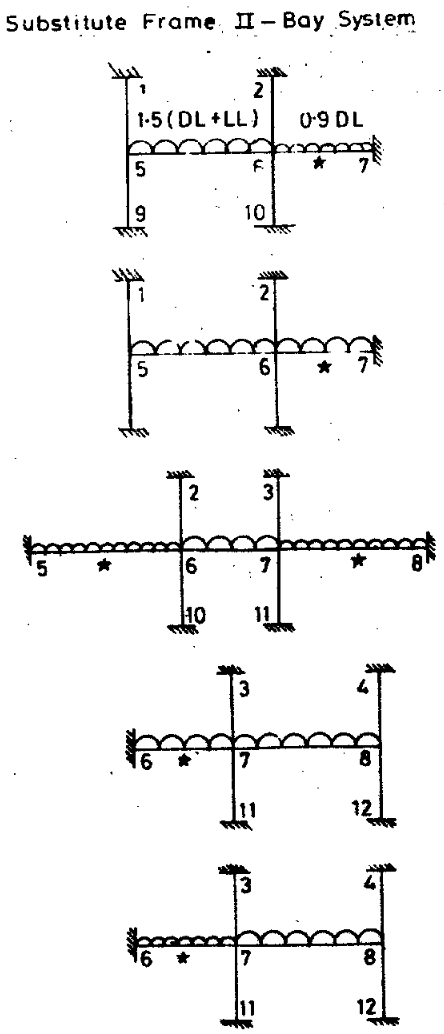


Fig.3.2.2(e) Substitute Floor Frame-I Loading Cases.

(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 as shown in Fig.3.2.2(f)(Page 31). The far ends of beam and columns are assumed as fixed. Such a frame is called Substitute Bay Frame. Since beams beyond the adjacent spans are not considered, their effect on the beam under consideration is accounted for by reducing the stiffness of the adjacent beams to half.[3.2]

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 unsymmetric frames and/or unsymmetric loading. However, the difference



★ Beam Stiffness to be reduced to half

Fig.3.2.2(f) Substitute Bay Frame-II
Intermediate Floor, Loading Cases.

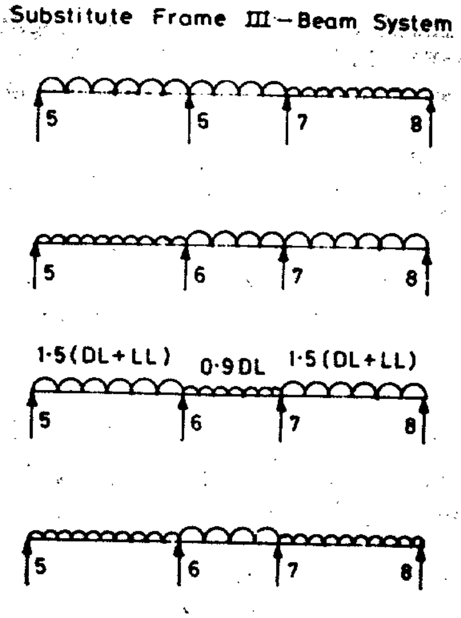


Fig.3.2.2(g) Substitute Frame-III
Beam System, Different Loading Cases.

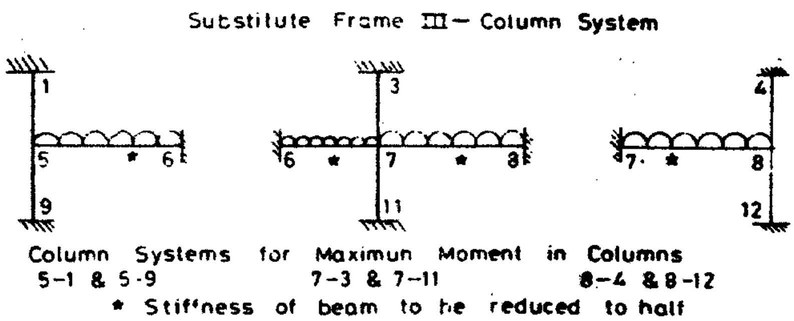


Fig.3.2.2(h) Substitute Frame-III Column Systems

is handily beyond 10%. Such frame is also analysed for different loading cases to get maximum forces in columns and beams as usual. It may be noted that analysing a full Substitute floor frame-I for all different loading cases is much involed than analysing number of substitute bay frames. However, authors are of the opinion that this degree of approximation, or any degree beyond this need not be used when analysis is to be done on a micro-computer.

(c) Substitute Frame – III : Beam and Column Systems

As a further fourth degree (level) of approximation, the floor frame – I in (a)above instead of subdividing into number of bay frames is broken or separated into a beam system made up of only a continuous beam at the floor, and column systems as shown in Fig.3.2.2(g). The continuous beam idealisation totally ignores the fixity offered by connected columns. The continuous beam is now analysed for all possible loading arrangements shown in Fig.3.2.2(g), and maximum shear, and moments are obtained in the beam assuming ends simply supported. Since, the rigidity offered by columns is ignored in this approach, the moments in beams work out to be very large, even, at times, to the extent of 30% to 50%, (especially in beams of inner bays) compared to actual moments. Thus, though the analysis is simplified, the design proves to be uneconomical. Eventhough, 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 two beams adjacent to the joint as shown in Fig.3.2.2(h).The far ends of beams and columns are assumed to be fixed and the stiffness of beams is reduced to half to compensate for the effect of bays beyond. The column sub-frames are analysed for such loading on beams so as to cause maximum column moments. For this, maximum load should be placed on longer beam on one side and minimum load on the shorter beam on the other side.See Fig.3.2.2(h).Code recommends not to use this method of analysis.[3.1]

If the results of this totally approximate method are required to be brought nearer to those of System-I in (a) above, 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 Project-2 later, 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 approximation is adopted, the analysis is 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 condition of the member at the far end.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

When two members are to be connected, no relative translatory 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.

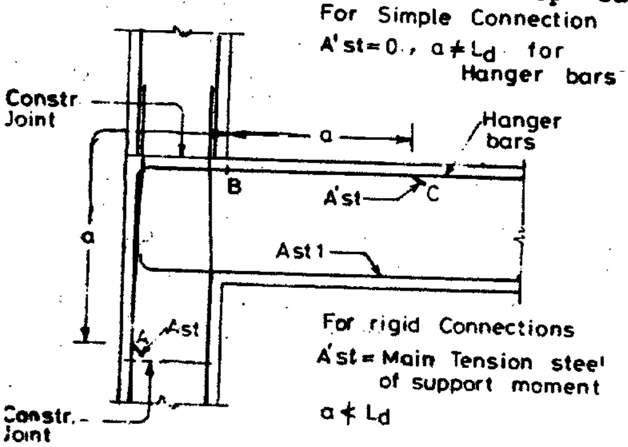
(i)There should be interconnecting tension steel between the two members on the tension face with area sufficient enough to effect the transfer of moment.
(ii)The interconnecting steel should be adequatly anchored in both the connected members either by requisite development length or by mechanical anchorage.

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.3.2.3(a). For the connection to be rigid, adequate area of interconnecting steel Ast', 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

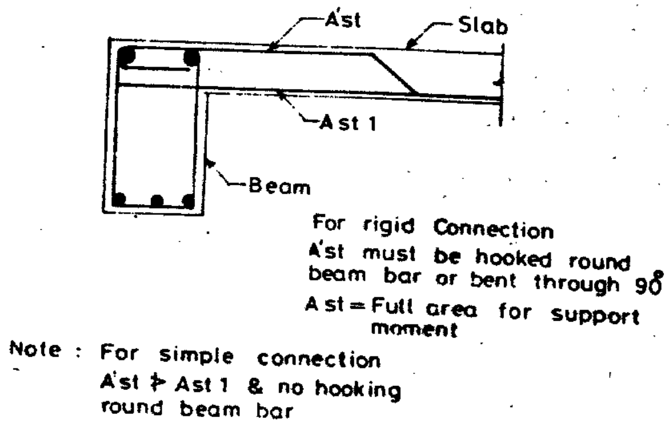
equal to anchorage length (which is equal to development length L_d). If it is found that this length is large, then instead of extending beam reinforcement into the column, the column bars should be bent and extended in the beam through a distance BC equal to the development length.

On the contrary, for a connection to be simple between the beam and column, no steel, other than anchor or hanger 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. See Note in Fig.3.2.3(a). 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 ealier.



BEAM-COLUMN CONNECTION

Fig.3.2.3(a) Rigid & Simple Connections



SLAB – BEAM CONNECTION

Fig.3.2.3(b) Beam-Slab Connection

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.3.2.3(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 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 along with the slab . The beam will act as a flanged beam only if there is a rigid connection between the beam and the slab.

3.2.4 Types of Support or End Condition

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 translatory 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 The same is true even for a beam rigidly connected to columns. If the supporting column also rotates along with the beam,the beam is said to have a rotation free (or hinged) end condition. 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 embeded 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 betweem 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 an 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 reaction .This resisting moment can be made available either by non-uniform pressure distribution at the base of footing in case of a concentric footing as shown in Fig.3.2.4(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.3.2.4(b).

If the bearing capacity of the soil is low, it is many times not possible to provide a fixed base i.e. moment resistant footing. In such 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.3.2.4(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 and such footings cannot be designed for axial loads only.

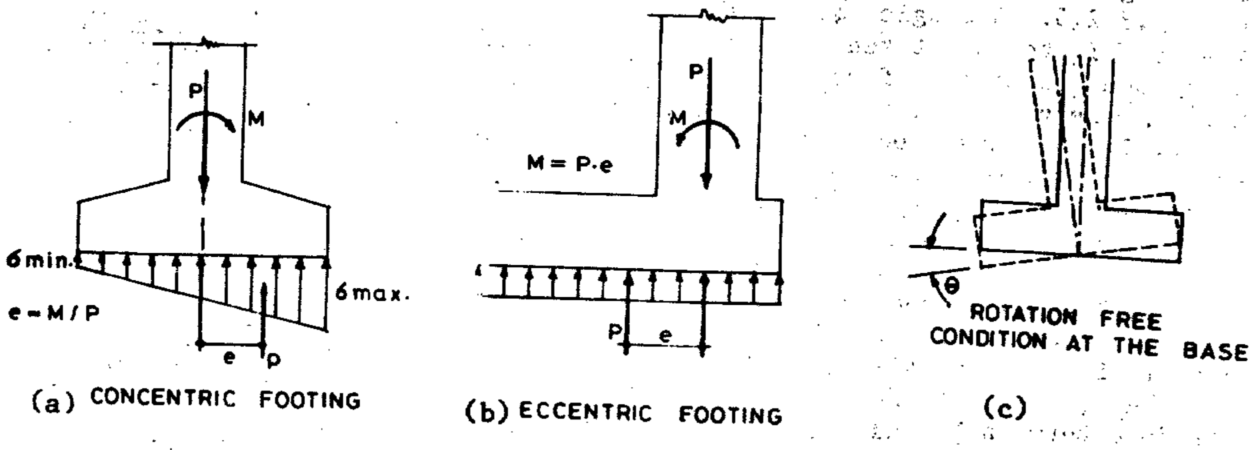


Fig.3.2.4 End Conditions for Column Footing

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.buiding 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.

3.2.5 Stiffness of Members

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) Gross-section : I of concrete gross cross-section (Igr) ignoring reinforcement.

(2) Transformed Gross Section : I of transformed gross concrete section including the reinforcement transformed on the basis of modular ratio.

(3) Transformed Cracked Section: I of concrete section in compression (Ir) including area of reinforcement transformed on the basis of 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 (Igr) ignoring reinforcement.

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

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

Flanged Section :

Depth of N.A. $x = (b_w D^2/2 + (b_f - b_w)D_f^2/2)/[b_w D + (b_f - b_w)D_f]$ - I

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

or $I_{gr} = k_f b_w D^3/12$ where

$k_f = 4[k_1 k^3 + (1-k)^3 - (k_1 - 1)(k - k_2)^3]$ - - III

$k_1 = b_f/b_w$, $k_2 = D_f/D$ and $k = x/D$.

Alternatively,the value of k_f may obtained from Fig.5.23(b) of Author's Handbook[3.4].

However,main difficulty in calculating I arises when the beam is continuous at both ends as in 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 of varying moment of inertia along the length. See Fig.3.2.5. A single equivalent or effective value of I to be taken for stiffness would depend upon the ratio of region (L_o) of positive moment to length L of the beam. However, since value of L_o depends upon the end conditions, the loading and the moments 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.[3.5]. As the calculations for I of flanged section are involved, the practice of taking I_{gr} of rectangular section is also prevalent.[3.6]. Taking I of beam on the basis of rectangular section only gives higher moments in columns. The values obtained from these practices differ appreciably from the actual sectional properties. Authors consider to take I_{gr} of beam=$2 \times b_w D^3/12$ i.e.2 times that of rectangular beam to be more acceptable[3.7]. In this case multiplying factor 2 corresponds to a flanged-section having $b_f/b_w = 6$ and $D_f/D = 0.2$.

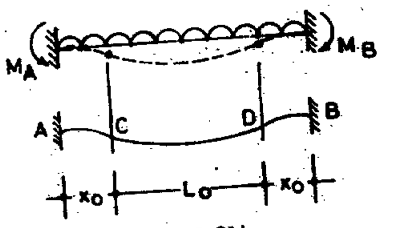


Fig.3.2.5 Varying M.I. in a Fixed/Continuous Beam.

In fact, taking I_{gr} throughout the length of the beam is also theoretically not correct. It can only be taken in the preliminary design of reinforcement.

A correct single equivalent value of effective moment of inertia I_{eff} can be obtained considering the effect of cracking throughout the span and by adopting the procedure given under Cl.B-2.1 of Appendix B of the Code using values of coefficient k_1 given in Table 21 of the Code. For details, reader may refer to Sect.8.4.3 of Author's Textbook[3.1]. However, since this calculation is very cumbersome and lengthy for hand calcultions with no special benefit in analysis and design, it may be used in computer aided analysis and design only.

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 bottom 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.

$$M_{col.a} = [K_{ca}/(K_{ca}+K_{cb}+K_b)].M_e \qquad M_{col.b} = [K_{cb}/(K_{ca}+K_{cb}+K_b)].M_e$$

$$M_{beam} = M_e\,[K_b/(K_{ca}+K_{cb}+K_b)]$$

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 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.3.2.6(a)]. The bending moments in the beam and column will both be nearly $wL^2/12$.[3.8] 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.

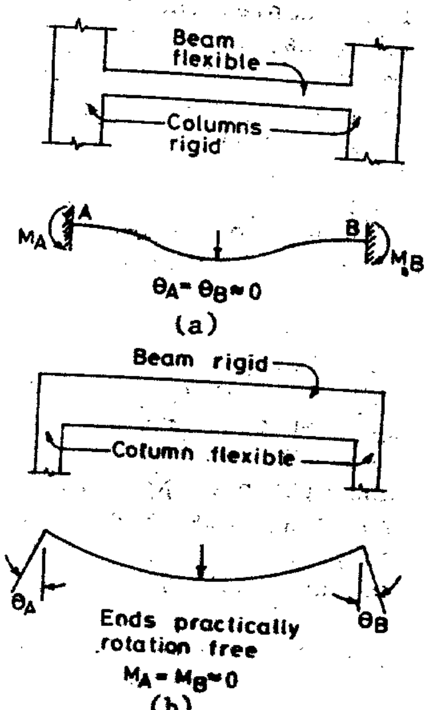


Fig.3.2.6 Effect of Stiffness on Moments in Beams and Columns.

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 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.[3.8] (practically entire value of fixed end moment Me at the joint gets released with no moment remaining in the beam).This situation gives rotation free simple support for the beam. (See Fig.3.2.6b).Thus, actual bending moment in the beam and column at the joint lies between 0 and $wL^2/12$ in general; the actal magnitude depends upon the degree of fixity which inter-alia 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, ofcourse, 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.

(a) 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 itself if the beam itself is simply supported at its end. 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 usually 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(a). 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 vice-versa). 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.

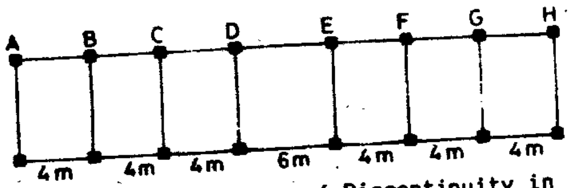


Fig.3.3(a) Introduction of Discontinuity in a long unequal span continuous beam

If a physically continuous beam cast monolithically is designed by assuming it as made up of a number of simply supported beams, it is quite likely that there may not be any steel or there may be very little insufficient steel over the top of intermediate support extending through anchorage length in both the spans. 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 midspan. The load is fully sustained if the midspan section is designed corresponding to moment for a simple support condition. However, this is a crude design practice which makes the beam heavy and misuses the principle of compensatory resistance. 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.

On the other hand, the analysis of a multispan continuous beam can be simplified by analysing and designing each beam (span) seperately considering approximate moments at continuous end based on redistribution of moments as explained in Sect. 3.3(b). In this approach, simplicity is achieved together with the desired economy, safety and a satisfactory structural performance.

Accordingly, in the illustration of Fig.3.3(a), the beam can be divided into three segments ABCD,DE and EFGH for the purpose of analysis only and each segment analysed independently assuming full 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.

(b) Approximations regarding Bending Moments in Beams and Slabs

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[3.4]. 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 distribted loads. These are given in Sect.3.5. 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.(Tables 2.6 and 2.7 in Author's Handbook[3.4]).

Such continuous beams/slabs can also be approximatly 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 a further 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 equal spans. Each one is analysed separately using the standard bending moment coefficients which are based on the ordinates of the bending moment envelope [See Fig.3.3(b)(i)] 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 midspan 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[3.9]

Moments at supports as well as midspan:

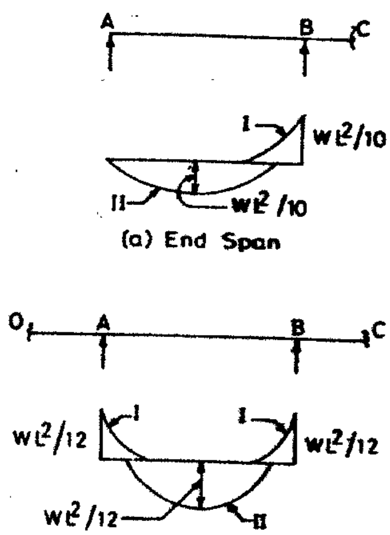
Uniformly Distributed Load (w) :

End spans	$wL^2/10$.
Intermediate spans	$wL^2/12$.

Central Point Load (W) :

End spans	$WL/6$
Intermediate spans	$WL/7$

The above approximation can be extended to continuous beams/slabs of unequal spans also provided longer span is not greater than twice the shortest span[3.9]. See Fig.3.3(b)(ii). If the short span occurs between two long spans, the hogging bending moment can occur at the centre of the short span. In such case, negative bending moments at intermediate supports adjacent to the short span shall be calculated as if all spans were equal to longest span and the moment of resistance should be provided throughout the short span for at least mean of the two negative moments and for positive bending moment as shown in Fig. 3.3(b)(ii). In other cases, the midspan moments may be obtained by using above coefficients. The support moments on either side of an intermediate support will be different either due to the difference in spans and/or due to difference in the load and support condition. The support section may be designed either for greater of the two moments



(b) Intermediate Span

Case I For Wmax on adjacent Spans

Case II For Wmax on AB & Wmin on OA & BC

Fig.3.3(b)(i) Approximate Moment Coefficients for an Isolated Span on safer side or for the average of the two. This practice is very common in case of floor made up of continuous two-way slab panels. It can be applied to continuous beams also. The former practice of providing steel for greater of the two moments is suitable when no bentup bars are used while the latter practice of providing steel for average of the two moments is suitable when alternate bars from adjacent spans are bent at supports. Though, this approximation is safe, it does not at all give economic design. The moments are on higher side by 10 % to 35 %. This approximation should definitely not be used for continuous beams rigidly connected to columns, because even the exact values of continuous beam analysis are by themselves on higher side as compared to analysis considering rigid frame action. The use of this approach and the difference of the results from exact solution of continuous beam are shown in example below. It will be evident from this example that it will be advisable not to use this approximation in case of frames in a R.C.structure which can be divided into standard substitute frames especially when desk-top computer is available in design office.

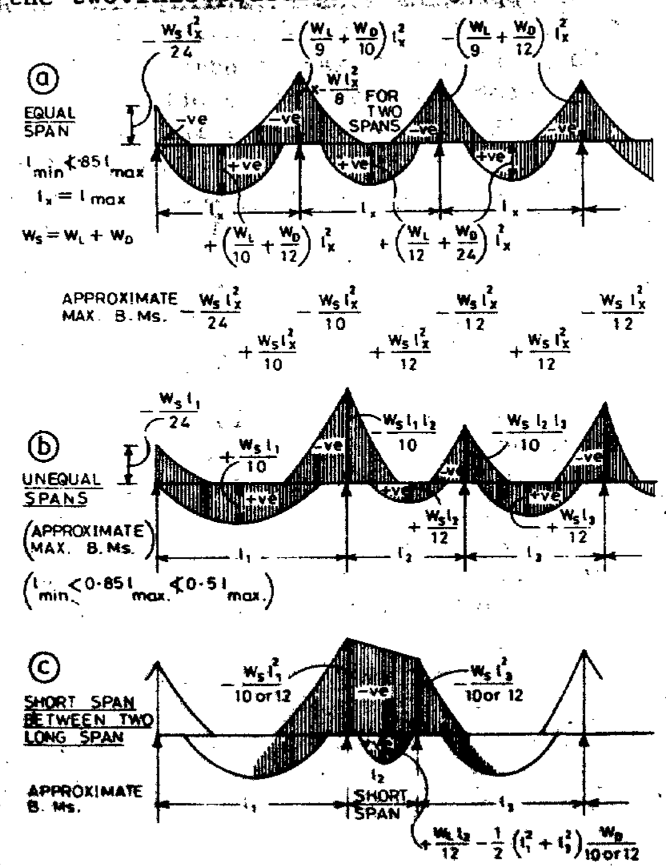
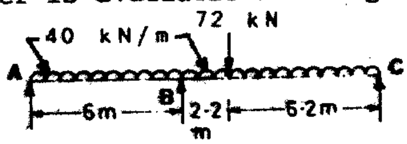


Fig.3.5(b)(ii)

Data : A two-span Continuous Beam
Span AB = 6.0 m, U.D.Load = 40 kN/m,
Span BC = 8.4 m, U.D.Load = 40 kN/m,
& Point load =72 kN at 2.2 m from B.



Design moments by above approximate method are as follows:

Span AB : $M = 40 \times 6^2/10 = 144$ kN.m at midspan as well as at support B.

Span BC : $M = 40 \times 8.4^2/10 + 72 \times 8.4/7 = 368.64$ kN.m at midspan, and

$M = 40 \times 8.4^2/10 + 72 \times 8.4/6 = 383.04$ kN.m at support B.

Design moment at support B,(taking greater of the two values)= 383.04 kN.m.

The comparison of these values with exact values is given below. (Results of Solution I of exact method have been obtained from Project II - Method III).

Type of analysis	Bending Moments in kN.m at Midspan-AB	Support	Midspan-BC
Exact Method:			
Sol.I - Without redistribution of moments	105.75	339.98	290.36
Sol.II- With 30% redistribution of moments	105.75	237.99	314.13
Approximate Method described above:			
Sol.I - taking greater of the two moments at B	144.00	383.04	383.04
Sol.II- taking average of the two moments at B	144.00	263.52	383.04
% Difference - Solution-I Approx.v/s Exact.	36%	12.7%	32%
- Solution-II Approx.v/s Exact.	36%	10.7%	22%

(c) *Assumptions regarding Rigidity of Support*

In the analysis of continuous beams or slabs, it is always assumed that the supports are rigid i.e. they do not deflect. If the support is a beam, it cannot be said to offer a rigid support unless it has the flexural rigidity far more than that of the supported slab or the secondary beam. Required rigidity of the supporting beam in relation to supported slab or beam would obviously depend on acceptable deviation of results (moments etc.) from the exact values. For one-way slab resting on beams, required value of relative rigidity $I_b/(Lt^3/12)$ should be at least 4 for 10% deviation of results where I_b and L are the moment of inertia and the length of supporting beam and t is thickness of slab See Fig.3.3(c)(i). For a secondary beam supported on main beam required $I_b = 10 I_s (L_1/L_2)^3$ for 10 % deviation where I_s is moment of inertia of secondary beam, L_1 is Length of main beam and L_2 is length of secondary beam. Taking the worst case of $L_1 = L_2$, required $I_b = 10 \times I_s$. See Fig.3.3(c)(ii). If this condition is not satisfied, the supporting beam will not act as a rigid support. In that case, the system as a whole will be required to be analysed as a stiffened or ribbed plate.

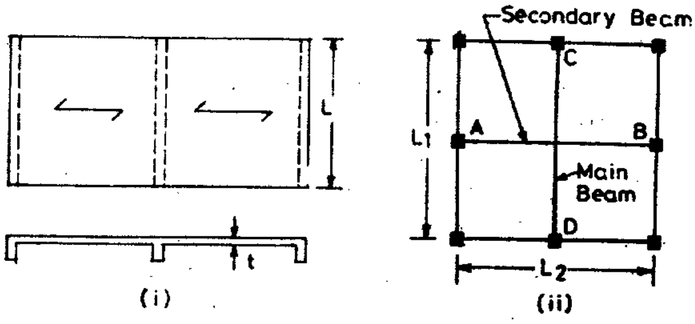


Fig.3.3(c) Assumptions regarding Rigidity of Support

(d) *Assumptions regarding Beam Section*

Another important assumption made in design of R.C.beams is about the type of the section (whether flanged or rectangular). A beam having a slab acting as flange on compression side and having minimum transverse reinforcement at top and which has stirrups running through total depth (including thickness of slab portion) will structurally act as a flanged beam. Since, design of a flanged beam is complicated and laborious as compared to design of rectangular beam, the design could be done assuming beam to be of rectangular section only when the design moment M_{DU} does not exceed $M_{ur.max}$ of the 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_{u1} 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 $[A_{st} = M_{DU}/(.87 f_y j_u d)]$ obtained on the basis of this assumption of rectangular section is always greater than that required for flanged section because the lever arm ($z_u = j_u d$) of the rectangular section is always less than that of a flanged section since x_u in a rectangular section is always greater than x_u for flanged section (for required total compression). See Fig.3.3(d). 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 flange portions depends upon the

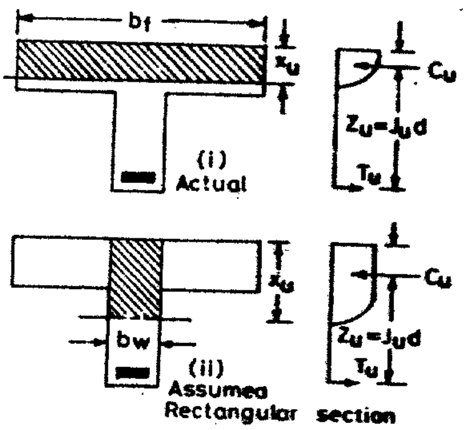


Fig.3.3(d) Assumption regarding Beam Section

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 20% for $x_u = D_f$. This simplification is very useful when the tables and charts for rectangular sections are readily available.

However, when the design moment M_{DU} exceeds $M_{ur.max}$ of the assumed rectangular section, this assumption should not be made i.e. the area of steel should not be worked out on the assumption of rectangular section because that would necessitate the rectangular section to be designed as a doubly reinforced.* In such case, the beam shall be designed as flanged 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:

Data : A beam continuous at both ends, Span = 4 m, Slab 110 mm thick, Section 230 mm x 380 mm, M15 - Fe 415, U.D.Load (a) $w_u = 24$ kN/m² (b) $w_u = 42.6$ kN/m².

Required : A_{st}.

Solution : Let d' = 35 mm, d = 345 mm.

$M_{ur.max} = R_{u.max} \times bd^2$. For M 15 - Fe 415, $R_{u.max} = 2.07$ N/mm².

$M_{ur.max} = 2.07 \times 230 \times 345^2 \times 10^{-6} = 56.67$ KN.m

(a) $M_{DU} = w_u L^2/12 = 24 \times 4^2/12 = 32.00$ kN.m.

I - <u>Calculation for design based on T-section:</u>

L_o = nearly 0.6L for beam continuous at both ends = .6x4000 = 2400 mm

$b_f = L_o/6 + b_w + 6D_f = 2400/6 + 230 + 6 \times 110 = 1290$ mm

For $x_u = D_f$, $M_{u1} = 0.36 f_{ck} . b_f . D_f (d - 0.42 D_f) = 0.36 \times 15 \times 1290 (345 - 0.42 \times 110) \times 10^{-6}$

$= 229$ kN.m > 32 kN.m $\therefore X_u < D_f$ and $b = b_f$.

Required $A_{st} = (0.5 \times 15/415)[1 - \sqrt{1 - (4.6 \times 32 \times 10^6/(15 \times 1290 \times 345^2)}] \times 1290 \times 345 = 262$ mm².

II- <u>Calculations based on assumption of Rectangular section:</u>

$M_{DU} < M_{ur.max}$ $\therefore$ Section shall be singly reinforced.

Required $A_{st} = (0.5 \times 15/415)[1 - \sqrt{1 - (4.6 \times 32 \times 10^6 /(15 \times 230 \times 345^2)}] \times 230 \times 345$
$= 286$ mm².

(b) $M_{DU} = 42.6 * 4^2/12 = 56.67$ kN.m

I - <u>Calculations for design based on T-section:</u>

Required Ast $= (0.5 \times 15/415)[1 - \sqrt{1 - (4.6 \times 56.67 \times 10^6 /(15 \times 1290 \times 345^2)}] \times 1290 \times 345$
$= 469$ mm².

---

* When design moment M_{DU} is greater than $M_{ur.max}$ of a rectangular section with $b = b_w$, the additional moment of resistance required ($M_{u2} = M_{DU} - M_{ur.max}$) is obtained by additional tension and compression steel in case of a rectangular section, while in case of a flanged section, the design moment M_{DU} is provided by M_{uw} of rectangular part of web, and the balance moment ($M_{DU} - M_{uw}$) is provided by M_{uf} i.e. by compression in concrete in the projecting flanges. Thus, M_{uf} in a flanged beam corresponds to M_{u2} in a doubly reinforced rectangular beam. In other words, the projecting flanges of a flanged beam take the place of compression steel in a doubly reinforced rectangular section. This is the reason why the flanged beams are capable of resisting moments greater than $M_{ur.max}$ of a rectangular beam especially when the neutral axis lies inside the web.

II- Calculation based on assumption of rectangular section:
$M_{DU} = M_{ur.max}$. Section is singly reinforced.

$$\text{Required } A_{st} = (0.5\times15/415)\left[1 - \sqrt{1 - (4.6\times56.67\times10^6/(15\times230\times345^2)}\right]\times230\times345$$
$$= 567 \text{ mm}^2.$$

Comments : It may be observed that A_{st} (= 286 mm²) obtained by assumption of rectangular section for $M_{DU} < M_{ur.max}$ is just about 9% higher than A_{st} (=262 mm²) given by T-section solution, while for $M_{DU} = M_{ur.max}$, the area of steel is about 21% higher than that given by T-section solution (= 469 mm²).

Important Note : All or any of the above assumptions shall not be used blindly without understanding. If demanded by the client, the designer will have to prove it by calculations based on explanations given above on the basis of theory.

3.4 CALCULATION OF LOADS ON STRUCTURAL ELEMENTS

Loads on beams and columns may be calculated using procedure given below:

3.4.1 Loads on Beams supporting One-way Slabs

Load on beam is the shear at the end of the slab. In case of an intermediate beam, it is the sum of slab shears on two sides of the beam. Thus, in Fig. 3.4.1(a),

Load on B1 = Shear of Slab S1 at left = V_{AB}
Load on B2 = Shear of Slab S1 to the right + Shear of Slab S2 to the left = $V_{BA} + V_{BC}$.

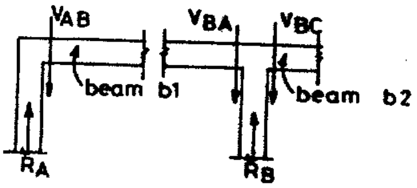


Fig.3.4.1(a) Slab End Shears as Loads on Beams

The shear V at the slab ends can be obtained by a general formula

$V = r.qL_x$ where
q = intensity of load on slab per unit area,
L_x = span of slab(or spacing of beams),
r = shear coefficient
= 0.45 at discontinuous end of a beam/slab simply supported at one end and continuous at the other (End condition No.2),
= 0.60 at continuous end of a beam/slab simply supported at one end and continuous at the other (End Condition No.2),
= 0.50 at both ends of a beam/slab either simply supported or continuous at both ends (End Condition Nos. 1 and 3) when live load is not greater than the dead load. The value shall be taken equal to 0.55 for live load when live load exceeds dead load See Fig. 3.4.1(b).

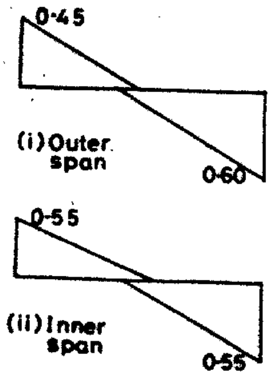


Fig.3.4.1(b) Shear Coefficients for an Isolated Span of a Continuous Beam

The above values of shear coefficients r are obtained from the shear force envelope.

Alternatively, load on beam may be calculated as follows.
Load on beam = Sum of shears from adjacent slabs(W) assuming slabs as simply supported x Continuity factor (C1) = C1.W

The values of continuity factors may be taken as follows.

	End beam	Penultimate beam	Intermediate beam
Continuity factor (C1)	0.9	1.2	1.0

Continuity factor accounts for change in shear due to continuity of slabs (reduction of load on end beam by 10 % and increase of load on intermediate beam by 10 %). Rigorously, if no redistribution of moments is done, the continuity factor for the penultimate beam varies from 25 % for a two-span slab to 5 % for a slab with more than 3 spans.

Besides load transferred from slab, the beam is subjected to UDL due to self weight of beam, wall load if any, and a point load due to the end reaction from secondary beams, if any.

For unsymmetrical loading, the loads on beams shall be calculated from first principles of structural mechanics.

When the slab is one-way reinforced, the loads transferred to beams is shown in Fig.3.4.1(c)(i). Areas of floor carried by different beams are as follows.

Beam	B1	B2	B3	B4
Area	A/2	A	Nil	Nil

where $A = L_x . L_y$

Load on beam = q x Area x C1.

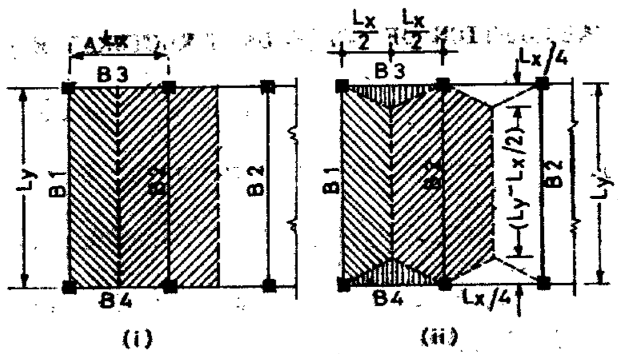


Fig.3.4.1(c) Loads on Beams supporting One-way Slabs

When reinforcement is provided across the beams B3 and B4 at top to prevent cracking, a part of the floor load is transferred to these short beams also. In case of an isotropic slab (i.e. slab having equal reinforcement in two perpendicular directions, as in case of a square slab simply supported on all sides), the load transferred to short beams is the load over a triangular area cosisting of a 45° isosceles triangle having base L_x and height of vertex = $L_x/2$. However, when the slab is designed as one-way and the main steel is only along the short span while there is just a minimum steel at top of short edge along the long span, the load transferred to the beam along the short edge may be approximately taken equal to the area of the triangle having height equal to $L_x/4$ instead of $L_x/2$, on the basis of yield line theory. (Refer to Fig.3.4.1(c)(ii). The load on long beam however is taken equal to $C1.q.L_x/2$ as obtained above.

The triangular load may be converted into an equivalent uniformly distributed load by using an equivalence factor explained in Sect.3.4.2 below, for loads on beams supporting two-way slabs. However, in the above case, since the height of triangle is $L_x/4$ instead of $L_x/2$ taken in two-way slabs, the equivalent UDL factor for B.M. C2 = (1/2)x(2/3) = (1/3), and that for shear C3 = (1/2) x (1/2) = (1/4). Thus, on short beam, the equivalent UD load for B.M. = C2 x (q x $L_x/2$), and the equivalent UDL for shear = C3 x (q x $L_x/2$), where q is the intensity of load on the slab per unit area, C2 = 1/3 and C3 = 1/4.

The loads on long beams shall be multiplied by continuity factors if the slab is simply supported on peripheral beams and continuous over inner beams.

In case of slab supported on secondary beams B2, it is always on the safer side to consider load on secondary beams to be that over rectangular area as shown Fig.3.4.1(d)(i) (page 45) instead of that over trapezoidal area shown in Fig.3.4.1(d)(ii). This is because the former assumption not only considers higher load on secondary beams but also transfers full load on main beam as central point load. On the other hand, the trapezoidal distribution transfers lesser load as central point load on main beam and the balance load also as triangular load over the beam in the form of two triangles with their vertices at quarter span points instead of at midspan. Such double triangular load can be replaced by equivalent uniformly distributed load of intensity equal to half the ordinate of trianglar load.

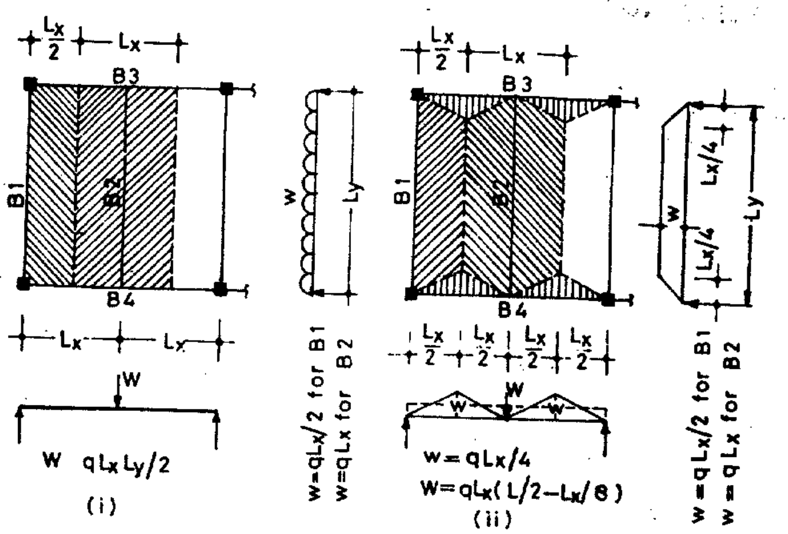


Fig.3.4.1(d) Loads on Secondary Beams supporting One-way Slabs

3.4.2 Loads on Beams supporting Two-way Slabs

In case of beams supporting two-way slab, the load distribution is trapezoidal on long beams and triangular on short beams with base angle of 45° as shown in Fig.3.4.2. The ordinate of trapezoidal and triangular load=$qL_x/2$

The triangular and the trapezoidal loads are converted into equivalent uniformly distributed loads by using the equivalence factors. The equivalent UDL factor (C2) for B.M. and that for shear (C3) are obtained as under.

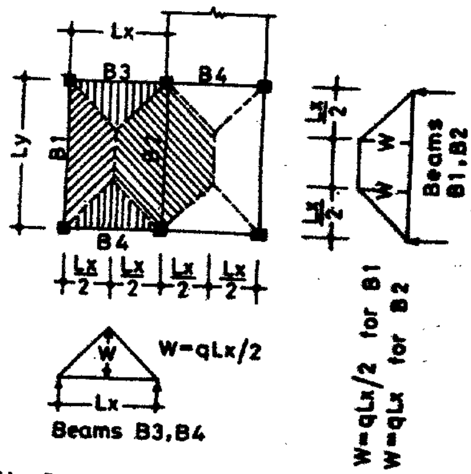


Fig.3.4.2 Loads on Beams supporting Two-way Slabs.

Short Beam :

The load on short beam is triangular having length L_x and height $L_x/2$. Therefore, to obtain equivalent UD load for B.M., we equate the maximum bending moment at centre equal to $w_{eq \cdot b}\ L_x^2/8$ with that due to triangular load $= (1/2)q\ (L_x/2)\ (L_x/2)\ (L_x/2-L_x/6) = qL_x^3/24$. This gives $w_{eq.b} = (qL_x^3/24)/(L_x^2/8) = (2/3)(q\ L_x/2) = C2(q\ L_x/2)$. Thus, $C2 = 2/3$. and $q\ L_x/2$ is the central ordinate of the triangular load. $\therefore w_{eqb} = q\ L_x/3$.

To get the equivalent UD load for shear, equate the total load to $w_{eqs} L_x$.

$\therefore w_{eqs}\ L_x = (1/2)(L_x/2)(q\ L_x/2)$ giving $w_{eqs} = (1/2)(q\ L_x/2) = C3\ (q\ L_x/2)$ and $C3 = 1/2$. Thus, $w_{eqs} = q\ L_x/4$.

Long Beam :

Proceeding on the same lines as for short beam above, by equating maximum values due to actual load with that due to equivalent UD loads, we obtain

for B.M. $w_{eq\,b} = [1-1/(3\beta^2)]\ (q\ L_x/2) = C2\ (q\ L_x/2)$ giving $C2 = [1-1/(3\beta^2)]$

for shear $w_{eq \cdot s} = [1-1/(2\beta)]\ (q\ L_x/2) = C3\ (q\ L_x/2)$ giving $C3 = [1-1/(2\beta)]$,

where $\beta = L_y/L_x$.

The loads are multiplied by continuity factors if the slab is simply supported along peripheral beams and continuous over inner beams. For further details, refer to Table 2.12 of Author's Handbook[3.4]

In case of slabs supported on secondary beams as shown in Fig.3.4.1(d)(ii), the equivalent uniformly distributed load on main beam replacing two triangular loads is $qL_x/4$ instead of $qL_x/3$ (because vertices of triangle are not at midspan but at quarter span points).

3.4.3 Loads on Columns

(a)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 algebric sum of the shears V at the end of all beams meeting at the column, the axial load P_a coming from the upper column and its self weight P_{self}.

Thus, at any floor level
$P = V1+V2+V3+V4+P_a+P_{self}$
See Fig.3.4.3(a).

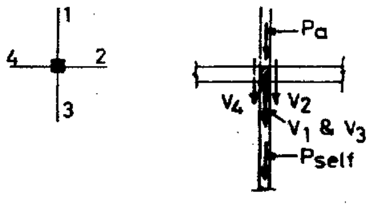


Fig.3.4.3(a) Loads on Column from Beam Shears

(b)When the load on the column is required prior to design of beams, it is sufficiently accurate to consider the load over the area of the floor supported by the column. The loads transferred from slab to columns C1, C2, C3, and C4 are shown by encircled figures 1, 2, 3, 4 in Fig.3.4.3(b). To this floor load, the wall loads between the floor to floor height and self weight (based on assumed cross-section) are added to obtain load on column at each floor level.* This load shall further be multiplied by continuity factors given in Sect.3.4.1 above. Since the load on column comes from both directions the continuity factors shall be as follows.

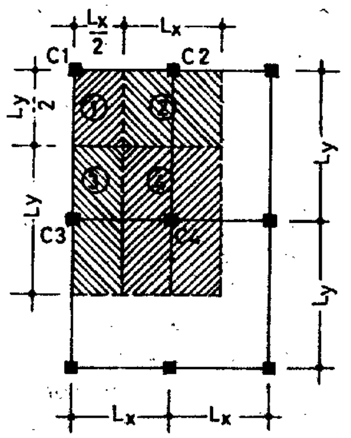


Fig.3.4.3(b) Loads on Column by Distribution of Floor Areas

Corner columns Cl = 0.8, Side columns Cl = 1.1 and Internal columns Cl = 1.2.
(For category of a column, refer to Sect.5.4.3.)

The design load may be obtained by adding certain allowance depending upon the position of the column in plan to account for the effect of moments due to partial fixity between the beams and column. This allowance is normally decided by individual designer on the basis of his past experience and judgement. The range of allowance is given in Fig.3.4.3(c) for ready reference.

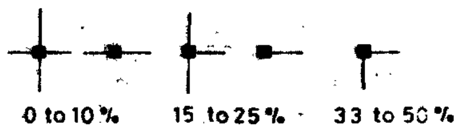


Fig.3.4.3(c) Allowances for bending due to effect of partial/full fixity

Remarks : The above procedure of calculation of column loads on the basis of column load area may not work in plans having number of secondary beams irregularly supported on main beams, because load on secondary beams lying within the column load area is not fully transferred to that column but to the columns supporting main beams on which this secondary beam rests. In such cases, first, only loads are obtained approximately on various beams, and beam shears are calculated using appropriate continuity factors according to the procedure explained in Sections 3.4.1 and 3.4.2. The column loads are then obtained by addition of beam shears. This has been illustrated in Project-III.

3.5 BENDING MOMENT AND SHEAR FORCE COEFFICIENTS FOR STANDARD CASES

The bending moment and shear force coefficients for some typical beams and loading cases are presented in Appendix - B. The reader is advised to refer to Section 2 of Author's Handbook for exhaustive information.

References:

3.1 Karve,S.R. and Shah,V.L. "Limit State Theory and Design in Reinf. Concrete", Structures Publishers, Pune, 4th Reprint - 1998 Pune, Chap.3, Ex.3.6.2,4,5.

3.2 Kong,F.K and Evans,R.H. "Reinforced and Prestressed Concrete",Thomson Nelson and Sons Ltd, Surrey,1980. Chap.11, Sect.11.4(a), p.345.

3.3 'Explanatory Handbook on Indian Standard Code of Practice for Plain and Reinforced Concrete', Bureau of Indian Standards,Special Publication SP:24 (S & T) 1983. Page 50.

3.4 Karve,S.R. "Handbook of Reinforced Concrete Design", Structures Publishers, Pune, 5th Reprint - 1997

3.5 Reynolds,C.E. and Steedman,J.C. "Examples of Design of Buildings to C.P.110 and Allied Codes", View Point Publication, C.and C.A., London, 1978.

3.6 Wilby,C.B."Structural Concrete",Butterworth and Co. London, 1983. p.169.

3.7 Wang,C.K. and Salman,C.G. "Reinforced Concrete Design",Harper and Row Publishers, New York, 1979. Third Edition, Sect.10.3, p.294.

3.8 Morgan,W. "Elementary Reinforced Concrete Design",Edward Arnold Ltd. London. Second Edition. Chap.14,p.283.

3.9 Reynolds,C.E. "Basic Reinforced Concrete Design", Concrete Publication Ltd., London.1966. Chap.8 .

Page for Personal Notes

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 deatil. 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:456-1978. For detailed study of the Limit State Theory, the reader may refer the Authors' textbook.

4.1 FLEXURE : (THEORY OF BEAMS AND SLABS)

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, $\varepsilon \propto x$. Graphically, the strain diagram across the section is triangular.
(2) Limit state of collapse in bending is said to have reached when the maximum strain in concrete ε_{max} at the outermost 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 shown in Fig.4.1.1.
(4) Concrete does not carry any tension. The tension is carried by steel only.
(5) Perfect bond exists between concrete and steel right upto 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 steel at ultimate state (i.e. at collapse) shall not be less than $0.002+f_y/(1.15E_s)$ i.e. the reinforcement must yield prior to crushing of concrete at collapse. See Fig.4.1.2(b). It may be observed from Fig.2.5 that yield strain in steel $\varepsilon_{sy} = 0.002+f_y/(1.15E_s)$.

Stress-Block Parameters :

The parameters giving maximum stress in concrete, average stress in concrete, and position of total compression, as given by the ~~I.~~S.Code are as follows. See Fig.4.1.2a

Parameter for maximum stress ($f_{c.max}$) in concrete
$k_3 = 0.446$ say 0.45,

Parameter for average stress ($f_{c.av}$) in concrete
$k_1 = 0.361$ say 0.36,

Parameter for position of total compression in concrete (i.e. the distance x of c.g of stress block from highly compressed edge) $k_2 = 0.416$ say 0.42.

Thus, $f_{c.max} = k_3\, f_{ck} = 0.45\, f_{ck}$,
$f_{c.av} = k_1\, f_{ck} = 0.36\, f_{ck}$, and
$x = k_2\, x_u = 0.42\, x_u$.

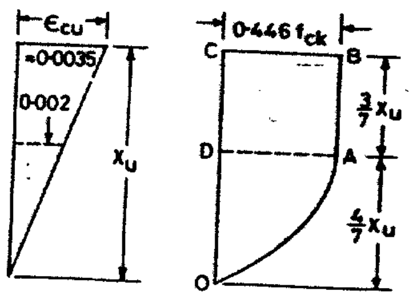


Fig.4.1.1 Stress-Block Parameters

4.1.2 Properties of Singly Reinforced Rectangular Section :

(a) Depth of Neutral Axis

The depth of neutral axis below the highly compressed edge (or the depth of concrete in compression, or the depth of stress-block) = x_u is obtained by equating the total longitudinal internal compression in concrete with the total internal tension in steel to satisfy the condition of equilibrium , $C_u = T_u$

$0.36\ f_{ck}\ b.x_u = 0.87\ f_y,$

$x_u = (.87 f_y\ A_{st})/(.36 f_{ck}\ b)$ .....(4.1-1),or

$k_u = 0.87 f_y\ p_t/(0.36 f_{ck})$ .....(4.1-1a), where neutral axis depth factor $k_u = x_u/d$ and steel factor $p_t = A_{st}/bd$.

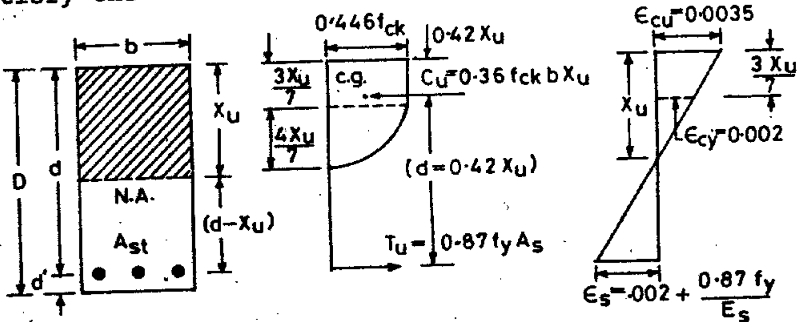


Fig. 4.1.2a Singly Reinforced Rectangular Section

(b) Lever Arm (z_u, j_u) :

Lever arm $z_u = d - .42\ x_u$.(4.1-2)

Lever arm factor $j_u = z_u/d$

$j_u = 1 - .42\ k_u$ ...(4.1-2a)

(c) Moment of Resistance (M_{ur} , R_u)

The moment of resistance is the moment of a tension - compression couple, and is equal to ultimate design moment M_u i.e. $M_{ur} = M_u$.

It can be expressed either as (C_u x lever arm) or (T_u x lever arm).

(i) Taking moment of total compression C_u about C.G. of steel we get,

$M_{ur} = C_u \times z_u = 0.36\ f_{ck} b\ x_u (d - 0.42\ x_u)$ ....(4.1-3)

Expressing moment of resistance factor

$R_u = M_u/bd^2 = 0.36\ f_{ck}\ k_u\ (1 - 0.42\ k_u)$ ....(4.1-3a)

(ii) Taking moment of total tension T_u about C.G. of compression

$M_{ur} = T_u . z_u = 0.87 f_y\ A_{st}\ (d - 0.42 x_u)$ ....(4.1-4a)

or $M_{ur} = 0.87 f_y\ A_{st}\ d\ [1 - (f_y/f_{ck})(A_{st}/bd)]$ ....(4.1-4b)

This equation is given by I.S. Code in Appendix E in Cl.E-1.1.(b).

(d) Area of Steel (A_{st}, p_t) :

The relation between A_{st} and M_u is obtained by solution of Eq.4.1.4 which is qudratic in A_{st} and is given by :

$A_{st} = (0.5 f_{ck}/f_y)[1 - \sqrt{1 - 4.6\ M_u/(f_{ck}.bd^2)}].bd$ .....(4.1-5)

or $p_t = (0.5 f_{ck}/f_y)[1 - \sqrt{1 - 4.6\ M_u/(f_{ck}.bd^2)}]$ .....(4.1-5a)

Note: Design engineers who are not familiar with the Limit State Method but are well versed with the Working Stress Method,the following lever arm approach is given for calculation of Ast.

$A_{st} = M_u/(0.87 f_y j_u d)$ .....(4.1-6)

The value of j_u may be obtained from the following Table.

Ratio $M_u/M_{ur.max}$ or $R_u/R_{u.max}$	1.0	0.90	0.80	0.70	0.60	0.55	0.50	0.40	0.30
Lever arm factor j_u	0.8	0.825	0.85	0.87	0.89	0.90	0.91	0.93	0.95

4.1.3 Design Constants for Balanced or Critical Section :

A balanced or critical section is that section in which maximum strains in both concrete and steel reach their ultimate values simultaneously. The strain distribution diagram for a such a section is shown in Fig.4.1.2b(Page 51). Since the maximum strain in steel $\varepsilon_{s.max}$ is not allowed to be less than ε_{sy} according to assumption No.7 given in Sect.4.1.1, the value of x_u for the critical section is the maximum allowable value $x_{u.max}$. It can be obtained from the strain diagram and is given by :

$x_{u.max} = 0.0035/[0.0055+f_y/(1.15\ E_s)]$.For $E_s = 2X10^5\ N/mm^2$

$x_{u.max} = [700/(1100 + 0.87\ f_y)]\ d$ .....(4.1-7)

or $k_{u.max} = x_{u.max}/d = 700/(1100 + 0.87\ f_y)$ .....(4.1-7a)

Once the value of $k_{u.max}$ is known, other constants can be obtained from Eq.(4.1-3) and Eq.(4.1-1a).

$R_{u.max} = 0.36 f_{ck}\ k_{u.max}\ (1-0.42\ k_{u.max})$ ..(4.1-8)

$M_{ur.max} = R_{u.max}\ bd^2$ ..(4.1-9)

$P_{t.max} = 0.36 f_{ck}\ k_{u.max}/(0.87 f_y)$ ..(4.1-10a)

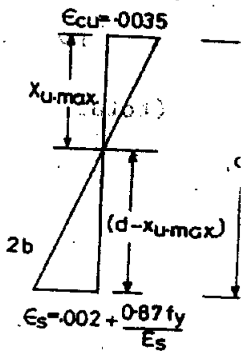


Fig. 4.1.2b Strain Distribution Diagram

If redistribution of moments is allowed by δM %,

$k_u = k_{u.limit} <= 0.6 - \delta M/100 < k_{u.max}$

The values of the design constants $R_{u.limit}$ and $p_{t.limit}$ on redistribution are obtained using Eq.4.1-8 and Eq.4.1-10a by replacing $k_{u.max}$ by $k_{u.limit}$.

Table 4.1-1 : Design Constants for Balanced or Critical Section

Concrete grade :		M15			M20		
Steel grade :		Fe250	Fe415	Fe500	Fe250	Fe415	Fe500
$K_{u.max}$		0.53	0.48	0.46	0.53	0.48	0.46
$R_{u.max}$	N/mm²	2.23	2.07	2.00	2.97	2.76	2.66
$P_{t.max}$	%	1.32	0.72	0.57	1.76	0.96	0.76

4.1.4 Properties of Doubly Reinforced Rectangular Section :

(a) Depth of Neutral Axis (x_u) (See Fig.4.1.3)

Equating total compression (in concrete and compression steel) with total tension in tension steel,

$0.36 f_{ck}\ bx_u + (f_{sc}-f_{cc})A_{sc} = 0.87 f_y A_{st}$ ..(4.1-11)

For simplification f_{cc} may be ignored or may be taken = 0.45 f_{ck}.

In above equation, f_{sc} is stress in compression steel corresponding to ϵ_{sc}. It can be obtained from the strain diagram, and is given by :

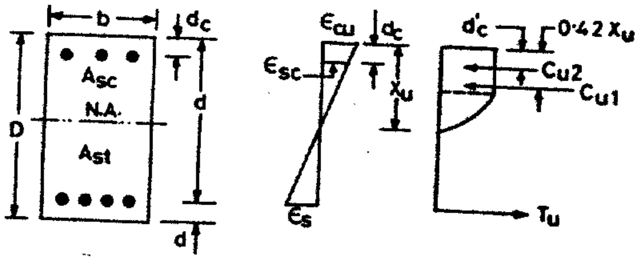


Fig.4.1.3 Doubly Reinforced Rectangular Sections

$\epsilon_{sc} = 0.0035(1 - d'_c/x_u)$

$= 0.0035[1 - (d'_c/d)/k_u]$ .....(4.1-12)

For mild steel (Fe250), $f_{sc} = \epsilon_{sc}\ E_s =< 0.87 f_y$

Sustituting $E_s = 2 \times 10^5$ N/mm² and ϵ_{sc} from Eq.4.1.12, we get,

$f_{sc} = 700\ [1 - (d'_c/d)/k_u] <= 0.87\ f_y$ .....(4.1-13)

For HYSD bars, the direct relation between f_{sc} and ϵ_{sc} cannot be obtained because stress-strain variation is not linear.Hence,the values of f_{sc} are obtained from stress-strain diagram of HYSD bars corresponding to values of ϵ_{sc} for different ratios d'_c/d and are given in Table.4.1-2 for ready reference.

Table 4.1-2 : Values of f_{sc} (in N/mm²) for Different Ratios of d'_c/d for HYSD Bars :

	Fe 415				Fe 500			
k_u d'_c/d	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
0.40	354	348	334	314	422	407	386	349
0.46	–	–	–	–	424	412	395	371
0.48	355	352	342	329	–	–	–	–

(b) Moment of Resistance (M_{ur}) :

The ultimate moment of resistance is obtained by taking moments of C_{u1} (total comression in concrete) and C_{u2} (total compression in compression steel) about cetroid of tension steel.

$$M_{ur} = M_{u1} + M_{u2}$$
$$= .36 f_{ck} b x_u (d - 0.42 x_u) + (f_{sc} - f_{cc}) A_{sc} (d - d'_c) \quad .....(4.1\text{-}14)$$

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

In design problems, we are required to determine the area of steel for a given section to resist a given design moment M_u. The section is normally designed as doubly reinforced when the design moment to be resisted is greater than the moment of resistance of a balanced section. Thus in design, the section is kept balanced to make full utilization of the resistance of concrete.

$\therefore$ $M_u = M_{ur} = M_{u1} + M_{u2}$ wherein $M_{u1} = M_{ur.max}$ of a singly reinforced balanced section. Now $M_{ur.max} = R_{u.max} \cdot bd^2$. The values of $R_{u.max}$ can be obtained from Table.4.1.1.Balance moment remains to be resisted is $M_{u2} = M_u - M_{ur.max}$. Area of tension steel and compression steel for resisting M_{u1} and M_{u2} can be obtained as follows.

(i) Tension Steel :

Area of tension steel $A_{st1} = A_{st.max}$ corresponding to $M_{u1} = M_{ur.max}$ is obtained from Eq.4.1.4(a) as, ...(4.1-15a)

$$A_{st1} = M_{u1} / [0.87 f_y (d - 0.42\, x_{u.max})]$$

where $x_{u.max} = k_{u.max} \cdot d$. Alternatively,

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

The area of tension steel A_{st2} is obtained by equating M_{u2} with the moment of T_{u2} ($= 0.87 f_y A_{st2}$) about the compression steel. (See Fig.4.1.3)

$$A_{st2} = M_{u2} / [0.87 f_y (d - d'_c)] \quad .....(4.1\text{-}15b)$$

Total area of tension steel $A_{st} = A_{st1} + A_{st2}$ .....(4.1-15c)

(ii) Compression Steel :

The area of compression steel(A_{sc}) is obtained by equating M_{u2} with the moment of C_{u2} [$= (f_{sc} - f_{cc}) A_{sc}$] about tension steel. .....(4.1-15d)

$$A_{sc} = M_{u2} / [(f_{sc} - f_{cc})(d - d'_c)]$$

or equating $C_{u2} = T_{u2}$ .....(4.1-15e)

$$A_{sc} = 0.87 f_y A_{st2}) / (f_{sc} - f_{cc})$$

where f_{sc} is the stress in compression steel.

For simplification f_{cc} may be ignored or may be taken equal to $0.45 f_{ck}$.

The stress in compression steel is obtained from Eq.(4.1-13).Alternatively, value of $[(0.87 f_y / (f_{sc} - f_{cc})]$ required in Eq.(4.1-15e) above are given in Table.4.1-3 below for different ratios d'_c/d for ready reference.

Table 4.1-3 : Values of $(0.87 f_y)/(f_{sc} - f_{cc})$ 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
M15	1.037	1.046	1.077	1.120	1.042	1.073	1.120	1.194
M20	1.043	1.053	1.084	1.128	1.048	1.079	1.126	1.201
Approximate % increase over A_{st2}	4 %	5%	8%	12%	5%	8%	12%	20%
	In general - 10 %				In general - 15 %			

4.1.5 Properties of Flanged Section

(a) Effective Width of Flange (b_f):

(i)For T-beams : $b_f = L_o/6 + 6D_f + b_w$ .....(4.1-16a)

(ii)For L-beams : $b_f = L_o/12 + 3D_f + b_w$ .....(4.1-16b)

where, L_o = Length of beam between points of zero bending moment.
b_w = breadth of web, D_f = Depth of flange (= slab thickness)
$b_f <= b_w$+half the sum of clear distances to adjacent beams on either side.

Values of L_o may be taken as under : Cantilever - 2L;
Beam simply supported at both ends - L; Beams fixed at both ends - 0.6L;
Beam simply supported at one end and fixed at the other end - 0.75L;
Beam simply supported at one end and continuous at the other - 0.75L to L depending upon moment at continuous end.
Beam continuous at both ends - 0.6L to L depending on moments at both ends. (0.7L according to the Code)

Since small variation in b_f does not materially change the strength of the section especially when $x_u<D_f$(which is the normal case). Hence value of L_o =0.7L is normally taken for calculating b_f of continuous beams.

(iii)For isolated beams, the effective width of flange is given by

$b_f = L_o/(L_o/b+4) + b_w$ for T beams .....(4.1-16c)

$b_f = 0.5L_o/(L_o/b+4) + b_w$ for L beams .....(4.1-16d)

In any case $b_f <=$ actual flange width.

(b) Depth of Neutral Axis (X_u):

(i) Neutral axis lying inside the flange (i.e. $x_u <= D_f$):

In this case, beam acts as a rectagular beam with b = b_f.Hence putting $b=b_f$ in Eq.(4.1.3) and Eq.(4.1.5), we get

$M_u = 0.36f_{ck}\ b_f\ x_u\ (d - 0.42x_u)$ .....(4.1-17a)

The solution of the quadratic equation gives the value of A_{st}.

$A_{st} = (0.5\ f_{ck}/f_y)\ [1-\sqrt{1-4.6\ M_u\ /(f_{ck}b_f d^2)}]b_f d$ .....(4.1-17b)

(ii) Neutral axis lying inside the web (i.e. $x_u>D_f$):

In this case, there are two possibilities.

I - 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 situation, the depth of neutral axis may be obtained from the relation,

$0.36f_{ck}\ b_w\ x_u + 0.45f_{ck}\ (b_f - b_w)\ y_f = 0.87f_y\ A_{st}$ .....(4.1-18a)

where $y_f = 0.15x_u + 0.65\ D_f$ not greater than D_f. .....(4.1-18b)

II - Depth of flange is less 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 neutral axis is obtained using Eq.(4.1.18) only except that y_f will be replaced by D_f.

Moment of Resistance (M_{ur}): :

$M_{ur} = M_{uw} + M_{uf}$
$= 0.36f_{ck}b_w x_u(d-0.42x_u) + 0.45f_{ck}(b_f-b_w)y_f(d-y_f/2)$....(4.1-19)

For $x_u >= 7D_f/3$, 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 neutal axis would lie 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

$M_{ur1} = 0.36f_{ck}\ b_f\ D_f\ (d - 0.42\ D_f)$ .....(4.1-20)

If $M_u <= M_{ur1}$,then $x_u <= D_f$ and A_{st} may be obtained by using Eq.(4.1-17b)

If $M_u > M_{ur1}$,then $x_u > D_f$ and then x_u will first be obtained by using Eq.(4.1-19 and 18b). A_{st} can then be obtained by using Eq.(4.1-18a).

Comments : In most of the cases, for normal loads and beam spanning, and when the depth of the beam is taken between L/12 to L/15, the neutral axis lies inside the flange only.

4.2 SHEAR AND TORSION

Part A : Shear

4.2.1 Critical Section for Shear :

Critical section for shear is at a distance 'd' from the face of support offering compressive reaction and at the face of support offering tensile reaction.

4.2.2 Design Shear Force (V_{uD}) :

The maximum shear force at the end of a beam carrying uniformly distributed load of intensity w_u is given by the expression ,

$V_{u.max} = r\, w_u\, L$

Where, r = 0.5 for beam simply supported or continuous at both ends,
= 0.45 at the simply supported end of end span of continuous beam,
= 0.60 at the continuous end of end span of continuous beam.

Design shear force for support offering compression reaction is equal to:

$$V_{uD} = V_{u.max} - (b_s/2+d)\, w_u \qquad .....(4.2\text{-}1a)$$

Design shear force for support offering tension reaction -

$$V_{uD} = V_{u.max} - w_u\, b_s/2 \text{ (where } b_s \text{ is width of support)} \qquad .....(4.2\text{-}1b)$$

4.2.3 Shear Strength of Section in Diagonal Compression ($V_{uc.max}$) :

$$V_{uc.max} = \tau_{uc.max}\, bd \qquad .....(4.2\text{-}2)$$

where $\tau_{uc.max}$ depends upon the grade of concrete.
$\tau_{uc.max}$ = 2.5 N/mm² for concrete of grade M 15, and
= 2.8 N/mm² for concrete of grade M 20.

If shear at a section exceeds $V_{uc.max}$, the section is inadequate and should be revised either by changing b or d

4.2.4 Shear Resistance of R.C. Member with Main Steel but without Shear Reinforcement : (V_{uc})

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

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 is functon of grade of concrete and of the percentage of tension steel. It is given by the following relation:

$V_{uc} = \tau_{uc}\, bd$

where, $\tau_{uc} = (0.85\sqrt{.8f_{ck}})(\sqrt{1+5\beta} - 1)/(6\beta) \qquad .....(4.2\text{-}3)$

$\beta = 0.8f_{ck}/(6.89p_t)$ but >= 1.0

$p_t = 100\, A_{st}/(bd)$

A_{st} = area of longitudinal tension steel at the section.

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 M15 and M20 are given in Table 4.2 below. These values will be increased by modification factors given below for caculating shear resistance of concrete in slabs.

Depth of slab in mm	>=300	275	250	225	200	175	<=150
Modification factor k	1.00	1.05	1.10	1.15	1.20	1.25	1.30

Table 4.2 : Shear Strength of Concrete in N/mm^2

Pt %	τ_{uc} M15	τ_{uc} M20	Pt %	τ_{uc} M15	τ_{uc} M20	Pt %	τ_{uc} M15	τ_{uc} M20
0.10	0.24	0.24	0.70	0.53	0.55	1.50	0.68	0.72
0.15	0.28	0.29	0.80	0.55	0.57	1.60	0.69	0.73
0.20	0.32	0.33	0.90	0.58	0.60	1.70	0.71	0.75
0.25	0.35	0.36	1.00	0.60	0.62	1.80	0.71	0.76
0.30	0.38	0.39	1.10	0.62	0.64	1.90	0.71	0.77
0.40	0.43	0.44	1.20	0.63	0.66	2.00	0.71	0.79
0.50	0.46	0.48	1.30	0.65	0.68	2.10	0.71	0.80
0.60	0.50	0.51	1.40	0.67	0.70	2.20	0.71	0.81
0.70	0.53	0.55	1.50	0.68	0.72	=>2.30	0.71	0.82

4.2.5 Shear Resistance of Shear Reinforcement : (V_{us})

(1) Types of Shear Reinforcement :
(i) Bent-up Bars, (ii) Vertical Stirrups,
(iii) Inclined Stirrups or Series of Bent-up Bars .

(2) Shear Resistance of Isolated Bent up Bars at a section :

$V_{usb} = 0.87 f_y A_{sb} \sin \alpha$ .....(4.2-4a)

where, A_{sb} = area of bent up bar/s.
α = inclination of bent-up bar/s with the axis of the member

(3) Shear Resistance of Inclined Stirrups or a series of Bent-up bars bent at different sections at a pitch distance apart :

$V_{usi} = 0.87 f_y A_{si}\, d\, (\sin \alpha + \cos \alpha)/s$ .....(4.2-4b)

(4) Shear Resistance of Vertical stirrups with area of vertical legs A_{sv} :

$V_{usv} = 0.87 f_y A_{sv}\, d/s$ .....(4.2-4c)

where, s = spacing of stirrups.

(5) Total Shear Resistance of Shear Reinforcement for combination of (4) with (2) or (3): $V_{us} = V_{usb} + V_{usv}$, but $V_{usb} <= 0.5 V_{us}$ .....(4.2-4d)

(6) Minimum Stirrups : When shear at a section V_u exceeds 0.5 V_{uc},

$A_{sv}/(bs) > 0.4/f_y$ .....(4.2-5a)

or $s <= A_{sv} f_y/(0.4b)$ .....(4.2-5b)

(7) Shear Resistance of Minimum Stirrups :

$V_{usv.min} = 0.348bd$ say .35bd .....(4.2-6)

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

$V_{ur.min} = V_{uc} + V_{usv.min}$ .....(4.2-7)

(9) Maximum Spacing : s <= 0.75d or 450mm whichever is less.

(10) Minimum Spacing : s >= 75 mm for ease of concreting.

4.2.6 Shear Design in case of Bar Curtailment :

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 1.5x shear at the section.

(ii)Additional stirrups with area equal to area of minimum stirrups are provided for a distance .75d beyond the cutoff point with resultant spacing not exceeding d/(8B) where B 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.

Part B : Torsion

According to the latest theory of torsion for R.C. members, the torsional moment acting on the member is converted into equivalent bending moment and equivalent shear which is added to actual B.M. and shear acting at the section. The member is then designed for this total resulting B.M. and shear.

4.2.7 Equivalent Bending Moment : (M_{ue})

$M_{ue1} = M_u + M_t$ .....(4.2-8a)

where $M_t = T_u.(1+D/b)/1.7$

T_u = Torsional moment at the section,

M_u = Actual bending moment at the section.

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

$M_{ue2} = M_t - M_u$. .....(4.2-8b)

4.2.8 Equivalent Shear (V_{ue})

$V_{ue} = V_u + 1.6T_u/b$ .....(4.2-9a)

The shear reinforcement consists of only closed vertical stirrups.
It will be designed for greater of the following:

$V_{us} = V_{ue} - V_{uc}$ .....(4.2-9b)

or $V_{us} = (T_u/b_1 + V_u/2.5)(d/d_1)$ .....(4.2-9c)

where b_1 = horizontal distance between the centres of outermost corner bars,
d_1 = vertical distance between the centres of outermost corner bars.

4.2.9 Spacing of Stirrups :

Spacing of closed stirrups $s = 0.87 f_y A_{sv} d/V_{us}$ .....(4.2-10)

Where V_{us} is given by Eq.(4.2-9).

$s <= x_1$ or $(x_1+y_1)/4$ or 300mm or 0.75d whichever is the least.

where x_1 = horizontal distance between centres of vertical legs of stirrups,
y_1 = vertical distance between centres of horizontal legs of stirrups.

4.2.10 Side Face Steel :

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

4.3 BOND

4.3.1 Definition :

Bond is defined as interfacial shear acting over the contact surface of the bar which prevents relative movement between the two constituent materials. It is necessary for transfer of strains and hence forces from concrete to steel so that the two materials act together as one composite material. In absence of bond, the force transfer can be made by mechanical anchorage at the end.

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 case of deformed bars.

4.3.2 Bond Strength :

The Code gives following values of bond stength for limit state method (for ultimate limit state) for bar diameters less than 36 mm, for plain round bars in tension.

Grade of Concrete	M15	M20	M25
Bond strength τ_{bd} in N/mm^2	1.0	1.2	1.4

Increase in strength = 25 % for bars in compression
= 60 % for deformed bars (irrespective of its grade)

4.3.3 Bond Length or Development Length :

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

$$L_d = \{\sigma_s/(4\ \tau_{bd})\}\ \phi = k\phi$$

where, $k = \sigma_s/(4\ \tau_{bd})$ is known as development length factor. .....(4.3-1)

For developing full strength in the bar, $\sigma_s = .87f_y$.

$\therefore k = [(0.87f_y/(4\ \tau_{bd})]$

For round bars under tension, the values of k are given below for different grades of concrete and steel.

Concrete grade	M15			M20		
Steel grade	Fe250	Fe415	Fe500	Fe250	Fe415	Fe500
Development length factor	55	57	68	46	47	57

4.3.4 Standard End Anchorages : Hooks and Bends

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.

4.3.5 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 cirums-tances shall be less than $2L_d$).

(b) Members under Bending :

(i)Check for Development Length for Support Steel or Negative Moment Steel: At least one-third of total tension steel provided for negative moment at support shall extend for a length not less than greter of [x_o + (d or 12 ϕ or clear span/16 whichever is greater) and L_d,from the point of critical stress where xo is the length of negative moment region from the centre of support.

The point of critical stress is a point of maximum stress which is either the centre of an intermediate support or the face of cantilever support.

(ii)For midspan steel or positive moment steel, this check shall be applied at the point of zero bending moment (i.e. discontinuous end of a beam supported on two supports, or the point of contraflexure).

Required $L_d \le$ Available bond length = $M_1/V + L_{ex}$,($1.3M_1/V+L_{ex}$ at supports offering compressive reaction) .....(4.3-2)

where, M_1 = Moment of resistance of beam at the section .

L_{ex} = extension of bars beyond the centre line of support (but <= d or 12ϕ whichever is greater)>= ($L_d/3-b_s/2$) where b_s is the breadth of the support.

If moment of resistance of bars at midspan (all having the same diameter) is assumed to be equal to the design moment M_{max} at midspan, then

$M_1/M_{max} = A_{st1}/A_{st.max} = N_1/N_{max}$ or $M_1 = (N_1/N_{max}).M_{max}$

where A_{st1} and $A_{st.max}$ are the areas of tension steel at the support and at the midspan respectively, and N_1 & N_{max} are the number of bars at support and at mid-span respectively.

Also, required minimum A_{st1} at support for adequate end anchorage is $A_{st}/3$ at discontinuous end and $A_{st}/4$ at continuous end where A_{st} = area of tension steel at midspan.

4.3.6 Curtailment of Bars

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 ø or 'd' whichever is greater and subject to provision of additional shear Reinforcement as explained in Sect. 4.2.6.

4.4 SERVICEABILITY : (Deflection and Cracking)

4.4.1 Defelction

(a) Allowable Deflections:

The Code prescribes the following allowable deflections.

Total deflection <= Span/250

Deflection after the erection of walls (load transfer) <= Span/350 or 20mm whichever is less.

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

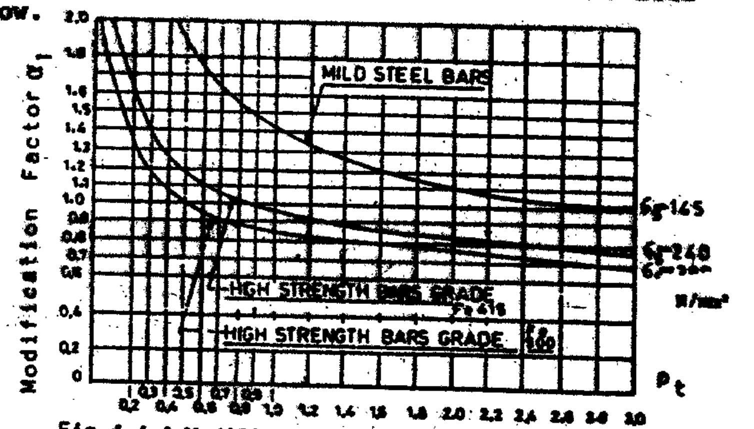


Fig.4.4.1 Modification factor α_1 for p_t

(b) Allowable L/d Ratio (r)

Allowable L/d ratio

$r_a = \alpha_1 x \alpha_2 x \alpha_3 x \alpha_4 x\ r_b$

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

α_1 = modification factor for percentage of tension steel(p_t) required as given by Fig.4.4.1

α_2 = modification factor for percentage of compression steel steel p_c as given by Fig.4.4.2.

α_3 = modification factor for b_w/b_f to be obtained from Fig.4.4.3 (page 59).

α_4 = modification factor for span > 10m = 10/span.

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

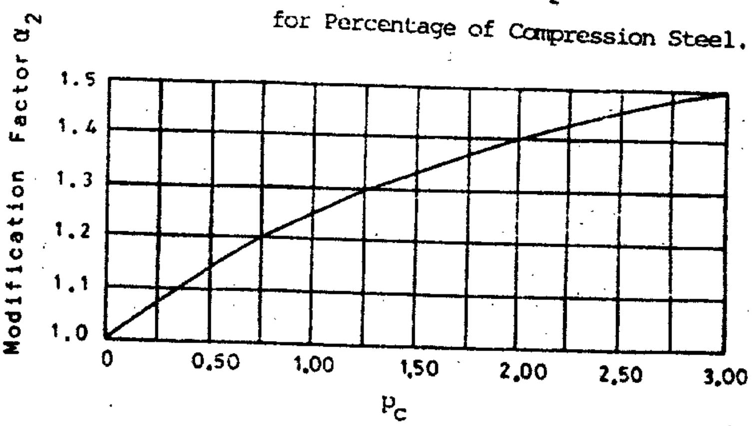
In case of slabs, α_2 =1 (for pc =0),α_3 and α_4 do not apply.

For two-way slabs having span L <=3.5 m and LL <=3 kN/m² and steel of grade Fe415, allowable L/D ratio r_a = 28 for simply supported and 32 for continuous.

4.4.2 Cracking

Control of cracking is achieved by observing all requirements prescribed by the code for minimum cover, minimum and maximum bar diameters and minimum-maximum amount of reinforcement for various members of the frame (e.g. slabs, beams, columns, footings). These are given in the design procedure for these members in the subsequent section.

Fig. 4.4.2 Modification factor α_2 for Percentage of Compression Steel.



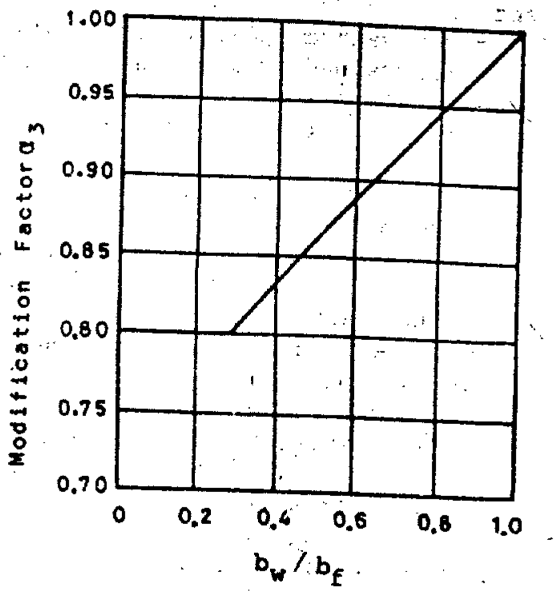


Fig.4.4.3 Modification Factor α_3 for b_w/b_f

4.5 AXIAL COMPRESSION AND BENDING

4.5.1 Basic Design Aspects :

(a) Basic Assumptions: :

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

0.002 for pure axial compression

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).

$0.0035 - 0.75 \times \varepsilon_{min}$ where ε_{min} is the minimum compressive strain in concrete under combined bending and axial compression for uncracked section (i.e.entire section under compression).

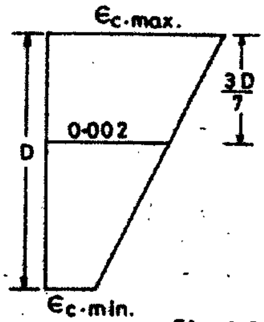


Fig.4.5.1

This is equivalent to a compressive strain of 0.002 at a distance of 3D/7 from the highly compressed edge. See Fig.4.5.1.

(b) Unsupported Length (L) :

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.

Thus, L = Floor to floor height – total depth of shallower beam.

(c) Effective Length (L_{eff}) :

(i) General : Effective length of a column is defined as the length between the points of contraflexure in a buckled column. It depends upon the restraints against rotation and translation (i.e. sway) at the two ends. The effective lengths of a column for various end conditions are given below.

For columns without sway (i.e. effectively held in position at both ends) and L_{eff} =

- restrained against rotation at both ends (both ends fixed) 0.65 L
- restrained against rotation at one end only (one end fixed, other hinged) 0.8 L
- free to rotate at both ends (both ends hinged) 1.0 L

For columns with sway (i.e. not effectively held in position at any one of the two ends): L_{eff} =

- Restrained against rotation at both ends. 1.2 L
- Restrained against rotation fully at one end and partially at the other end. 1.5 L
- One end is held in position but free to rotate while the other end is not held in position but restrained against rotation. 2.0 L
- One end is held in position and restrained against rotation also while the other is fully free in position and rotation. 2.0 L

(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.**4.5.2(a) for columns not free to sway and from Fig.4.5.2(b) for columns which are free to sway.

The ratio L_{eff}/L depends upon the rotation release factors $\beta 1$ and $\beta 2$ at top and bottom of column respectively where,

$$\beta = \Sigma k_c / (\Sigma k_c + \Sigma k_b) \qquad (4.5\text{-}1)$$

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 $k_b = \frac{1}{2}$ I/L and for unbraced frames $k_b = (3/2)\,(I/L)$

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), $k_c = 0.75I/L$. β has two values β_1 at top and β_2 at bottom of column.

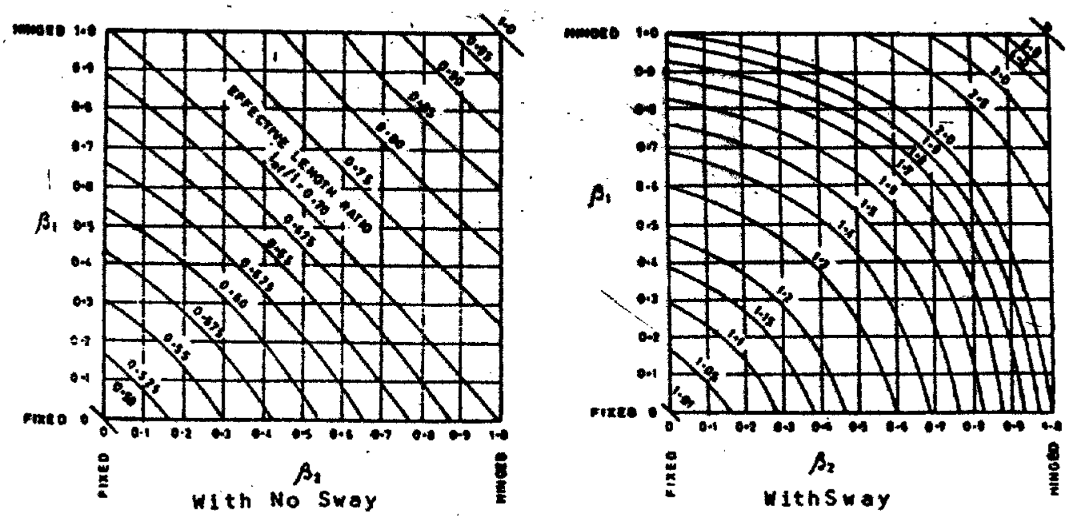


Fig.4.5.2 Effective Lengths of Columns in Frames

(d) Slenderness of Column :

It is decided by the slenderness ratio of the column and is defined as the ratio of the effective length of column to its lateral dimension perpendicular to the axis of bending. It is equal to:

$L_{eff.x}/D$ for bending @ x axis i.e. an axis perpendicular to D.
$L_{eff.y}/b$ for bending @ y axis i.e. an axis perpendicular to b.

A column is said to be short when the slenderness ratio is less than or at the most equal to 12; else the column is slender or long.

(e) Minimum Eccentricity (e_{min}) :

Every column will be designed for a minimum eccentricity given by

$e_{x.min} = L/500 + D/30$ from x-axis : Both subject to
$e_{y.min} = L/500 + b/30$ from y-axis : minimum of 20 mm. .....(4.5-2)

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, $e_{x.min}$ will be taken while for buckling about y-axis, $e_{y.min}$ will only be taken.

(f) Requirements of Reinforcement :

I - Longitudinal Reinforcement

(i) Minimum Percentage 0.8% (ii) Maximum Percentage 6% preferably <= 4%
(iii) Minimum Number = Number of corners of the cross section
= 6 for circular section,
(iv) Maximum Spacing between bars 300 mm.

II - Transverse Reinforcement (Lateral Ties) :

(i) Minimum Diameter (ϕtr) =1/4th the diameter of main bar >= 5mm ,
(ii) Maximum Spacing s = least of (16 ϕ, 48 ϕtr and least lateral dimension).

4.5.2 Axially Loaded Columns :

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)

$$P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc} \quad .....(4.5\text{-}3)$$

(ii) When minimum eccentricity <= 0.05 x lateral dimension,

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc} \quad .....(4.5\text{-}4)$$

(iii) For columns with width < 400mm and e_{min} = 20 mm (i.e. $e_{min} > .05b$)

$$P_u = \lambda(0.4 f_{ck} A_c + 0.67 f_y A_{sc}) \quad .....(4.5\text{-}4a)$$

where A_c = Area of Concrete in compression,
A_{sc} = Area of Steel in Compression, and the values of λ are as under:

Grade of Concrete Steel	Width of column (b) in mm. 150	200	230	250	300	350 & above
	Reduction Factor λ					
M 15 - Fe 250	0.75	0.87	0.92	0.94	0.98	1.00
M 15 - Fe 415	0.70	0.85	0.90	0.92	0.96	1.00

(b) Columns with Helical Ties :

P_u = 1.05 x axial strength of columns with lateral ties; provided $V_h/V_k \Rightarrow 0.36[(A_g/A_k)-1)].(f_{ck}/f_y)$ .....(4.5-5)

where V_h = Volume of helical steel per pitch length = $\pi.D_k.A_{sh}$,
V_k = Volume of concrete core per pitch length = A_k s
A_g = gross cross-sectional area,
A_k = Area of concrete core = $\pi.D_k^2/4 - A_{sc}$,
D_k = Diameter of concrete core measured to outside of helical steel,
= D - 2 x clear cover + ϕ_{sh} where ϕ_{sh} is diameter of helical steel.
A_{sh} = Area of helical steel,
s = spacing of helical steel.

4.5.3 Eccentrically Loaded Columns - Uniaxial Bending :
(Strength under Combined Axial Compression and Uniaxial Bending)

(a) Neutral Axis lying Outside the Section ($X_u > D$) :

$P_u = P_{uc} + \Sigma P_{usi}$
where, $P_{uc} = C_1 f_{ck} bD$ .....(4.5-6a)
$\Sigma P_{usi} = \Sigma A_{si} (f_{si} - f_{ci})$ .....(4.5-6b)
$\therefore P_u = C_1 f_{ck} bD + \Sigma A_{si} (f_{si} - f_{ci})$ .....(4.5.6c)
$M_u = M_{uc} + \Sigma M_{usi}$ .....(4.5-6d)
where, $M_{uc} = P_{uc} D (0.5 - C_2) = C_1 f_{ck} bD^2(0.5 - C_2)$ .....(4.5-7a)
$\Sigma M_{usi} = \Sigma P_{usi} x_i = \Sigma A_{si} (f_{si} - f_{ci}) x_i$ .....(4.5-7b)
$\therefore M_u = C_1 f_{ck} bD^2 (0.5 - C_2) + \Sigma A_{si} (f_{si} - f_{ci}) x_i$ .....(4.5-7c)
where, $C_1 = 0.446 (1 - C_3/6)$, ...(4.5-7d)
$C_2 = (0.5 - C_3/7)/(1.0 - C_3/6)$, ...(4.5-8a)
$C_3 = (8/7)[4/(7k_u - 3)]^2$, ...(4.5-8b)
$k_u = x_u/D$, ...(4.5-8c)
i = serial number of row of reinforcement, i varies from 1 to n where, 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 $= .002(x_u-D/2+x_i)/(x_u-3D/7)$ (4.5-9a)
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 $\epsilon_i > 0.002$,
$f_{ci} = 446\, \epsilon_i (1 - 250\epsilon_i) f_{ck}$ for $\epsilon_i <= 0.002$, ...(4.5-10a)
For Fe250, $f_{si} = \epsilon_i E_s$ but $<= 0.87 f_y$, ...(4.5-10b)
For Fe415, $f_{si} = \epsilon_i E_s$ when $\epsilon_i <= 0.00144$, and
f_{si} shall be obtained from Table 2.10 when $\epsilon_i > 0.00144$.

(b) Neutral Axis lying Inside the Section ($X_u <= D$) :

Equations (4.5-6),(4.5-7),(4.5-8) above hold good in this case also but the values of C_1, C_2 and ϵ_i will be taken as under.
$C_1 = 0.36k_u$, and $C_2 = 0.416k_u$ .....(4.5-8d)
and $\epsilon_i = 0.0035 (x_u - D/2 + x_i)/x_u$ .....(4.5-9b)

(c) $P_u - M_u$ Interaction Diagram :

A diagram showing strength of a member in axial compression (P_u) and the accompanying ultimate flexural strength M_u, is known as P_u-M_u interaction diagram

For a member of given cross-section, the values of P_u and M_u are given by Eq.4.5-6d and 4.5-7d. In these equations, C_1 and C_2 are functions of x_u (and hence of k_u), A_{si} is a function of number of rows and number of bars in each row and hence of arrangement of bars, f_{si} and f_{ci} are functions of x_u and x_i (i.e. k_u and x_i/D).

Consequently, P_u and M_u are functions of

(i) Grade of concrete f_{ck},
(ii) Grade of steel f_y,
(iii) Cross-section b,D,
(iv) Area of steel A_{si},
(v) Arrangement of steel, and
(vi) Position of (Depth) of neutral axis.

A curve showing values of P_u and M_u can be obtained for given values of quantities (i) to (v) above for different values of x_u (or k_u). A typical P_u–M_u interaction curve is shown in Fig.4.5.3(a).

Following are 5 critical points on the curve.

Point A corresponding to zero axial load (pure bending),
Point B corresponding to balanced or critical section,
Point B' corresponding to P_u for $x_u = D$ (Limit of cracked section),
Point C corresponding to P_u with minimum eccentricity,
Point D corresponding to P_{uz} with zero eccentricity.

In columns, the section is considered to be balanced when the maximum compressive strain in concrete reaches a value 0.0035 and the maximum tensile strain in steel reaches a value 0.002. See Fig.4.5.3(b).

The balanced section has maximum curvature and maximum bending strength. The values of P_u and M_u for this section corresponding to k_u = .0035/(0.0055)=7/11 are known as P_{ub} and M_{ub} respectively.

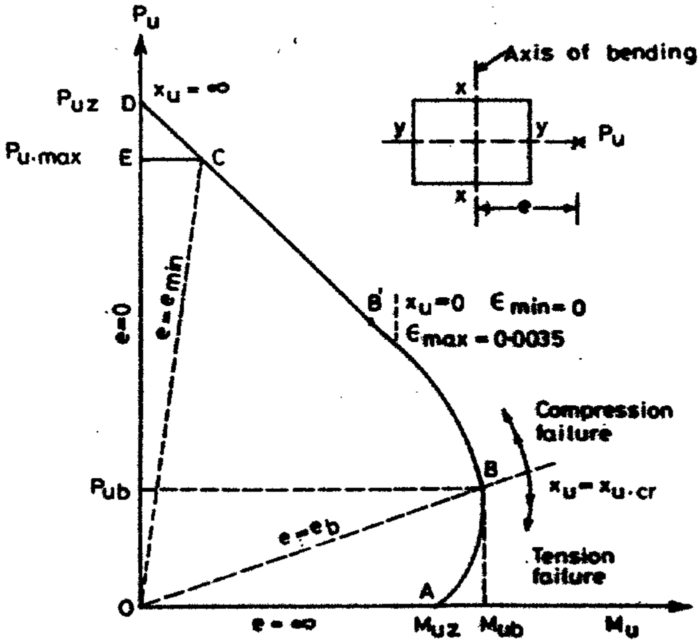


Fig.4.5.3(a) P_u-M_u Interaction Diagram for a Column

The above diagram between B and D can be approximated to a straight line BD and following equations can be obtained corelating P_u and M_u with P_{ub}, M_{ub} and P_{uz} for any point on BD i.e. for $P_u > P_{ub}$.

$$M_u/M_{ub} = (P_{uz}-P_u)/(P_{uz}-P_{ub})$$

$$\text{or } M_u = \{(P_{uz}-P_u)/(P_{uz}-P_{ub})\}.M_{ub} \quad .....(4.5\text{-}11)$$

For sections reinforced with equal steel only on opposite faces, the values of P_{ub} and M_{ub} can be obtained using following equations (ignoring the concrete displaced by A_{sc}) :

$$P_{ub} = C_4 f_{ck} bD + C_5 f_y A_{sc} \quad .....(4.5\text{-}12)$$

$$\text{and } M_{ub} = C_6 f_{ck} bD^2 + C_7 f_y A_{sc} D \quad .....(4.5\text{-}13)$$

where values of constants C_4,C_5,C_6 & C_7 are as follows:

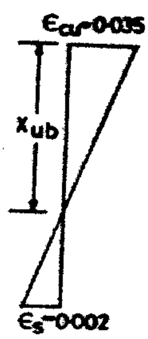


Fig.4.5.3(b) Stress Diagram for Balanced Section of a Column

d'_c/D	0.05	0.10	0.15	0.20
C_4	0.218	0.206	0.195	0.183
C_5	0.053	0.090	0.169	0.250
C_6	0.054	0.054	0.0535	0.053
C_7	0.416	0.350	0.267	0.190

The interaction diagrams available in most of the design aids are usually in non-dimensional form. They give variation of $P_u/f_{ck}bD$ with $M_u/f_{ck}bD^2$.

For drawing these diagrams, their basic equations, and their use refer to Sect.9.5.4. of Author's Textbook on Limit State Theory and Design of Reinforced Concrete.

4.5.4 Eccentrically Loaded Columns - Biaxial Bending :

The safety of a column subjected to axial compression and biaxial bending can be verified using the following interaction formula.

$$(M_{ux}/M_{ux1})^{\alpha_n} + (M_{uy}/M_{uy1})^{\alpha_n} <= 1 \qquad .....(4.5\text{-}14)$$

where M_{ux} = Given bending moment about x axis (bisecting D),
M_{uy} = Given bending moment about y axis (bisecting b),
M_{ux1} = Moment resisting capacity of the column under axial load Pu for uniaxial bending about x-axis only,
M_{uy1} = Moment resisting capacity of the column under axial load Pu for uniaxial bending about y-axis only.

α_n is a factor accounting for the effect of given P_u in relation to pure axial resisting capacity P_{uz}, and is a function of ratio P_u/P_{uz}.

α_n = 1 for P_u/P_{uz} <= 0.2
α_n = 2 for P_u/P_{uz} >= 0.8

The intermediate values are obtained by linear interpolation.

4.5.5 Slender Columns :

In case of slender columns i.e. columns having L_{eff}/Lateral dimension > 12, bukling effect gives rise to additional moments which are required to be taken in addition to the initial moments.

(a) Additional Moments due to Slenderness :

$$M_{ax} = (P_u\ D/2000)(L_{eff.x}/D)^2\ k \qquad .....(4.5\text{-}15a)$$
$$M_{ay} = (P_u\ b/2000)(L_{eff.y'}/b)^2\ k \qquad .....(4.5\text{-}15b)$$
where, $$k = (P_{uz} - P_u)/(P_{uz} - P_{ub}) <= 1.0 \qquad .....(4.5\text{-}16a)$$

It may be noted that for design, a trial section with trial reinforcement is required to be assumed, to begin with, to arrive at additional moments because the reduction factor k depends upon P_{uz} and P_{ub} which can be obtained only when the section is known. $P_{ub} = (k_1+k_2p/f_{ck})f_{ck}bD = k_1 f_{ck}bD + (k_2 A_{st} \times 100)$ (4.5 -16b)

Values of K_1 & K_2 are given in Table E-6

(b) Initial Moments :

(i) For braced columns :
Single curvature, $M_i = 0.6M_{u2} + 0.4M_{u1}$ but >= $0.4M_{u2}$ .....(4.5-17a)
Double curvature, $M_i = 0.6M_{u2} - 0.4M_{u1}$ but >= $0.4M_{u2}$ ....(4.5.17b)
where M_{u1} = Smaller end moment, and M_{u2} = Larger end moment.

(ii) For unbraced columns :
$M_i = M_{u2}$
The initial moment M_i <= Moment due to minimum eccentricity.

(c) Total Design Moment :

M_{uT} = Initial Moment + Additional Moment = $M_i + M_a$
but not less than $M_{u.min}$ or M_{u2} whichever is greater. .....(4.5-18)

Chapter – 5
Design of Members

The procedure for design of component members, namely, slab, stair, beam, column, and column footing using theory and equations i.e. from first principles has been presented in Part A of this chapter. Part B gives the procedure for design of members using design aids i.e. charts and tables suitable for calculations in a tabular form.

Part A : Design by use of Theory and Equations
(From first principles, without the use of Design Aids)

5.1 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 between columns i.e. the span lengths of slabs and beams,
(iii) marking of slabs, beams, and columns using one of the marking schemes given in Sect. 1.4 or any other standardised/established method.

After the preparation of structural plan, the calculations will be done for unit loads such as

(i) unit loads on slabs of roof, floor, balconies, stairs, W.C. and bathrooms, lofts etc, (in kN/m/m).
(ii) unit loads of walls (external, internal) per metre height, floor to floor height, and floor to beam soffit height etc, (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 slabs, 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 is to be started urgently, it may be necessary to give sizes of footing and ground floor columns first. In such case, the design will first be done of footings and columns by estimation of approximate equivalent axial load on columns as detailed in Sect. 3.4.3 giving sufficient allowance for effect of continuity of slabs and beams, uniaxial/biaxial bending in columns due to fixity with beams; slenderness of column etc. wherever necessary. This approach, ofcourse, should be avoided as far as possible.

In the text that follows, both the procedures have been given. However, the presentation follows the sequence slab-stair-beam-column and column footing.

5.2 DESIGN OF SLABS

5.2.1 General

To begin with, classify the slabs to be designed into the following two main classes, namely,

A-One-way Slab, and B-Two-way Slab.

This is because the design procedure for above two types is different.

After classification, categorise one-way slabs into following categories depending upon the end conditions because end conditions do affect the design, particularly the detailing of reinforcement.

(1) Simply Supported Slabs : End Condition No. 1 (EC = 1).
(2) Slabs Simply Supported at One End and Continuous at the Other : End Condition No. 2 (EC = 2).
(3) Slabs Continuous at Both Ends : End Condition No. 3 (EC = 3).
(4) Cantilever Slabs. : End Condition No. 4 (EC = 4).
(5) Overhanging Slabs. : End Condition No. 5 (EC = 5).

Two-way slabs may be categorised not only according to the boundary conditions but mainly according to the condition of the corner restraint 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 may be designed according the procedure given in Sect.5.2.2 and 5.2.3.

5.2.2 Design of One-Way Slab

Steps:

1. *Slab Mark*: Write the slab mark or designation such as S1,S2,etc.
2. *End Condition* (EC) : Write the end condition No. according to the category of the slab as given in Section 5.1.1 above, or describe the end condition wherever necessary.
3. *Span* (L) : Write the effective span (L) from the given clear span (L_c), and the width of support. Theoretically, the effective span for slab or beam is taken according to Cl.21.2 of the Code as follows :

End Condition No. Effective Span

EC = 1 : L = Centre to Centre distance between the supports (L_{cc}) or clear span (L_c) + effective depth (d), whichever is less.
Effective depth = d = Total Depth (D) − effective cover (d') to the centre of tension steel.

For slabs,
Effective cover d' = clear cover + diameter of main bar ø/2.
Clear cover = 15 mm or ø whichever is greater.
ø is, normally = 8 mm to 12mm
Thus, d' = 20 mm approximately.

EC = 2 : When the width of support at continuous end b_{s1} <= $L_c/12$:
L = L_{cc} or (L_c + d) whichever is less. This is same as end condition EC = 1 given above.
When the width of support at continuous end $b_{s1} > L_c/12$:
L = (L_c + $b_{s2}/2$) or (L_c + d/2) whichever is less
where bs2 is the width of support at simply supported end.

EC = 3 : L = L_{cc} or (L_c+d) whichever is less when $b_s <= L_c/12$.
L = L_c when $b_s > L_c/12$.

EC = 4 : L = L_c+d/2 where L_c is the length of cantilever to face of support

EC = 5 : For overhanging portion of the slab/beam, L = L_c + $b_{s1}/2$.
For portion of beam between two supports:
For beam/slab overhanging at one end only L= L_c+$b_{s1}/2$+$b_{s2}/2$
For beam/slab overhanging on both sides, L= L +b_{s1}.
where, b_{s1} = width of support at continuous end, and
b_{s2} = width of support at discontinuous end.

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 effectice span is taken equal to centre to centre distance between the supports on safer side.

4. *Trial Section* : In case of slabs, it is the total depth of slab D in mm. It is normally, governed by serviceability criteria.

Reqired D = span/(Allowable L/d ratio,ra)+effective cover(d') or $D = L/r_a + d'$

r_a = Basic L/d ratio(r_b) x α_1.

r_b = 20 for simply supported, 26 for continuous & 7 for cantilever.

α_1 depends upon p_t%.

Assume p_t = 0.3 % to 0.45 % for steel of grade Fe415.

Obtain α_1 from Fig.4.4.1 corresponding to assumed p_t.

For p_t = 0.3 %, α_1 = 1.4 for Fe415.

Then $r_a = r_b \times \alpha_1 = 20 \times 1.4 = 28$ for simply supported slab.

Multiply above value by (26/20=1.3) for continuous slab and by (7/20=0.35) for cantilever slab.

Eff.cover d'= 20 mm approximately.

∴ Trial D = L/28 + 20 mm (for s.s.slab) to be rounded to multiple of 10 mm

Trial d = Trial D - 20 mm.

5. *Loads* :

Calculate load in kN/m on a 1 metre wide strip of slab.

Dead Load : Self weight w_s = 25D where D shall be in metre.

Floor Finish FF As per data. See Table 2.1 or A-1.

Total dead load DL = w_s + FF.

Live Load : LL As per data. See Table 2.3.

Total working load w = DL + LL

Total imposed load w_i = FF + LL (i.e. w - w_s)

Total ultimate load w_u = 1.5xw.

When maximum and minimum loads are required for analysis,

Maximum design load w_{max} = 1.5(DL +LL) = w_u, and

Minimum design load w_{min} = 0.9 DL.

6. *Design Moments* :

$M_{u.max} = \alpha \times w_u L^2$ where α is design moment coefficient.

In general, the values of design moment coefficients may be taken as under.

End Condition No.	EC	1	2	3	4	
Design Moment Coefficient	α	1/8	1/10	1/12	1/2	for UD load.

Alternatively, coefficients may be taken from Table in Appendix B or bending moments calculated from first principles.

7. *Check for Concrete Depth*:

Calculate the maximum moment carrying capacity of a balanced section.

$M_{ur.max} = R_{u.max}\ bd^2$. For slabs, b = 1 metre i.e. 1000 mm.

For M15-Fe415 , $R_{u.max}$ = 2.07 N/mm². For other grades, refer to Table 4.1.1.

Check that $M_{ur.max}$ => $M_{u.max}$ for adequacy of concrete depth for strength.

Normally, this condition is satified and therefore it may safely be omitted.

8. *Main Steel* (A_{st}):

Required $A_{st} = (0.5 f_{ck}/f_y)\left[1 - \sqrt{1 - 4.6 M_{u.max}/(f_{ck} bd^2)}\right] bd$ >= $A_{st.min}$

where $A_{st.min} = p_{t.min}\ bD$.

$p_{t.min}$ = 0.12 % for Fe415 , and 0.15 % for Fe250.

Assume bar diameter (8mm for steel of grade Fe415, and 10mm, 12mm for Fe250).

Required spacing = s = 1000 a_{st}/A_{st} where, a_{st} is area of one bar.

Ø mm	8	10	12
a_{st} mm²	50	78	113

Maximum spacing s<= 3d or 450 mm whichever is less (In practice, 200 mm).

Minimum spacing s>= 75 mm from practical consideration of good concreting.

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 to 150 mm.

9. *Check for Serviceability* :

Calculate required p_t% (at critical section as defined in Sect. 4.4.1)

= Required A_{st} x 100/bd (b = 1000 mm for slabs). This shall not exceed assumed p_t in Step 4 above. If it exceeds, apply the following check.

Calculate α_1 corresponding to the required p_t, from Fig.4.4.1.
Calculate the depth required for serviceability, $D_{ser} = L/(r_b \times \alpha_1) + d'$.
Check that $D >= D_{ser}$. If not, increase the trial depth D and revise design calculations from Step 4 above.

10. *Distribution Steel* :

Required $A_{st.min} = p_{t.min}\ bD/100 = 10xp_{t.min}xD$ since b = 1000mm for slabs
$p_{t.min}$ = 0.15 % for steel grade Fe250 and 0.12 % for HYSD bars.
Assume bar diameter (6 mm for steel grade Fe250 and 8 mm for Fe415).
Required spacing, $s = 1000\ a_{st}/A_{st.min}$ to be rounded off on lower side in multiple of 10 mm or 25 mm as desired.
Maximum spacing, s=< 5d or 450 mm whichever is less.
In practice, spacing is kept between 150 mm to 300 mm.

11. *Check for Shear* :

(a) Calculate design (maximum) shear. In case of slabs, design shear may be taken equal to maximum shear at support.

$V_{u.max}$ = 0.50 q_u L at both ends in case of slabs with EC = 1 & 3.
$V_{u.max}$ = 0.45 q_u L at simply supported discontinuous end for a slab with end condition EC = 2.

while, $V_{u.max}$ = 0.60 q_u L at continuous end of the above slab with EC = 2.

In other cases, the maximum shear may be calculated from principles of mechanics.

(b) Calculate shear resistance (V_{uc}) of slab. This may be obtained from relation, $V_{uc} = \tau_{uc}.bd$ (b = 1000 mm in case of slabs).

τ_{uc} depends upon $p_t = 100\ A_{st1}/bd$. It is obtained from Table 4.2. where, A_{st1} = area of tension steel at support. It is the bottom steel at simply supported end and top steel at continuous end.

$A_{st1} = A_{st}/2$ if alternate bars from midspan are bent to top at simple support and $A_{st1} = 2A_{st}/3$ if every third bar from midspan steel is bent up.

Shear resistance (V_{uc}) of slab is greater than that of beam because of the membrane action of the slab due to its thinness in relation to its width. The increased resistance is obtained by applying a multiplying factor k given below.

Depth of slab in mm	>=300	275	250	225	200	175	<=150
Multiplying factor k	1.00	1.05	1.10	1.15	1.20	1.25	1.30

Normally, the depth of slabs in buildings is less than 150 mm and therefore a factor 1.3 is used.

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 distribted load and therefore, many times omitted in design calculations.

It may be noted that when the check for shear is obtained, in case of slabs, it is not necessary to provide minimum stirrups as they are required in case of beams.

(c) Calculation of End Shears (Loads) on Supporting Beams :

On Long Edge : Same as $V_{u.max}$ above.
On Short Edge when it does not carry the long beam as a secondary beam and that too for the purpose of design of supporting beam only and not for load on column. (Refer to Sect 3.4.1).
Triangular load with central ordinate = $q_u\ L_x/4$.
Equivalent Load on beam for bending $q_u\ L_x/6$
for shear $q_u\ L_x/8$.

12. *Check for Development Length* :

Required L_d <= Available $L_d = 1.3M_1/V+L_{ex}$ (See Sect.4.3.5, Eq.4.3-2).
For slabs, $L_{ex}=(b_s-50)/2$. $\therefore$ Required $M_1 = V(L_d-L_{ex})/1.3$. Now, $V = V_{u.max}$.
Required $(A_{st1}/A_{st.max}) = (M_1/M_{u.max}) = V_{u.max}(L_d-L_{ex})/(1.3M_{u.max})$.
Assuming $V_{u.max} = r.q_uL$ and $M_{u.max} = \alpha\, q_uL^2$
Required $(A_{st1}/A_{st.max}) = r.q_uL(L_d-L_{ex})/(1.3x\,\alpha\, q_uL^2) = r(L_d-L_{ex})/(1.3x\alpha L)$.

5.2.3 Design of Two-Way Slab

Steps:

1. *Slab Mark* :
2. *End Condition No.* :Write End (Boundary) Condition No. according to Table B-2.
3. Spans : Write Short Span L_x, Long Span L_y,and the aspect ratio L_y/L_x.
4. *Trial Depth* (D) :

It will be decided by serviceability criteria based on short span L_x.

(i)According to Note-2 of Cl. 23.1 of the Code, for a slab with mild steel reinforcement, L_x upto 3.5 metres and live load upto 3 kN/m²,

Allowable L/D ratio r_a = 35 for simply supported slab, and
= 40 for continuous slab.

Above values will be multiplied by 0.8 for slab with steel of grade Fe415.

Normally, this condition is satisfied incase of residential buildings.

(ii) If L_x > 3.5 metre or live load > 3 kN/m², allowable L/d ratio will be same as that for one-way slab. This may be obtained by assuming p_t = 0.2 % to 0.3 % and proceeding as in Step-4 of Sect.5.2.2 above.

Thus,for condition-(i)above for slabs with reinforcement of grade Fe415,

Required D = L/(0.8 x 35) = L/28 for simply supported slabs, and
= L/(0.8 x 40) = L/32 for continuous slabs.

Comments : It may be noted that for condition-(i)above,the ratio is L/D while it is L/d in general.

5. *Loads* : Calculate load in kN/m on 1 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 coefficents,α ,may be obtained from Table B-2 for given boundary condition No. and the aspect ratio. Calculate the value of $w_u L_x^2$ and multiply it with four values of α , as shown below to get the values of bending moments at four locations.

Span	Location	B.M.Coef.	Bending moment
Short	- Support	α'_x	$m'_{ux} = \alpha'_x w_u L_x^2$
	- Midspan	α_x	$m_{ux} = \alpha_x w_u L_x^2$
Long	- Support	α'_y	$m'_{uy} = \alpha'_y w_u L_x^2$
	- Midspan	α_y	$m_{uy} = \alpha_y w_u L_x^2$

From above four values, write down the value of maximum B.M.=$m_{u.max}$.

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 same as that of α_x for aspect ratio = 1.

7. *Check for Concrete Depth* :

In case of a two-way slab,effective depths for reinforcement in short span and long span differ by a bar diameter since long span steel is placed above the short span steel. Therefore, M_{ur} and requirement of area of steel is based on effective depth d_o of outer layer for short span steel and on effective depth d_i of inner layer for long span steel at midspan. As far as support section is concerned,the effectice depth is d_o only for both spans.

d_o = D - (clear cover + ϕ/2) say (D - 20) mm.
d_i = d_o - ϕ say (D-30) mm.

$M_{ur.max.outer} = R_{u.max}\ bd_o^2$ which shall be greater than $m_{u.max}$

$M_{ur.max.inner} = R_{u.max}\ bd_i^2$ which shall be greater than m_{uy}.

This condition is normally satisfied and it can safely be omitted in limit state design.

8. *Main Steel* :

Calculate area of steel required at four different locations by equation

$A_{st} = (0.5f_{ck}/f_y)[1 - \sqrt{1 - 4.6M_u/(f_{ck}bd^2)}]bd$ using appropriate m_u obtained in Step-6 and effective depth obtained from Step-7.

Assume bar diameter and calculate the required spacing as explained in Step-8 of one-way slab design.

Span	Location	m_u	d	Ast	ø - s
Short	Support	m'_{ux}	do	A'_{stx}	
	Midspan	m_{ux}	do	A_{stx}	
Long	Support	m'_{uy}	do	A'_{sty}	
	Midspan	m_{uy}	di	A_{sty}	

Reinforcement calculated above is provided only in 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 1/8 th 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 explained in Step- 10 of Sect.5.2.2.

In actual practice,the main reinforcement is provided over the entire width of slab to make provision for torsion reinforcement required at the corners as described in Step-10 below.

9. *Check for Serviceability* :

If $L_x <= 3.5$ m and $LL <= 3$ kN/m², serviceability check is normally satisfied. However ,if it is not satisfied, this check will be required to be applied for reinforcement at middle of short span i.e. A_{stx}.

Calculate $p_{tx} = 100\ A_{stx}/bd$. Check that this is less than or equal to assumed pt %. If not, then calculate the depth required for serviceability corresponding to p_{tx} required as explained in Step-9 of one-way slab design.

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 is 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° & 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 realtions.

At middle of short edge, $V_{u.max} = q_u L_x/3$ per unit width.
At middle of long edge, $V_{u.max} = q_u L_x [\beta/(2\beta + 1)]$ 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 slabs (Step-11b of Sect.5.2.2).

(c) Calculation of End Shears (Loads) on supporting Beams :

Long Edge : Trapezoidal load with ordinate q_u $Lx/2$.
Equivalent UD load for bending $w_{u.eqb}$ = $(q_u\ Lx/2)[1-1/(3\beta^2)]$
for shear w_{ueqs} = $(q_u\ Lx/2)[1-1/(2\beta)]$

Short Edge : Triangular load with central ordinate = q_u $Lx/2$.
Equivalent UD load for bending $w_{u.eqb}$ = q_u $Lx/3$
for shear w_{ueqs} = q_u $Lx/4$

For details, see Sect.3.4.2. In case of span 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).Refer Sect.3.4.1. & 25% at continuous end of two span continuous beam (i.e.shear coe.=.625)

12. *Check for Development Length*:
It will be applied in the same way as it is applied for one-way slabs.

5.2.4 Design of Stairs

Steps:

1. *Data* : Staircase room size,floor to floor height(H),live load .
2. *Functional details*:

Assume Rise R (150 mm to 200 mm)
Tread T (200 mm to 300 mm) $\sec\phi = \sqrt{R^2 + T^2}\ /\ T$

Number of risers = H/R
Number of risers for any flight <= 12
Based on these requirments 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 waist thickess of slab.
4. *Loads* : w = self weight of slab + weight of steps + FF + LL
= 25 D $\sec\phi$ + 25 R/2 + FF + LL (D and R in metres)
w_u = 1.5 w

Remaining design steps are same as those for one-way slab.(See Sect.5.2.1).

5.3 DESIGN OF BEAMS

5.3.1 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 beams may have same end conditions,spans,and/or loadings.Under such cicumstances, it is always advisable to categorise them and group them to facilitate design, and reduce the computational efforts.

5.3.2 Categorisation of Beams

The categorisation of beams may be done on the basis of design which depends on the following factors.
(1) End Condition (EC = 1,2,3,4,5), (2) Span,
(3) Load Type (UD load, Point load, triangular/trapezoidal load etc),
(4) Section Type (Rectangular, Flanged). (5) Load Magnitude.

Since categorisation 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 analysed and designed as a whole or as made up of independent beams with appropriate end conditions as explained in Sect.3.3.

(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, the design may be for a small single unit - a bungalow or a single residential unit. But the same may be required to be used for a big residential complex with large number of such units. A 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 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 regarding the end conditions of beams and simplifying assumptions and approximations very carefully. 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.

In general, the beams may now be categorised on the basis of end conditions as follows.

Category - I(a): Beams simply supported at both ends and carrying only Uniformly Distributed (UD) load.

- I(b): Beams simply supported at both ends but carrying UD load as well as point load/s.
- II : Beams simply supported at one end and continuous at the other, and carrying UD load only.
- III : Beams continuous at both ends and carrying UD load only.
- IV : Cantilever beams.
- V : 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.

5.3.3 Procedure for Design of Beam

Steps:

1. *Beam Mark* : Specify beam mark (e.g. B1, B2, etc) as per selected scheme of marking.
2. *End Condition No.* : Write down End condition No. as described in Step-2 of Sect.5.2.2.
3. *Span* (L) : In general, it may be taken as centre to centre distance between the supports, on safer side. For exact value, refer to Sect.5.2.2.

4. *Section and Materials*: Assume grades of concrete and steel to be used and
   (a) breadth b => Breadth of Wall. Common values - 150,225 (230),300 mm.
   (b) Depth D = L/10 to L/15. Common values - 300,375,450,525,600,675,750,825,900 mm (module 75 mm/3") 300 to 900 in multiples of 50 mm for module 50 mm.
   (c) Effective Cover (d') for tension steel in mm :

Assume for -		Fe250	Fe415/Fe500	
Rectangular beams	d' =	40-50	35-50	mm
Flanged beams	d' =	60-80	40-60	mm

   (d) Effective Cover(d'_c) for compression steel : Assume 35 mm to 50 mm.
   (e) Effective depth d =(D - d') in mm.
   (f) Depth of flange D_f = Depth of slab.
   (g) Breadth of support b_s

5. *Loads* :
   (a) Uniformly Distributed Load : (w) in kN/m
      (i) Self : $w_d = 25bD_r$ (b and D_r in metres). D_r = Depth of rib = $D - D_f$.
      (ii) Masonary Wall : w_w = 20BH (B & H in metres),B = Breadth,H = Height.
      R.C.Wall/Grill: w_w = 25tH (t & H in metres),t = thickness. Make appropriate deduction for openings in grills.
      (iii) Slab from Left Side: w_{s1}. (Left or Right side will be decided as we look the building plan from below and from right side of the plan.
      Give Data: Slab mark (S1, S2 etc.), Slab Depth (D_f in mm), Short span of Slab in metres, Load distribution on slab-(rectangular,trapezoidal or triangular), Slab End Shear as calculated in Step-11(c) of slab design = w_{s1}.
      If slab end shear is not calculated in slab design, the same shall be calculated here according to Step-11(c) of Sect.5.2.2 and Sect. 5.2.3 or as explained in Sect.3.4.1 and 3.4.2.
      (iv) Slab on Other Side : w_{s2}. It is obtained in the same way as in step (iii) above.
      (v) Total Working Load : w in kN/m.
      $w = w_d + w_w + (w_{s1} + w_{s2})$ for calculation of B.M., and
      (vi) Design (Ultimate) Load : w_u = 1.5w in kN/m.
   (b) Point Loads : W in kN.
      Give total No. of loads = Number of secondary beams supported.
      Give Data for each beam as follows : Secondary Beam Mark B_i, Beam Shear W_i in kN, Distance(x_i) of W_i from nearest support.

6.(a) *Design Moment* : ($M_{u.max}$)
   Design moments are determined at support (i.e. at continuous end) and at midspan by using either the standard bending moment coefficients given in Appendix B-1 or by using any other method of structural analysis explained in Chapter-3.

(b) *Maximum Ultimate Moment of Resistance* : Calculate $M_{ur.max} = R_{u.max}\, bd^2$. Values of $R_{u.max}$ are taken from Table 4.1-1 corresponding to the grade of concrete and grade of steel used.

(c) *Section type* : Decide from availability and spanning of slab whether the midspan section can act as flanged section or not. If it can act as flanged section, decide by comparing the design moment $M_{u.max}$ at midspan with the value of $M_{ur.max}$ whether at all it is necessary to design it as a flanged section (See Sect.3.3d). If it is to be designed as a flanged section, proceed to Part-III of Step-7 else decide whether the rectangular section will be required to be designed as singly reinforced or doubly reinforced section.

If $M_{u.max} <= M_{ur.max}$, the section will be designed as singly reinforced in which case proceed to Part-I of Step 7 below.
If $M_{u.max} > M_{ur.max}$, the section will be required to be designed as doubly reinforced, in which case proceed to Part-II of Step-7 below.

7. *Main Steel* : Part I : **Singly Reinforced Rectangular Section**

Required $A_{st} = (0.5f_{ck}/f_y)[1 - \sqrt{1 - 4.6M_{u.max}/(f_{ck}\ bd^2)}]\ bd$

Alternatively, if lever arm approach is used
$A_{st} = M_{u.max}/(0.87f_y\ j_u d)$ wherein the values of lever arm factor ju may be obtained from Table given in Note of Sect.4.1-2(d).
Proceed to Part-IV

7. *Main Steel* : Part II : **Doubly Reinforced Rectangular Section**

(a)Tension steel : $M_{u1} = M_{ur.max}$, $M_{u2} = M_{u.max} - M_{u1}$
(i) $A_{st1} = M_{ur.max}/[0.87f_y(d - 0.42\ x_{u.max})]$ or alternatively,
$A_{st1} = p_{t.max}\ b\ d\ /100$, $p_{t.max}$ is given as %.
For values of $x_{u.max}$ and $p_{t.max}$ see Table 4.1-1.
(ii) $A_{st2} = M_{u2}/[0.87f_y(d - d'_c)]$
(iii) $A_{st} = A_{st1} + A_{st2}$

(b)Compression steel .
$A_{sc} = M_{u2}/[(f_{sc} - f_{cc})\ (d - d'c)]$ or alternatively,
$A_{sc} = A_{st2}\ [0.87f_y/(f_{sc} - f_{cc})]$
where, $f_{cc} = 0.45f_{ck}$ and f_{sc} may be obtained from Table 2.10b; or the value of the factor $0.87f_y/(f_{sc}-f_{cc})$ may be directly taken from Table.4.1-3.
Proceed to Part-IV .

7. *Main Steel* : Part III : Flanged Section

(a)Width of Flange (b_f) :
For T-beam : $b_f = L_o/6 + b_w + 6D_f$.
For L-beam : $b_f = L_o/12 + b_w + 3D_f$.
For details and value of Lo,see Sect.4.1.5(a).

(b)Position of Neutral axis (i.e. whether $x_u<D_f$ or $x_u>D_f$)
Calculate M_{ur1} for $x_u = D_f$ from relation:
$M_{ur1} = 0.36\ f_{ck}\ b_f\ D_f\ (d - 0.42\ D_f)$
If $M_u <= M_{ur1}$, $x_u <= D_f$. This case is very common.
If $M_u > M_{ur1}$, $x_u > D_f$. This case is very rare.

(c)Area of Tension Steel:
(i) If $x_u <= D_f$, $A_{st} = (0.5\ f_{ck}/f_y)[1 - \sqrt{1 - 4.6M_{u.max}/(f_{ck}\ b_f\ d^2)}].b_f\ d$
(ii) If $x_u > D_f$, calculate A_{st} as explained in Sect.4.1-5(c).

7. *Main Steel* : Part IV : Detailing of Bars

(d)Number(N)-Diameter(Ø) Combination : Select N-Ø combination from Table.D-6(a) such that (A_{st})provided >= (A_{st})required.
State the Number of Rows(NR) and Number of Bars(NB) in each row.
(i) Check for width : For Ø <= 25 mm,
Required b = NBxØ + (NB + 1)x25mm <= assumed b.
If this check is not obtained, increase the bar diameter and reduce the number of bars (NB) in a row.

(ii)Check for effective cover (d') : Required effective cover

$d' = 25 + \phi/2$ for 1 row.

$= 25 + \phi + (\phi/2$ or $15/2$ mm whichever is greater) for 2 rows.

$= 25 + 1.5\,\phi + (\phi$ or 15 mm whichever is greater) for 3 rows.

$<=$ asssumed cover d'.

If this check is not obtained, increase the bar diameter and reduce the number and hence number of rows.

(e)Detailing : Decide

(i)No. of bottom straight bars at midspan.

(ii)No. of bentup bars,

(iii)No. and diameter of straight bars at top at midspan as anchor bars. Normally, 2 of 8,or 2 of 10, or 2 of 12 mm diameter.

(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.

8. *Shear Design* :

(a)Calculation of total unifirmly distributed load on beam for $V_{u.max}$.

(i)In case of beams carrying uniformly distributed loads : w_u is same as that calculated earlier for bending (for beams supporting one-way slab) .Go to Step-(b) below.

(ii)In case of beams carrying triangular/trapezoidal loads (supporting two-way slab), calculate equivalent uniformly distributed load for shear replacing trianguular/trapezoidal load.

For trapezoidal load, $w_{u.eqs} = (q_u\ L_x/2)[1-1/(2\beta)]$,

For triangular load on short edge beam carrying

two-way slab, $w_{u.eqs} = q_u\ L_x/4$, : where

one-way slab, $w_{u.eqs} = q_u\ L_x/8$. : q_u = load on slab.

Total w_u = self + wall + $w_{u.eqs}$ from slabs on both sides.

(b)Calculation of maximum shear $V_{u.max}$ at centre of support:

(i)In case of beams either simply supported or continuous at both ends:

$V_{u.max} = 0.50\ w_u\ L$ at both ends.

(ii)In case of beams simply supported at one end & continuous at the other.

$V_{u.max} = 0.60\ w_u\ L$ at continuous end.

$V_{u.max} = 0.45\ w_u\ L$ at simply supported end.

where, w_u = intensity of loading.

In case of central point load, substitute W_u instead of $w_u\ L$ in (b) above. For other loading,calculate shear at supports from principles of mechanics.

(c)Checking adequacy of concrete section :

Compute maximum shear resistance of concrete in diagonal compression.

$V_{uc.max} = \tau_{uc.max}\ bd$

where $\tau_{uc.max}$ = shear strength of conorete in diagonal compression, $= 2.5\ N/mm^2$ & $2.8\ N/mm^2$ for M15 & M20 concrete [respectively.

Thus, if $V_{uc.max} >= V_{uD}$, the section is adequate.

If $V_{uc.max} < V_{uD}$, revise the section by increasing b or D.

(Note: This situation hardly ever arises in case of beams with UD loads.)

(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-4.2 corresponding to percentage (p_t) of tension steel at support.

and $p_t = 100\,A_{st1}/\,bd$, where A_{st1} = Area of steel at support.

For determining the area of steel A_{st1} at support, decision is required to be taken in the beginning regarding number of bottom bars(midspan steel) to be bent up (may be for resisting either negative moment or shear at support), or number of bars to be curtailed if any.

The normal practice of continuation of bars to supports is as follows.

No.of bars at midspan	2	3	3	4	4	5	5	6	6	6
No.of bars continued to support	2	3	2	4	2	5	3	6	4	3

(ii) Calculate the Shear Resistance of Minimum Stirrups ($V_{usv.min}$) :

$V_{usv.min} = 0.348\ bd = 0.35\ BD$

(iii) Calculate Shear Resistance of Section with minimum Stirrups ($V_{ur.min}$) :

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

(iv) Determine whether shear reinforcement is required to be designed or minimum stirrups are sufficient.

If $V_{ur.min} >= V_{u.max}$,minimum stirrups are sufficient.Go to Step-(vi).

If $V_{ur.min} < V_{u.max}$, then only calculate design shear (V_{uD}) at critical section for comparing $V_{ur.min}$ with V_{uD}.

(v) Calculation of Design Shear at critical section:

$V_{uD} = V_{u.max} - w_u\ (b_s/2 + d)$ for supports offering compressive reaction, and beam carrying UDL. b_s = width of support.

$V_{uD} = V_{u.max} - b_s/2$ for supports offering tensile reaction.

If $V_{ur.min} >= V_{uD}$,minimum stirrups are sufficient.Proceed to Step-vi.

If $V_{ur.min} < V_{uD}$,minimum stirrups are not sufficient. However, proceed to step-(e) only after determining minimum stirrups in step-(vi).

(vi) Compute the spacing (s) of minimum stirrups.

Assume diameter of stirrups, usually

6 mm for grade Fe250 (A_{sv} = 56 mm² for 2 legged stirrups),or

8 mm for grade Fe415 (A_{sv} = 100 mm² for 2 legged stirrups).

$s = A_{sv}\ f_y/(0.4b) <$ (450 mm or 0.75 d whichever is less).

where, A_{sv} = total area of all legs of stirrups at the section.

If minimum stirrups are sufficient, proceed to Step-(f) else go to next step-(e).

(e) Design of Shear Reinforcement :

(i) Determine the shear required to be resisted by shear reinforcement(V_{us})

$V_{us} = V_{uD} - V_{uc}$

(ii) Determine the shear resisted by bent-up bars, if used as shear reinforcement. If not, take $V_{usb} = 0$ and proceed to Step-(iii) below.

$V_{usb} = 0.87 f_y\ A_{sb}\ \sin\alpha$ but not greater than $0.5 V_{us}$.

Normally, $\alpha = 45°$. Bars shall be bent-up at a distance not exceeding 2d from the centre of support.(Common practice is to bent the bars at a distance 1.25d or 1.5d from the face of support). Effective region of bent-up bar is only this distance up to the point of bend.

(iii)Determine the balance shear to be resisted by stirrups (V_{usv}) :

$V_{usv} = V_{us} - V_{usb}$, but not less than 0.5 V_{us}.

If now, $V_{usv} <= V_{usv.min}$,minimum stirrups are still sufficient in which case, proceed to step (f).

If $V_{usv} > V_{usv.min}$, continue to next step to design the stirrups.

(iv)Design of Stirrups :

Assume diameter of bar and grade of steel to be used for stirrups.
Normally, 2-legged 6 mm of grade Fe250 (A_{sv} = 56 mm²), or
8 mm of grade Fe415 (A_{sv} = 100 mm²).

Spacing of stirrups is given by

$s = 0.87 f_y A_{sv} d/V_{usv}$ < (450 mm or 0.75 d whichever is less)

Round off the value on lower side to multiple of 10mm or 25 mm as desired.

(f)Determination of Zones of Shear Reinforcement : See Fig. 5.1.

(i)Determine the Length (L_{s1})of Shear zone of designed shear reinforcement.

This zone exists only when shear reinforcement is required to be designed, i.e. when minimum stirrups are not sufficient, or $V_{ur.min} < V_{uD}$.

When minimum stirrups are sufficient $L_{s1} = 0$,proceed to step-(ii).

$L_{s1} = (V_{u.max} - V_{ur.min})/w_u$

for uniformly distributed load.

For other loading types, L_{s1} is given by the distance from the support to the point where,shear force $V_u = V_{ur.min}$

(ii)Determine the Zone of Nominal Stirrups (L_{s3}) :

It is that region in which shear force $V_u <= 0.5V_{uc}$ and minimum stirrups are not necessary. It is given by

$L_{s3} = 0.5\ V_{uc}/w_u$

for uniformly distributed load.

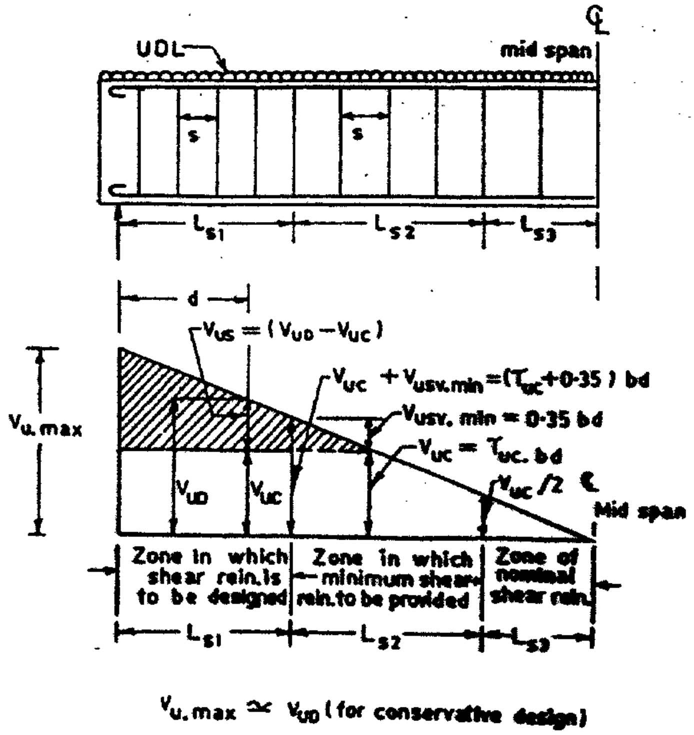


Fig. 5.1 Shear Reinforcement along the beam

In other loading cases, L_{s3} is the distance from the point of zero shear to a point where shear force $V_u = 0.5\ V_{uc}$.

(iii)Determine the Zone of Minimum Stirrups (L_{s2}) :

$L_{s2} = L/2 - L_{s1} - L_{s3}$.

For details, see Fig.5.1.

9. *Check for Bond or Check for Development Length :*

(a)For Negative Moment Steel or Support Steel :

Required development length $L_d = k\ Ø$, k = 57 for M15 - Fe415.
For other grades, refer to Sect.4.3.3.

Available development length is actual length of bar from critical section which is at centre of intermediate support of a continuous beam, and at face of support of a cantilever.

In case of cantilever, available length is nearly equal to span length.

In case of a continuous beam, available length is

x_o + (12 ø or d or clear span/16 whichever is greater) where

x_o = length of negative moment region i.e. the distance of point of contraflexure from the centre of support.

In practice, usually bars are provided for a length equal to L/4 from the centre of support which then becomes the available length.

It will be checked that available length of bar is greater than the required development length.

(b) *For positive moment steel i.e. bottom steel at point of zero bending moment*

Required development length = L_d = k ø , (57 ø for M15- Fe415).

Available L_d at discontinuous end or at point of contraflexure

= $1.3M_1/V+L_{ex}$ at support offering compressive reaction.

= $M_1/V+L_{ex}$ at support offering tensile reaction, or at point of contraflexure,

where Lex = $L_d/3-b_s/2$ or actual length available beyond centre of support whichever is greater but not exceeding effective depth d, at discontinous beam end.

L_{ex} = d or 12 ø whichever is greater at point of contraflexure.

M_1 = Moment of resistance of beam at the section,

= $(A_{st1}/A_{st}).M_{max}$ where A_{st} is area of steel at midspan.

= $(N_1/N_{.max}).M_{max}$ when all bars of same diameter and where

N_1 and N_{max} are number of bars at support and at mid-span respectively.

Equating required L_d and available L_d, we have,

$L_d = 1.3\ M_1/V + L_{ex}$.

∴ Required $M_1 = V(L_d-L_{ex})/1.3$ at support offering compressive reaction.

(c) *Required Minimum Steel at Support for End-anchorage :*

$A_{st} = A_{st.max}/3$ at discontinuous end, and

$= A_{st.max}/4$ at continuous end.

This steel will be required to be continued through a minimum distance of $L_d/3$ inside the inner face of support.

10. *Check for Serviceability:*

Allowable L/d ratio (r_a) = α_1x basic L/d ratio (r_b).

Basic L/d ratio (r_b) = 7 for cantilever,

= 20 for simply supported beam,

= 26 for continuous or fixed beam.

α_1 is a modification factor for p_t and is given by Fig.4.4.1.

Calculate p_t = 100 A_{st}/(bd) at midspan of a beam supported at both ends and at support in case of a cantilever.(b=b_f for flanged sections). Obtain α_1 corresponding to this value of p_t from Fig. 4.4.1 and hence allowable (L/d) = α_1 x Basic (L/d) =α/1 α_b .

Alternatively, in case of a rectangular beam, since α_1 = 1 for p_t=.87 % for steel of grade Fe415,allowable L/d ratio > basic L/d ratio for p_t<.87%, because the modification factor increases with decrease in p_t. Therefore,in case of a rectangular beam having p_t < 0.87 %, it is sufficient to check that actual L/d ratio is less than basic L/d ratio.

Similarly, in case of a flanged beam,

Allowable L/d ratio = $\alpha_1 . \alpha_3$ xBasic L/d ratio, where $\alpha 3$ is a modification factor accounting for effect of flange. For $b_w/b_f < 0.3$, $\alpha_3 = 0.8$. Therefore, for allowable L/d ratio to be equal to Basic L/d ratio, $\alpha_1 x.8 = 1$ or $\alpha_1 = 1.25$ which corresponds to $p_t = 0.4$ % for M15 - Fe415 combination.

∴ In case of flanged beams having $b_w/b_f < 0.3$, and having p_t based on $A_{st}/(b_f\ d) < 0.4$ %, it is sufficient to check that actual L/d ratio is less than basic L/d ratio. Normally, in practically all the cases of beams these conditions are satisfied, and therefore, the check for serviceability is invariably obtained if depth of beam is chosen greater than L/15, and this check can safely be omitted.

5.4 DESIGN OF COLUMNS

5.4.1 Introduction

The design of column necessitates determination of loads transferred from beam 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 enables one to assess the loads on columns more accurately and thereby the design of column becomes realistic and economical.

However, in practice, manytimes situations arise which require the design of columns and footings to be given to the clients prior to the design of slabs and beams. In such situations, loads on columns and footings are required to be assessed using judgement based on past experience or 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.3.4.3(b). 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. However, authors are of the opinion that if there is no exigency of time, slabs and beams should be designed first in order to know the actual design loads on columns. Only in case of pressing need of giving the design of columns and footings prior to the design of beams, the approximate method of assessing the load on column based on the floor area shared by the columns or, approximate beam shears, be used.

5.4.2 Design Procedure

Design of columns involves following steps.

(1) *Categorisation of Columns*:
- (a) Internal Columns or Axially Loaded Columns.
- (b) Side Columns or Columns subjected to Axial Load and Uniaxial Bending.
- (c) Corner Columns or Columns subjected to Axial Load and Biaxial Bending.

(2) *Computation of Floor Loads*:
- (A) Exact Method.
- (B) Approximate Method.
  - (I) Assessment of Unit Loads of (a) slab, (wall), (c) column.
  - (II) Assessment of Total Load on Column in Each Storey.
    - (a) Marking of Column Load Transfer Areas.
    - (b) Calculation of Loads at Each Floor Level.

(3) *Calculation of Moments in Columns*:
- (a) Exact Method.
- (b) Approximate Method.

(4) *Determination of Effective Length and Type of Column - Short or Long*

(5) *Grouping of Columns:*

(6) *Design of Column Section:*

(A) Approximate Equivalent Axial Load Method.
  I - Preliminary Design.
    (a) Allowance for Moment in Column.
    (b) Allowance for Slenderness of Column.
    (c) Calculation of Total Equivalent Axial Design Load.
    (d) Section Design.
  II- Check for Moment in Column.

(B) Exact Theoretical method.
  I - Axially Loaded Short Columns.
  II - Short Columns under Combined Axial Load and Uniaxial Bending.
  III - Short Columns under Combined Axial Load and Biaxial Bending.
  IV - Slender Columns.

5.4.3 Categorisation of Columns

Catagorisation of columns is extremely helpful as it decides the approach to column design at a latter stage 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) *Internal Columns or Axially Loaded Columns :*

Internal columns carrying beams either in all four directions or only in opposite directions are predominently subjected to axial loads 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, these columns can be termed as Axially Loaded columns. Therefore, they require practically very little or no allowance in axial load to account for the effect of bending moments due to full or partial fixity between the beam and the column.

(II) *Side Columns or Columns subjected to Axial Load 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 approxiately 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 arrieved is called Equivalent axial load for the pupose of design of column section.

(III) *Corner Columns or Columns subjected to Axial Load 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.

5.4.4 Computation of Floor 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

$P_{u.floor} = V1 + V2 + V3 + V4 + P_a + P_{self}$ where V1,V2,V3,V4 are the end shears of beams meeting at the column at the floor under consideration from all the four directions 1,2,3,4.

P_a = axial load coming from above.

P_{self} = self weight of the column at the floor under consideration. For details see Sect.3.4.3.

(B) Approximate Method : This method may be used when the column design is required to be done prior to design of slabs and beams.

(I) *Assessment of Unit Loads*:

(a) Assessment of Unit Slab Load : This may be assessed roughly by assuming the depth of floor/roof slab (D) on the basis of following guide lines if slab depth is not designed earlier.

Type of slab		Approximate Depth of slab in mm.	
One-way	Two-way	Residential building LL = 2 kN/m²	Public/office building LL = 4 kN/m²
upto 2.5 m	upto 3 m	100 mm	120 mm
2.5m to 3 m	3 m to 4 m	120 mm	140 mm
3 m to 4 m	4 m to 5 m	140 mm	150 mm

Weight of slab = 25D where, D is in metre.

Weight due to floor finish = FF = 1 kN/m²

Live Load = LL to be taken from Table 2.3 depending on the use of floors of the building.

Ultimate load = $q_u = 1.5(25D + FF + LL)$ kN/m²

(b) Assessment of Unit Wall Load : This may done from available data regarding the material, thickness, and height of wall. The unit loads shall be multiplied by 1.5 to get the wall load rate at ultimate state. Refer to App. A-1.

(c) Calculation of Self Weight of Column : This may be calculated for the floor to floor height for different standard sections. This will also be a factored value at ultimate state.

(II) *Assessment of Total Load on COlumn in Each Storey*:

(a) Marking the Load Transfer Area : The load transfer area for each column is the area contained between the intersecting lines drawn perpendicular from the mid-points of the lines joining the adjacent columns (See Fig.3.4.3b). This area pertaining to each column will, in general, consist of rectangles. Compute the load transfer area for each column ($A_{col.load}$).

(b) Calculation of Loads at each Floor Level :

(i) Load from slab :

Load transferred from slab to column = $P_{us} = q_u (A_{col.load})$

Apply continuity factors to above values to account for the effect of continuity of beams as given below.

Corner columns 0.8; Side columns 1.1; and internal columns 1.2.

(ii) Load from Walls : Determine the total lengh of walls within and on the boundary of the load transfer area and compute the wall loads. The load of wall on the common boundary of two column load areas will be equally divided to the respective columns. For external walls, where the width of beam is equal to the wall thickness, the wall load may be taken for full floor to floor height ignoring the depth of the beam.In case of internal walls,if they are thiner than the width of the beam,wall load may be calculated for the height upto the soffit of the beam, and the self weight of the beam be added to it. The depth of beam is assumed depending on the span of the beam, normally Span/10 to span/12 .

Total wall load transferred to column at each floor level = $P_{uw} = \Sigma w_{uw} L$
where, w_{uw} L = sum of all wall loads multiplied by corresponding lengths of walls within the load transfer area ($A_{col.load}$) including walls at the outer edges of this area.

Apply continuity factors to above values to account for the effect of continuity of beams.

Corner columns 0.8 ; Side columns 1.1 ; Internal Columns 1.2 .

(iii) Calculation of Total Design Load transferred to column at any floor :
$P_{u.floor} = P_{us} + P_{uw} + P_a + P_{self}$ (assumed)
where, P_a = load on column from above, and
P_{self} = self weight of column. This is obtained assuming section approximately. (See Table 2.2 or Table A-1).

At plinth level, there is no load of the slab, and wall load also includes the weight of wall between top of plinth beam to underside of first floor level.

Above procedure of calculation of column loads does not work when there are number of secondary beams. In such cases, approximate loads are required to be calculated on beams first and column loads are obtained from beam shears. (For details, see remarks in Sect.3.4.3.)

5.4.5 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.5.4.7 directly. The moments in column are obtained directly and exactly if the entire structural frame is analysed 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 using Substitute Frame Method - III (See Sect.3.2.2c) as detailed below.

The moment in the column can be calculated using the equation,

$M_{col} = (K_c/\Sigma K)\ M_e$

where K_c = stiffness of column under consideration = I_c/L_c

ΣK = sum of the stiffnesses of members meeting at the joint. Stiffnesses of beams shall be reduced to half to account for effect of members beyond the adjacent spans ignored.

M_e = unbalanced fixed end moment at the joint.

= $w_u L^2/12$, if a single beam is connected to column on one side.

= $(w_{u1}L_1^2/12 - w_{u2} L2^2/12)$, 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 L1,L2 are lengths of beams on two sides.

The calculated moment in column shall not be less then $M_{u.min} = P_u\ e_{min}$, where , e_{min} is the minimum eccentricity given by Eq.4.5-2.

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.

5.4.6 Determination of Effective Length of Column and Type of Column (Short or Long)

(a) *Determination of Effective Length of Column* :When there are longitudinal and cross walls in both directions, the frame is assumed to be a non-sway frame. In such cases, the effective length lies between 0.65L to L,wherein L represents the unsupported length of the column. The effective length of the column in a building frame may be taken as follows :

(i) For any intermediate storey :

L_{eff} = L = Unsupported Length
= Floor to floor height - Depth of shallower floor beam (300 mm ,or more depending on the span).

(ii) For top storey :

L_{eff} = 1.2L where L is unsupported length as defined above.

(iii) For columns in bottom storey :

-When plinth beams are not provided :

L_{eff} = L = Distance between bottom of footing to the underside of the shallower beam at first floor level.

-When plinth beams are provided :

L_{eff} = L = Distance between top of plinth beam to the underside of the shallower beam at first floor level.

It may be noted that plinth beams are normally provided just below ground level and not at the ground floor 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 below 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 bottom of footing.

(b) *Determination of Type of Column (Short or Long)* :

If $L_{eff}/b > 12$, the column is slender or long. 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 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 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 150 mm, 225 mm (or 230 mm), and 300 mm. (Width 200 mm also may not be uncommon in future). Column having width equal to 225 mm acts as a short column when its unsupported length does not exceed 2700mm i.e. when the floor to floor height does not exceed 3 metres assuming minimum depth of shallower beam to be 300mm.The columns having width of 150mm are likely to be slender for floor to floor height greater then 2.1 m (depth of beam has been assumed equal to 300 mm) provided they are not braced at the lintel level.

5.4.7 Grouping of Columns

Once the load on each column and effective lengths are determined, the columns which have total loads on them not varying by more than 10 to 20 % and which have their effective lengths equal may be grouped together. The 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.

5.4.8 Design of Columns Section

The design of column 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 is required.

(A) Approximate Equivalent Axial Load Method

In this approach, total equivalent axial load is obtained by adding to calculated axial load, the allowances for moments and slenderness, if any. Preliminary section is designed for this total equivalent axial load using the procedure for design of axially loaded columns explained in Sect.5.4.9(I). The section so obtained is later checked by exact method for actual axial load and bending moment and actual slenderness as explained in Sect. 5.4.9.

I - Priliminary 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.3.4.3 to account for the effect of bending moment due to partial fixity/full fixity between beams and column. This allowance is to be made on the load coming from the floor 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.

% allowance = $(1/C_r - 1) \times 100$ where, $C_r = 1.25 - L_{eff}/(48b)$

This allowance approximately works out to 14 % of axial load on column having width b equal to 150 mm and floor to floor height 3000 mm (with minimum beam depth 300 mm). The 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 ($P_{u.eq}$):

$P_{u.eq}$ = actual axial load on column obtained in Sect.5.4.9 (I) + allowance for fixity + allowance for slenderness.

(d) Section Design :

The section of the column is obtained according to the procedure explained for design of axially loaded columns in Sect.5.4.9 (I).

II - Check for moment in Column :

Calculate the moment carrying capacity of the designed section using either Table E-2 or the interaction charts. 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 $M_{ur} >= M_u$ 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.5.4.9(III). For slender columns, check the safety of column as detailed in Sect.5.4.9 (IV).

5.4.9 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 for the same.

(I) Axially Loaded Short Columns :

In practice, this is done by use of available readymade design tables. In absence of such design tables, the design is done by use of equations given in Sect.4.5.2.

(a) *Practical Design by Use of Tables :*

The brief design tables suitable for G+3 buildings are given in Tables-E-1 and E-2. The appropriate depth and the number-diameter combination of bars is selected from these tables. The ties are designed using Table E-5.

Students or beginners can themselves prepare such tables for their design using the following formulae.

Load carried by concrete $P_{uc} = 0.4f_{ck}$ b D. Obtain P_{uc} for standard sizes.

Load carried by steel $P_{us} = (0.67f_y - 0.4f_{ck})A_{sc}$. Obtain values of P_{us} for different standard Number - Diameter Combinations of bars.

Load carried by the column $P_u = \lambda\ (P_{uc} + P_{us})$ where values of λ depend on the width of column assumed and can be obtained from Sect.4.5.2a.For b=150mm, λ=0.7 and for b=225 (or 230)mm, λ =0.9. The table would be of the following form.

Section b x D mm mm	P_{uc}	P_{us}	4-12	6-12	8-12	4-16	6-16	8-16
			No. Diameter Combinations					
			Values of P_{us} in kN					
225x300	Values							
225x375	of P_{uc}		Values of P_u in kN					
225x450	in kN							

Such table helps in appropriate choice of depth and number diameter combination from the various possible solutions for required P_u. See Tables E-1,E-2.

(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 0.8% 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 rearranged and revised version of Eq.4.5.4a putting $A_c = A_g - A_{sc}$ and $A_{sc} = pA_g$ as given below.

$$\text{Required } A_g = (P_u/\lambda)/[0.4f_{ck} + (0.67f_y - 0.4f_{ck})p].$$

(ii) From assumed width b, obtain depth $D = A_g/b$. Use practical dimension such as 300,375,450,525,600,675,750 mm etc.

(iii) Calculate now A_{sc} for selected values of b and D using the formula

$$A_{sc} = (P_u/\lambda - 0.4f_{ck}\ bD)/(0.67f_y - 0.4f_{ck}).$$

(iv) Select appropriate Number - Diameter combination of bars from practical combinations, namely, 4-12, 6-12, 8-12, 4-16, 6-16, 8-16 and so on.

(v) Assume diameter of lateral ties (ϕtr not less than 5 mm or 1/4th the diameter (ϕ) of main bar whichever is greater. Normally, 6mm 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, 48ϕtr, 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 column, if not calculated earlier, as explained in Sect.5.4.5. 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. It may be noted that the second arrangement requires larger 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 :

Assuming effective cover (minimum 46 mm to maximum 53 mm), compute d'/D, $P_u/(f_{ck}\,bD)$, and $M_u/(f_{ck}\,bD^2)$. Referring to chart of the interaction diagrams corresponding to the value of d'/D and the assumed arrangement of bars and grade of steel, obtain the value of p/f_{ck} corresponding to the calculated values of $P_u/(f_{ck}\,bD)$ and $M_u/(f_{ck}\,bD^2)$. If the value of d'/D lies between the standard values for which the charts have been drawn, then refer to these two charts and obtain the two values of p/f_{ck}. The desired value can be obtained by linear interpolation between the above two values of p/f_{ck}. (If the value of d'/D lies outside the standard chart values of d'/D , obtain the required value of p/f_{ck} by extrapolating the two values of p/f_{ck} obtained from charts of two adjacent values of d'/D). Note that p represents percentage of total steel A_{sc}. Hence, the required A_{sc} can be worked out using relation $A_{sc} = (p/f_{ck}).f_{ck}\ bD/100$. Select appropriate number diameter combination corresponding to the required value of A_{sc}.

III Short Columns Subjected to Axial Compression and Bi-axial 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 the periphery as this arrangement is ideal for biaxial bending.

(ii) For the trial section, obtain $M_{ux1} = M_{uxr}$ and $M_{uy1} = M_{uyr}$, the moment carrying capacities of the section for bending about x - axis and y - axis respectively under given axial load P_u by proceeding as in Sect.5.4.9 (II).

(iii) Calculate P_{uz} using Eq.4.5-3, and hence ratio P_u/P_{uz}. From this ratio, obtain α_n as given in Sect.4.5.4.

(iv) Verify using Eq.4.5-14, whether the section is safe or not. If the section is unsafe,revise the calculations from Step-(ii) above assuming a bigger section or increasing the reinforcement. 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.Continue with the trials untill the section is safe and economical.

IV Slender Columns

(i)Calculate additional moment due to slenderness using Eq.4.5-15. Obtain P_{uz} using Eq.4.5-3 and P_{ub} by using Eq.4.5-12.

(ii)Calculate initial moments using Eq.4.5-17.

(iii)Obtain total moment using Eq.4.5-18. This is now the design moment for the column accompanied by givem P_u.

(iv)Check the safety of column for combined effect of P_u and the above total moment M_{ut} using the procedure explained in Sect.5.4.9 (II) or Sect.5.4.9 (III) above as the case may be.

(v)Revise the section if unsafe or uneconomical.

5.5 ISOLATED SLOPED FOOTING FOR AXIALLY LOADED COLUMNS (See Fig. 5.2)

The design of an isolated column footing is done in following 5 stages.

(I) *Determination of Size of footing:*

Size of footing required is decided by the bearing area required to carry the load acting on the base of footing and the bearing capacity of the soil. As the bearing capacity of the soil corresponds to working load on the foundation, the required area of the footing is determined from the working column load ($P_w = P_u/1.5$) on footing incremented by 10 % to account for the self weight of the footing.

(a)Area of footing required, $A_f = 1.1\ (P_u/1.5)/\sigma_{bs}$ .....(5.5-1)

where, σ_{bs} = safe bearing capacity of the soil.

(b)The dimensions of the footing are decided by the criterion that projections of the footing beyond column faces in two orthogonal directions shall be equal. These is achieved by using the following formula.

Required Length of footing $L_f = (D-b)/2 + \sqrt{[(D-b)/2]^2 + A_f}$ .....(5.5-2a)

where b = width of column, D = Depth of column at the base.

Required breadth of footing $B_f = A_f/L_f$ .....(5.5-2b)

Provide actual dimensions in multiples of 50, 100 or 250 mm as desired.

(II) *Determination of Depth of footing :*

Depth of footing is decided from requirements of two-way bending due to upwards soil reaction, and checked for two-way shear (or punching shear) and one-way shear (or beam shear). The footing is normally provided with sloping top i.e. of vaying depth with depth ($D_{f.min}$) at the edge of footing between 150 mm to 300 mm.

(a)Calculate net upward pressure causing bending in footing as follows :

$w_u = P_u/(L_f\ B_f)$ .....(5.5-3)

(b)Calculate projections of footing beyond the face of column in the direction parallel to – depth of column $x = (L_f - D)/2$ .....(5.5-4a)

– width of column $y = (B_f - b)/2$ .....(5.5-4b)

(c)Calculate bending moments about the faces of column due to upward soil reaction acting over the projected areas.

$M_{ux} = w_u\ B_f\ x^2/2$ and $M_{uy} = w_u\ L_f\ y^2/2$ .....(5.5-5)

(d)Required depth of footing depends upon the widths (b' and D') at top of the footing. To facilitate seating of formwork for concreting of column, an off-set of 50 mm is provided. ∴ b' = (b + 100) mm and D' = (D + 100) mm.

Required total depth of footing :

$D_{fx} = \sqrt{M_{ux}/(R_{u.max}\ b')} + d'_x$; $D_{fy} = \sqrt{M_{uy}/(R_{u.max}\ D')} + d'_y$ ....(5.5-6)

where, Effective cover $d'_x = 50 + \phi/2$ and $d'_y = 50 + 1\frac{1}{2}\phi$ mm

ϕ = diameter of the bar. Bottom steel provided with clear cover = 50mm.

D_f = greater of D_{fx} & D_{fy} rounded off on higher side in multiple of 50mm.

Effective depth provided, $d_x = D_f - 50 - \phi/2$, and $d_y = d_x - \phi$

(III) *Check for Two-way Shear:* Critical section is a peripheral section at a distance of $d_y/2$ from the face of the column.

(a) Calculation of Design Shear: V_{uD}

Dimensions of the peripheral critical section :

$L_1 = (D + d_y)$; $B_1 = (b + d_y)$ .....(5.5-7a)

Effective depth $d_1 = d_y - [(\frac{1}{2}d_y - 50)/(x - 50)][D_f - D_{f.min}]$ .....(5.5-7b)

Area resisting shear $= 2(L_1 + B_1)\ d_1$ .....(5.5-7c)

Design shear $V_{uD} = w_u\ (L_f . B_f - L_1 . B_1)$ .....(5.5-7d)

(b) Calculation of Shear Resistance (V_{uc}) and Safety Check.

Shear strength $\tau'_{uc} = k_s \tau_{uc}$ .....(5.5-8)

where $\tau_{uc} = 0.25\sqrt{f_{ck}}$ N/mm², $k_s = (0.5 + B_c) <= 1$, $B_c = b/D$.

Shear resistance $V_{uc} = \tau'_{uc}$ x area resisting shear .....(5.5-9)

For safety, $V_{uc} >= V_{uD}$ else increase the depth.

(IV) Calculation of Area of Steel including Check for Development Length :

(a) Area of Steel:

Required $A_{stx} = (0.5f_{ck}/fy)[1-\sqrt{1-4.6M_{ux}/(f_{ck}\ b'dx^2)}]b'd_x$.....(5.5-10a)

Required $A_{sty'} = (0.5f_{ck}/fy)[1-\sqrt{1-4.6M_{uy}/(f_{ck}\ D'd_y{}^2)}]D'd_y$....(5.5-10b)

(b) Check for Development Length (L_d):

Required development length shall not exceed, approximately, the lesser of the two projections (x or y) i.e. L_d ($=k\phi$) <= lesser of x and y. Or Maximum allowable bar diameter $\varnothing$ = (x or y whichever is less)/k, where $k = 0.87fy/(4\ \tau_{bd})$. For M15 - Fe415, k = 57.

If x or y is not adequate and if depth is not to be changed, A_{st} shall be increased by the ratio L_d/x Or L_d/y as the case may be.

(c) Calculation of Number and spacing of bars:

Assume bar diameter equal to or less than the maximum allowable and calculate required number(N)of bars and their spacing (which should lie preferably between 75 mm and 200 mm).

$N_x >= A_{stx}/a_{st}+1$; $s_x = (B_f-50)/(N_x-1)$, $N_y >= A_{sty}/a_{st}+1$; $s_y = (L_f-50)/(N_y-1)$.

(V) Check for One-way Shear :

If effective depth $d_x >=$ projection of footing x, then check for one-way shear is not necessary.

Critical section for one-way shear occurs at a distance of d_x (or d_y) from the face of the column (as the case may be).

For bending about x-axis bisecting the length of footing:

Width at top of footing at critical section $B_2 = b + 2dx$.

Effective depth at critical section $d_2 = d_x - [(d_x-50)/(x-50)](D_f - D_{f.min})$

Area resisting shear $= B_f\ d_2 - 0.5(B_f - B_2)\ [d_2 - (D_{f.min} - d'_x)]$

Shear resistance $V_{uc} = \tau_{uc}$ x (area resisting shear)

τ_{uc} depends on p_t based on $p_t = A_{stx}/$(area resisting shear)

Design shear $= w_u\ B_f(x-d_x) < V_{uc}$

Similar check should be applied for bending about y axis taking respective diamensions and quantities.

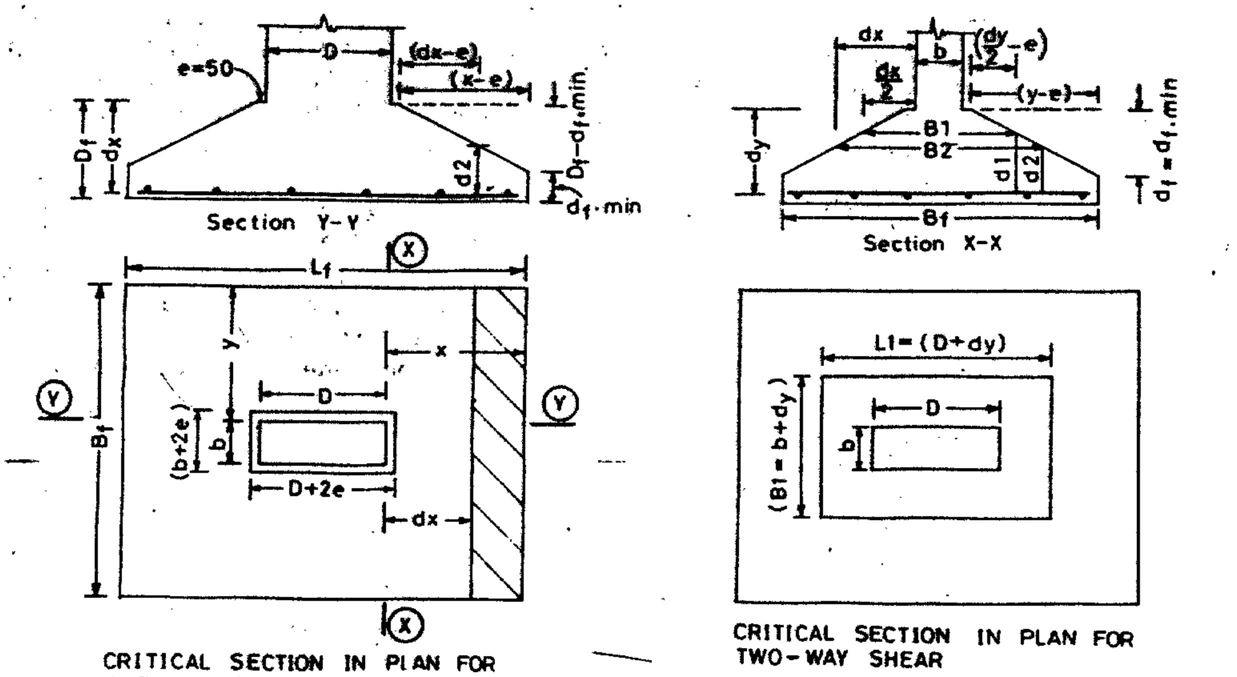


FIG. 5.2 COLUMN FOOTING

Part B : Design Procedure by Use of Design Aids

In this part, the steps for design procedure using design aids have been presented to suit writing of design calculations in a tabular form.

5.6 CALCULATION OF UNIT LOADS

Unit loads of the following components will be calculated using Table A-1.
(1) Slabs : Roof; floors of rooms, balconies, stairs, lofts, W.C., bath.
(2) Walls : External, internal, parapets, walls of sanitary block, grills.
(3) R.C.Members : Beams, columns.

5.7 DESIGN OF SLABS

Slabs will be designed in following sequence.
(1) Roof slabs, (2) Floor slabs, (3) Stairs, (4) Lofts.
The design of roof and floor slabs will be done categorywise as follows.

5.7.1 Design of One-Way Slab

Table 5.1 : Steps for Design of One-way Slab

Step No.	Details			Reference	Note No.
1.	Slab Mark				
2.	Span	(L)	meters		
3.	End Condition No.	EC		Sect.5.2.1	
4.	Imposed Load	q_i	kN/m²	Table A-1	
	Ultimate Load	qu_i	kN/m²	"	
5.	Depth	D	mm	Table C-1 or C-4	
6.	Design Moment	$M_{u.max}$	kN.m	Table B-1	(1)
7.	Main Steel (Short Span)	N - #	mm	Table C-2 or C-4	
8.	Distribution Steel(Long Span)	N - ∅	mm	Table C-3	
9.	End Shears (Loads on supporting beams)			Sect.3.4.1	(2)
	Long Edge - Simply Supported / Continuous				
	Short Edge - Simply Supported / Continuous				
10.	Check for Shear and Development Length (if required).				

Notes: (1) $M_{u.max}$ is not required in case of design of slabs for residential buildings when the Depth D and steel are obtained directly using Table C-4
(2) It is advised to show the loads transferred from slabs to beams on separate plan (See Project-III).

5.7.2 Design of Two-Way Slab

The steps for design of a two-way slab are shown in Table-5.2 (page 90)

5.8 DESIGN OF BEAMS

5.8.1 Categorisation and Grouping of Beams

Category No.	I(a)	I(b)	II	III	IV	V
One end	S.S.	S.S.	S.S.	Conti.	Fixed	Miscellaneous
Other end	S.S.	S.S.	Conti.	Conti.	Free	Any End Cond
Loading	UDL only	UDL & Pt.load	UDL only	UDL only	UDL only	Any load

Table 5.2 : Steps for Design of Two-way Slab

Step No.	Details			Reference	Note No.
1.	Slab Mark				
2.	Span – Short	L_x	metres		
	– Long	L_y	metres		
	– Aspect ratio	L_y/L_x			
3.	Boundary Condition No.			Table B-2.	
4.	Depth : (a) Assumed	p_t %		Sect.5.2.3 – Step 4.	(1)
	(b) Allowable L/d ratio			Table C-1 or Sect.4.4.1	(1)
	(c) Depth	D	mm		
5.	Load – ultimate total	q_u	kN/m²	Table A-1.	
6.	Design Moments	B.M.Coef	mu in kN.m/m		
	Short Span – Support	α'_x	m'_{ux}	Table B-2	
	– Midspan	α_x	m_{ux}		
	Long Span – Support	α'_y	m'_{uy}	Table B-2	
	– Midspan	α_y	m_{uy}		
7.	Main Steel				
	Short Sapn – Support	A'_{stx}	N – #	Table C-2	
	– Midspan	A_{stx}	N – #		
	Long Span – Support	A'_{sty}	N – #	Table C-2	
	– Midspan	A_{sty}	N – #		
8.	Torsion steel				
9.	End Shears : Equivalent UD load on supporting beam				
	(a) For bending : Long edge – Simply Supported			Sect. 5.2.3 item 11(c)	
	– Continuous				
	Short edge – Simply Supported				
	– Continuous				
	(b) For shear : Long edge – Simply Supported			Sect. 5.2.3 item 11(c)	
	– Continuous				
	Short edge – Simply Supported				
	– Continuous				

Notes: (1) This is not required if allowable L/d ratio is obtained from Table C-1 or Sect. 4.4.1 directly.

5.8.2 Design Steps for Beams of Category – I

Steps for design of beams of Category –I are shown in Table 5.3.

Table 5.3 : Steps for Design of Beam Category – I

Step No.	Details			Reference
1.	Beam Mark			
2.	Span	L	metre	
3.	Section-Assumed			
	(a) Width	b_w	mm	Sect.5.3.3 Step-4(a)
	(b) Depth	D	mm	" Step-4(b)
	(c) Depth of slab	D_f	mm	

Step No.	Details		Reference	Note No.
4.	Load for Bending:			
	I-UD Load:			
	(a) Slab Right :		Slab Mark	
	Load (End Shear) w_{us1}	kN/m	Sect.5.7.2 Step-9	
	(b) Slab Left : Slab Mark			
	Load (End Shear) w_{us2}	kN/m	Sect.5.7.2 Step-9	
	(c) Wall w_{uw}	kN/m	Table. A-1	
	(d) Imposed $w_{ui}=w_{us1}+w_{us2}+w_{uw}$	kN/m		
	(e) Self (beam) w_{ub}	kN/m	Table. A-1	
	(f) Total $w_u = w_{ui} + w_{ub}$	kN/m		
	II-Point Load:			
	(a) Supported Beam Mark			
	Supported Beam End (Left/Right)			
	(b) Point Load (W)(Shear from cross beam)	kN		
	(c) Distance 'a' from nearest support	metre		
5.	Design Midspan Moment: $M_{u.max}$	kN.m	Table B-1	
6.	Section at Midspan:			
	(a) Type (Rect./T/L)			
	(b) Flange width b_f	mm	Sect.4.1.5(a)	
	(c) Ratio b_f/b_w			
7.	Main Steel (at Midspan):			
	(a) Top St. N - #	mm	: Table D-1 or	
	(b) Bottom Bt. N - #	mm	: Table D-2 or	
	(c) Bottom St. N - #	mm	: Table D-3	
	(d) M_{ur} provided M_{ur}	kN.m	:	
	(e) Ast provided A_{st}	mm^2	:	
	(f) Effectice depth d	mm	:	
8.	Design for Shear (at each of the two ends):			
	(a) UD Load for shear :			
	Slab right: w_{us1}	kN/m	:Table - 5.1 Step-9 for one-way	
	Slab left : w_{us2}	kN/m	:Table - 5.2 Step-9(b) for two-way	
	Total $w_u=wus1+w_{us2}+w_{uw}+w_{ub}$	kN/m		
	(a) $V_{u.max}$	kN	Sect.4.2.2	
	(b) A_{st1} N - ø	mm		
	(c) $V_{ur.min}$	kN	Table D-2	
	(d) V_{uD}	kN		
	(e) V_{uc}	kN	Table D-2	
	(f) $V_{us} = V_{uD} - V_{uc}$	kN		
	(g) Designed Stirrups ø - s	mm	Table D-8	
	(h) Length of Shear Zone L_{s1}	mm	Sect.5.3.3 Step-8(f)	
	(j) Minimum Stirrups ø - s	mm	Table D-7	
9.	Check for Bond :			
	(a) Maximum bar diameter ø	mm		
	(b) Required Development Length Ld	mm	Sect.4.3.3	
	(c) $L_{ex} = L_d/3 - b_s/2$	mm		
	(d) Available M_1 corresponding to A_{st1}	kN.m	Table D-1 or D-2 or D-3	
	(e) Available $L_d=1.3\ M_1/V_{u.max} + L_{ex}$	mm		
10.	Check for Deflection :			
	(a) p_t % at midspan			
	(b) Allowable L/d ratio		Fig. 4.4.1 to Fig. 4.4.3	
	(c) Actual L/d ratio			
11.	Load transferred to support at each end (excluding triangular load on short beam from one-way slab.)			

5.8.3 Design Steps for Beams of Category II and III

Category -II : Beam Continuous at One End only, and
Category -III: Beam Continuous at Both Ends)

Steps for design of beams in these categories are same as those for beam of category I except that calculations shall be done for section at midspan as well as section at continuous end in Steps 5,7,8 and 9. Additions and alterations shall be as follows.

Table 5.3 : Additional Steps in Design of Beams of Categories II and III

Step No.	Details	Reference
5.	Design Moments:	
	(A) At Midspan	
	(B) At Continuous End	
7.	Main Steel:	
	(A) At Midspan:	: Same as in Step-7 of Table 5.3.
	(B) At Continuous End:	
	(a) Top St. N - # mm	:(This will include bent-up bars from midspan and
	(b) Bottom St. N - # mm	: may include anchor bars at top at midspan if
		: extended beyond support.)
	(c) M_{ur} provided kN.m	: Table D-1 or Table D-2 or Table D-3
	(d) A_{st} provided mm²	:
	(e) Effective depth mm	:
8.	Design for Shear :	
	(A) At Simply Supported End	: As in Step-8 of Table 5.2,for Category I only
	(B) At Continuous End	: All sub-steps (a to j) of Step-8(a) repeated.
		: These will be at both ends for Category III.
9.	Check for Bond :	
	(A) For Bottom Bars at Simply Supported End	: As in Step-9 of Table 5.2,for Category I only
	(A) For Bottom Bars at (PI)-Points of Contraflexure	: As in Step-9 of Table 5.2,for Category I only
		: except that L_{ex} = d, V_1 = V_u at point PI
		: (at a distance L/5 from support), and
		: Available $L_d = M_1/V_1 + L_{ex}$.
	(B) For Top Bars at Continuous End :	
	(a) Maximum Bar Diameter:	Same as Step-9(a) of Table 5.2.
	(b) Required L_d	: Same as Step-9(b) of Table 5.2.
	(c) Available L_d = L/4	

5.8.4 Design Steps for Beams of Category IV (Cantilevers)

The design steps for beams of category IV are same as those for beams of Category I (Table 5.2 page 91) except that steps 5,6,7,8 and 9 will be for section at support (fixed end) instead of section at midspan.

5.8.5 Design Steps for Beams of Category V

For beams of this category , it is advisable to give separate calculations for each beam giving reference to appropriate design aids (Charts/Tables) whereever they are used.

5.9 DESIGN OF COLUMNS

5.9.1 Categorisation of Columns

Categorise columns as under as explained in Section 5.4.3.

Category I : Internal Columns or Axially Loaded Columns.
Category II : Side Columns or
Columns under Combined Axial Load and Uniaxial Bending.
Category III : Corner Columns or
Columns under Combined Axial Load and Biaxial Bending.

5.9.2 Computation of Floor Loads on Columns

A : Exact Method (When Beam Shears are known in advance)

Table 5.4(a) : Computation of Floor Loads on Columns - Exact Method.

Step No.		Reference
1. Column Mark		
2. Load on Column (in kN) in Storey		
(a) R-3 (between Roof and 3rd floor) :		
(i) Shear from roof beam in direction-1	V1	From Step 8 of Table.5.3
Shear from roof beam in direction-2	V2	From Step 8 of Table.5.3
Shear from roof beam in direction-3	V3	From Step 8 of Table 5.3
Shear from roof beam in direction-4	V4	From Step 8 of Table 5.3
(ii) Assumed Section		
(iii) Self weight of assumed section	P_{self}	Table A-1
(iv) Total load from roof including weight of column in storey below	P_{R3}	$= V1 + V2 + V3 + V4 + P_{self}$
(v) Column Load Area of roof/floor	$A_{col.load}$	Sect.5.4.4(B)(II)(a)
(vi) Load on Column due to Live Load(roof)	P_{RL}	$= A_{col.load} \times LL(roof)$
(b) 3-2 (Between 3rd floor and 2nd floor) :		
(i) Shear from floor beam in direction-1	V1	From Step-8 of Table.5.3
Shear from floor beam in direction-2	V2	From Step-8 of Table.5.3
Shear from floor beam in direction-3	V3	From Step-8 of Table.5.3
Shear from floor beam in direction-4	V4	From Step-8 of Table.5.3
(ii) Total load from floor	P_f	$= V1 + V2 + V3 + V4$
(iii) Assumed Section		
(iv) Self weight of assumed section	P_{self}	
(v) Load from above	P_a	$= P_{R3}$
(vi) Total gross load on column (3-2)	$P_{32.gross}$	$= P_f + P_{self} + P_a$
(vii) Load on column due to live load(floor)	P_{FL}	$= A_{col.load} \times LL(floor)$
(viii) Reduction in load due to reduction in live load on floors above	$P_{32.LL}$	$= 0.1(P_{RL} + P_{FL})$
(ix) Net load on column in storey 3-2	P_{32}	$= P_{32.gross} - P_{32.LL}$
(c) 2-1 (2nd floor to 1st floor)		
(i),(ii),(iii),and (iv)		Same as in (b) above
(v) Load from above	P_a	$= P_{32.gross}$
(vi) Total gross load on column (2-1)	$P_{21.gross}$	$= P_f + P_{self} + P_a$
(vii) Reduction in load due to reduction in live load on floors above	$P_{21.LL}$	$= 0.2\ (P_{RL} + 2P_{FL})$
(viii) Net load on column in storey 2-1	P_{21}	$= P_{21.gross} - P_{21.LL}$

(d) 1-PL (1st floor to Plinth)	
(i),(ii),(iii), and (iv)	Same as in (b) above
(v) Load from above	$P_a = P_{21.gross}$
(vi) Total gross load on column (1-PL)	$P_{1P.gross} = P_f + P_{self} + P_a$
(vii) Reduction in load due to reduction in live load on floors above	$P_{1P.LL} = 0.3(P_{r1} + 3P_{FL})$
(viii) Net load on column in storey 1-PL	$P_{1P} = P_{1P.gross} - P_{1P.LL}$
(e)PL-FT (Plinth to Footing)	
(i),(ii),(iii), and (iv)	Same as in (b) above except that shears V1,V2,V3,V4 will be for plinth beams
(v) Load from above	$P_a = P_{1P.gr}$
(vi) Total gross load on column (PL-FT)	$P_{PF.gr} = P_f + P_{self} + P_a$
(vii) Reduction in load due to reduction in live load on floors above	Same as in (d)(vii) above.
(viii) Net load on column below plint	$P_{PF} = P_{PF.gr} - P_{1P.LL}$
Final Load on Footing	$P_u = P_{PF}$

Note :It is better to prepare a line diagram showing load transfered to column.

5.9.3 Moments in Columns = Exact Method

Moments in columns are obtained by Substitute frame anlysis [Sect.3.2.2(a)] They be written down in a form shown in Table 5.4(b). This step may be omitted in case of columns under Category - I.

Table 5.4(b) : Moments in Columns

Step No.	Details		M_{ux}	M_{uy}	Reference
1.	Column Mark				
2.	Column in Storey R-3				
	Moments M_{ux} and M_{uy} at (in kN.m)	- Top	M_{R3}		Moment in lower column (M_{cl}) for Substutute frame at roof level
		- Bottom	M_{3R}		Moment in upper column (M_{cu}) for Substitute frame at 3rd floor
	Design Moment M_u in kN.m				Greater of the values at top and bottom
3.	Column in Storey 3-2 at	- Top	M_{32}		M_{cl} of frame at 3rd floor
		- Bottom	M_{23}		M_{cu} of frame at 2nd floor
	Design Moment				Greater of M_{32} and M_{23}
4.	Column in Storey 2-1 at	- Top	M_{21}		M_{Cl} of frame at 2nd floor
		- Bottom	M_{12}		M_{cu} of frame at 1st floor
5.	Column in Storey 1-PL at	-Top	M_{1P}		M_{Cl} of frame at 1st floor
		- Bottom	M_{P1}		M_{cu} of frame at Plinth level
6.	Column below Plinth at	- Top	M_{PF}		M_{Cl} of frame at Plinth level
		- Bottom	M_{FP}		Moment at footing end of frame at Plinth level

M_{cu} = moment in upper column at any floor level,
M_{Cl} = moment in lower column at any floor level.

5.9.4 Effective Length of Column : Exact Method

Effective length of column is obtained as explained in Sect.4.5.1(c)(ii) and may be written in the form shown in Table 5.4(c).

5.4(c) Effective Lengths of Column and Column Type (Short or Long)
Exact Method

Details	Reference
(i) For bending about X – axis (bisecting the depth)	
umn Mark	
umn in Storey R-3:	
Top Joint – I_{c1} Moment of inertia of lower column(R-3)	
(at roof level) – L_{c1} Length of lower column	
– K_{c1} Stiffness (I/L) of lower column(R-3)	
– $\Sigma K_c = K_{c1}$	
– I_{b1} Moment of inertia of beam to the left	
– L_{b1} Length of beam to the left	
– K_{b1} Stiffness (I/L) of beam to the left	
– I_{br} Moment of inertia of beam to the right	
– L_{br} Length of beam to the right	
– K_{br} Stifness (I/L) of beam to the right	
– $\Sigma K_b = K_{b1} + K_{br}$	
– $\beta 1 = K_c/(\Sigma K_c + \Sigma K_b/2)$	
Bottom Joint – K_{cu} = K of upper column=K_{c1} calulated above for top jt.	
(at 3rd floor – K_{c1} = K (=I_{c1}/L_{c1}) of lower column (3-2 in this case)	
level) – $\Sigma K_c = K_{cu} + K_{c1}$	
– K_{b1} = K (=I_{b1}/L_{b1}) of beam to the left	
– K_{br} = K (=I_{br}/L_{br}) of beam to the right	
– $\Sigma K_b = K_{b1} + K_{br}$	
– $\beta_2 = K_c/(\Sigma K_c + \Sigma K_b/2)$	
Ratio $r_{eff} = L_{eff}/L$ corresponding to $\beta 1$ and $\beta 2$ obtained from Fig.4.5-2.	
$L_{eff.x} = r_{eff} \times L$	
Ratio $L_{eff.x}/D$	
Column type : Short if $L_{eff.x}/L <= 12$, otherwise long.	
umn in Storey 3-2 : All sub-steps (a) to (f) are same as those of	
umn in Storey 2-1 : Step-2 above.	
umn in Storey 1-PL :	
umn below plinth :	
) For bending about Y-axis (bisecting width)	
The steps will be same as those in part (i) above except that now D and be interchanged and beams to the left and right will have different sec- nd lengths.	

.9.5 Grouping of Columns

olumns under each category of same type (short or long) , of same section, bjected to nearly equal values of P_u, M_{ux} and M_{uy} (within ± 20% variation) f nearly equal ratio $L_{eff}/(b$ or $D)$ if long, be grouped together for the of design to reduce computational efforts and save labour during ion of work.

.9.6 Design of Reinforcement

he longitudinal and transverse reinforcement will be determined by use of and tables given in Appendix E. If the column is slender design moments ated in Sect.5.9.3 will be incremented to account for the effect of slen- s according to the procedure given in Table 5.4(d) before designing the rcement. If the column is short, calculations for design of reinforcement carried out directly according to the procedure shown in Table 5.4(e),for subjected to Axial Load and Uniaxial bending and as given in Table 5.4(f) lumn subjected to Axial Load and Biaxial Bending.

Table 5.4(d) : Steps for Modifying the Moments in Design of Slender Columns

Step No.	Details			Reference
I.	Column mark			
II.	Column in Storey R-3			
1.	Assumed section - Width	b	mm	
	- depth	D	mm	
2.	Effective Length-	$L_{eff.y}$	mm	
3.	Ratio $L_{eff.y}/b$			
4.	Axial Load	P_u	kN	
5.	Moment at Top	M_2	kN.m	
6.	Moment at Bottom	M_1	kN.m	
7.	Minimum Moment	$M_{u.min}$	kN.m	
8.	Initial Moment	M_i	kN.m	Eq.4.5-17
9.	Additional Moment	M_{ay}	kN.m	EQ.4.5-15b
10.	Ideal Axial Load	P_{uz}	kN	Eq.4.5.3 or Chart 9
11.	Load (Balanced)	P_{ub}	kN	Eq.4.5-12 or Table E6
12.	Ratio $k = (P_{uz}-P_u)/(P_{uz}-P_{ub})$			Eq.4.5-16
13.	Revised additional moment		kN.m	Eq.4.5-15b
14.	Equivalent Total moment for a short column	M_{uT}	kN.m	Eq.4.5-18

III. Column in Storey 3-2 : All sub-steps are same as those of Step-2 above.
IV. Column in Storey 2-1 :
V. Column in Storey 1-PL :
VI. Column below Plinth :

Proceed to design of reinforcement as given in Table 5.4(e) or (f) as the case may be.

Table 5.4(e) : Design of Reinforcement for Short Columns subjected to Axial Load and Uniaxial Bending - Exact Method

Step No.	Details	Reference
I.	Column Mark	
II.	Column in Storey R-3 :	
1.	Assumed Section : b x D mm	
2.	Unsupported Length L m	
3.	(a) Minimum Eccentricity $e_{x.min}$	
	(b) Minimum Eccentricity $e_{y.min}$	
4.	Axial Load P_u (kN)	
5.	(a) $M_{ux.min} = P_u \times e_{x.min}$	
	(b) $M_{uy.min} = P_u \times e_{y.min}$	
6.	(a) External Moment M_{ux} (kN.m)	
	(b) External Moment M_{uy} (kN.m)	
7.	(a) Design Moment M_{ux} (kN.m)	
	(b) Design Moment M_{uy} (kN.m)	
8.	Main Steel	
	(A) By Use of Tables :	
	Required N - # for Bending @ X-axis	: Table E-1
	Allwable P_u for this steel	: or
	for bending @ Y-axis	: Table E-2

Table 5.4(e) Continued

Step No.	Details	Reference
II.	Column in Storey R-3 Continued	
	8. Main Steel : Continued	
	(B) By Charts :	
	(a) Design for bending @ X-axis :	
	(i) Assumed d'	
	(ii) Ratio d'/D	
	(iii) f_{ck} bD	
	(iv) Assumed steel arrangement for chart selection	
	(v) $P_u/(f_{ck}\ bD)$	
	(vi) $M_{ux}/(f_{ck}\ bD^2)$	
	(vii) Required p/f_{ck}	Charts in Appendix E-3
	(viii) Required $A_{sc}=(p/f_{ck}).f_{ck}\ bD/100$ mm²	
	(ix) Provided N - # mm	
	(x) Provided A_{sc} mm²	
	(b) Check for bending @ Y-axis :	
	(i) Ratio d'/b	
	(ii) $M_{uy}/(f_{ck}\ bD^2)$	
	(iii) Required p/f_{ck}	Charts in Appendix E-3
	(iv) Required $A_{sc}=(p/f_{ck}).f_{ck}\ bD/100$ mm²	
III.	Column in Storey 3-2 :	All sub-steps are
IV.	Column in Storey 2-1 :	same as those
V.	Column in Storey 1-PL:	given in
VI.	Column below Plinth :	Step-II above.

Table 5.4(f) Design of Reinforcement for Columns subjected to Axial Load and Bi-axial Bending - Exact Method

Step No.	Details	Reference
I.	Column Mark	
II.	Column in Storey R-3	
	(1) to (7)	Same as those in Table 5.4(e) above
	(8) Main Steel : By Charts only	
	(i) Assumed d'	
	(ii) Ratio d'/D	
	(iii) Ratio d'/b	
	(iv) f_{ck} bD	
	(v) Assumed Steel percentage p and arrangement	
	(vi) Ratio p/f_{ck}	
	(vii) $P_u/(f_{ck}\ bD)$	
	(viii) $M_{ux1}/(f_{ck}\ bD^2)$	Charts in Appendix E-3
	(ix) $M_{uy1}/(f_{ck}\ b^2D)$	Charts in Appendix E-3
	(x) M_{ux1} in kN.m	
	(xi) M_{uy1} in kN.m	
	(xii) P_{uz}	Chart 9
	(xiii) Ratio P_u/P_{uz} and α_n	Table E-4
	(xiv) Value of L.H.S. of Interaction Equation.	Eq. 4.5.14
	(xv) Required A_{sc} = p bD/100 mm²	
	(xvi) Provided Main Bars N - # mm	
	(xvii) Lateral Ties : Diam. & spacing mm	Table E-5

5.9.7 Assessment of Load and Preliminary Design of Columns : Approximate Method

(When Exact Beam Shears are not known prior to Column Design)

In case, when need arises of designing columns and column footings prior to design of slabs and beams, following procedure may be adopted.

I - For plan with regular column positions, obtain the load on column at each floor from the area of floor shared by the column. See Project-II.

II- For plan with irregular layout and having number of secondary beams the above approach fails to give proper estimation of load. In such case the column loads are worked out approximately from loads on beam and end shears as explained below. See Project - III (Chapter - 8).

A - Assessment of Slab End Shears on Plan

1. Draw a plan showing spanning of slabs(one-way or two-way).
2. Assuming unit loads of slabs according to Table 2.1 or Table A-1 and Note of Table 2.1, write down end shears for each slab using shear coefficients given in Sect.3.4.1 and Sect.3.4.2.

B - Assessment of Approximate Loads on Beams and Beam End Shears

3. Assuming unit loads of walls,chhajas, grills,and beams as given in Table A-1, assess approximately load on every beam and calculate end shears using shears coefficients given in Sect.5.3.3, Step-8(b). Start with Secondary beams first, followed successively by beams of Categories I,II,III,IV,and V or baywise with bays of equal spans as illustrated by calculations of Project-III. Write down beam loads and beam end shears as shown in Table 5.5(a) below.

Table 5.5(a) : Calculation of Loads on Beams and End Shears

Beam Mark	Span	Ec No.	Slab Mark Load	Left w_{u1}	Right w_{u2}	Wall/Grill w_{ux}	Self Weight w_{us}	Total Load w_u	End Shears Left V_{uL}	End Shears Right V_{uR}

These calculations will be done separately for beam at roof level, floor level, and plinth level.

4. Draw a plan showing beams. Write down beam end shears at roof level, floor level and plinth level on the plan.
5. Categorise the columns as explained in Sect.5.4.3.
6. Calculate approximate loads on columns of each category as shown in Table 5.5 ignoring self weight of columns (since this is approximately taken into account in computation of wall loads between centre to centre of columns)

Table 5.5(b) : Assessment of Loads and Preliminary Design of Columns Approximate Method

Step No.	Details		Reference
I.	Column Mark		1
II.	Column Load area	$A_{Col.load}$	:
III.	Load on Column in		:
	(a) Storey R-3 (between roof and 3rd floor)		4------C------2
	1. Beam end shears from direction-1 V1 :	From	:
	from direction-2 V2 :	Tab.5.5(a)	:
	from direction-3 V3 :	or from	3
	from direction-4 V4 :	plan mentioned in Step-4.	
	Total shear from all beams roof	$P_{uR} = V1 + V2 + V3 + V4$	
	2. Allowance for moment due to fixity	$P_{al.f}$ = % of P_{uR} See Sect.3.4.3	
	3. Total equivalent axial load	$P_{uR3} = P_{uR} + P_{al.f}$	
	4. Assumed Section		
	5. Allowance for slenderness	$P_{al.s}$ % of P_{uR} only	
	6. Total axial design load	$P'_{uR3} = P_{uR3} + P_{al.s}$	

Table 5.5(b) Continued

Step No.	Deatils	Reference
III.	Load on Column in (a) Storey R-3 continued :	
	7. Preliminary design of reinforcement	
	Main Steel : N - # mm	Table E-1
	Lateral Ties : ⌀ - s mm	Table E-5
	(b) Storey 3-2 (between 3rd and 2nd floor)	
	1. Beam end shear from direction-1 V1 :	From Table 5.5(a)
	from direction-2 V2 :	
	from direction-3 V3 :	From Table 5.5(a)
	from direction-4 V3 :	
	Total load from floor beams P_{uF}	$= V1 + V2 + V3 + V4$
	2. Load from above P_a	$= P_{uR}$ in Step-(a) above
	3. Total load at 3rd floor P_{u3}	$= P_a + P_{uF} = P_{uR} + P_{uF}$
	4. Allowance for bending (fixity) $P_{al.f}$	% of P_{uF} only. Sect.3.4.3
	5. Total equivalent axial load P_{u32}	$= P_{u3} + P_{al.f}$
	6. Section assumed	
	7. Allowance for slenderness $P_{al.s}$	% of P_{u3}
	8. Total axial design load P'_{u32}	$= P_{u32} + P_{al.S}$
	9. Preliminary design of reinforcement	
	Main Steel : N - # mm	Table E-1
	Lateral Tis : ⌀ - s mm	Table E-5
	(c) Storey 2-1 (between 2nd and 1st floor)	:Steps same as in (b) above
	(d) Storey 1-PL (between 1st floor and plinth):	except that P_{uf} gets added at every floor level and
	(e) Column below plinth	P_{uP} at plinth level
	Total load on footing $P_u = P_{uR} + 3P_{uF} + P_{uP}$.	

This can be obtained directly for design of footings if required prior to design of columns.

5.9.8 Assessment of Moments due to Fixity and Check for Combined Effect of P_u and M_u - Approximate Method

This may be done for each column as shown in Table 5.5(c) below.

Table 5.5(c) : Assessment of Moments due to Fixity and Check for Combined Effect of P_u and M_u - Approximate Method

Step No.	Details	Reference
I.	Column Mark	
II.	Column in any Storey	
	(i) Bending @ X-axis	
	(a) Assessment of Column Moments	
	1. Axial load on column - P_u	
	2. Cross beam mark - Left	
	3. Load on cross beam - Left	
	4. Fixed end beam moment- Left	
	5. Cross beam mark - Right	
	6. Load on cross beam - Right	
	7. Fixed end beam moment- Right	
	8. Unbalanced moment - M_{ue}	Value may be reduced to half for partial fixity

Table 5.5(c) Continued

Step No.	Details	Reference
II.	Column in any Storey (i) Bending @ X-axis	
	(a) Assessment of column moment continued	
	9. M.I. of left beam – I_{bL}	
	10. Length of left beam – L_{bL}	
	11. Stiffnes of left beam – K_{bL}	
	12. M.I. of right beam – I_{bR}	
	13. Length of right beam – L_{bR}	
	14. Stiffness of right beam – K_{bR}	
	15. M.I. of upper column – I_{cu}	
	16. Length of upper column – L_{cu}	
	17. Stiffness of upper col – K_{cu}	
	18. M.I. of lower Column – I_{c1}	
	19. Length of lower column – L_{c1}	
	20. Stiffness of lower col – K_{c1}	
	21. Sum of Stiffnesses – ΣK	
	22. Moment in upper column-M_{cu} = $(K_{cu}/\Sigma K).M_{ue}$	
	23. Moment in lower column-M_{cl} = $(K_{c1}/\Sigma K).M_{ue}$	
	(b) Determination of Column Type	
	1. $L_{eff.x}$	
	2. $L_{eff.x}/D$	
	3. Column Type Short or Slender. If short, proceed to Step (d) below.	
	(c) Calculation of Modified Moment for Slender Column. Steps 1 to 14 of Table 5.4(d).	
	(d) Checking the Safety of Section for Combined Action of P_u and M_u. Steps 1 to 7 of Table 5.4(e).	
	ii) Bending @ Y-axis : Steps as in (a),(b),(c) and (d) above repeated.	
	(iii) Biaxial Bending : Steps as in (a),(b),(c) and (d) above except that in (d) checking of safety will be done according to Steps in Table 5.4(f) instead of Table 5.4(e)	
III.	Column in Storey 3-2 : All steps	
IV.	Column in Storey 2-1 : same as in	
V.	Column in Storey 1-P1: Step-II	
VI.	Column below plinth : above.	

Note: After designing number of columns in different buildings, the designer gets adequate data and judgement regarding safety of a column under combined bending and axial load from calculations of similar columns done earlier. He may keep a record of calculations of typical columns for future use to save calculation efforts in future. After adequate experience, designer is also in a position to judge the necessity of above calculations for check for moments.

5.10 DESIGN OF COLUMN FOOTING

1. Load from column P_u kN
2. Column Section b x D mm
3. Bearing Capacity of Soil kN/m²
4. Required Size of Footing L_f x B_f mm
5. Required Depth of Footing H_f mm
6. Required Reinforcement : Long N_x – #
   : Short N_y – #

Project – I : Design of Single Storey Public Building

6.1 INTRODUCTION

6.1.1 General

As stated earlier in objective and scope of this book, three projects have been presented 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 (except the Code), using Limit State Method of design and S.I.units, conforming to IS:456-1978.

Since, the object of this project is to illustrate the design of R.C.members rather than the analysis of a framed structure, a 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 stair.

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 with curtailment (B19),
Continuous with two equal spans (B22-B23),
Continuous with two unequal spans (B17-B18), and
Continuous with non-central point load (B5).

Columns : Axially Loaded (C2), Eccentricaly Loaded - Uniaxial Bending (C8),
Eccentrically Loaded - Biaxial bending (C15).

Column Footings.

This design forms a good exercise for a beginner in the field of design in understanding the design principles and practices.

6.1.2 Data

1. Type : Single Storeyed R.C.Framed Structure
2. Plan : As shown in Fig.6.1
3. Use : Public purpose,Bank,Office,Shop,Assembly hall
4. Geometric Details : Floor to floor hight = 3.2 Metres
   Height of plinth = 0.6 Metre above ground level
   Depth of foundation = 0.72 Metre below ground level
5. Loads : Roof : Live Load-access provided = 1.5 kN/m²
   -access not provided = 0.75 kN/m²
   Floor finish = 1.75 kN/m²
6. Specifications for Materials and Building Components:
   Roof : R.C.Slab,flat type with waterproofing course
   Walls : Brick Masonary 230mm thick duly plastered
   Concrete : Grade M15
   Steel : Grades, Main - Fe415, Secondary - Fe250.
7. Foundation : Bearing Capacity of Soil = 150 kN/m².
8. Assumptions for Design: Slab simply supported over beams and beams simply supported over columns
9. Design Philosophy : Limit State Method conforming to IS:456-1978.

Fig. 6.1

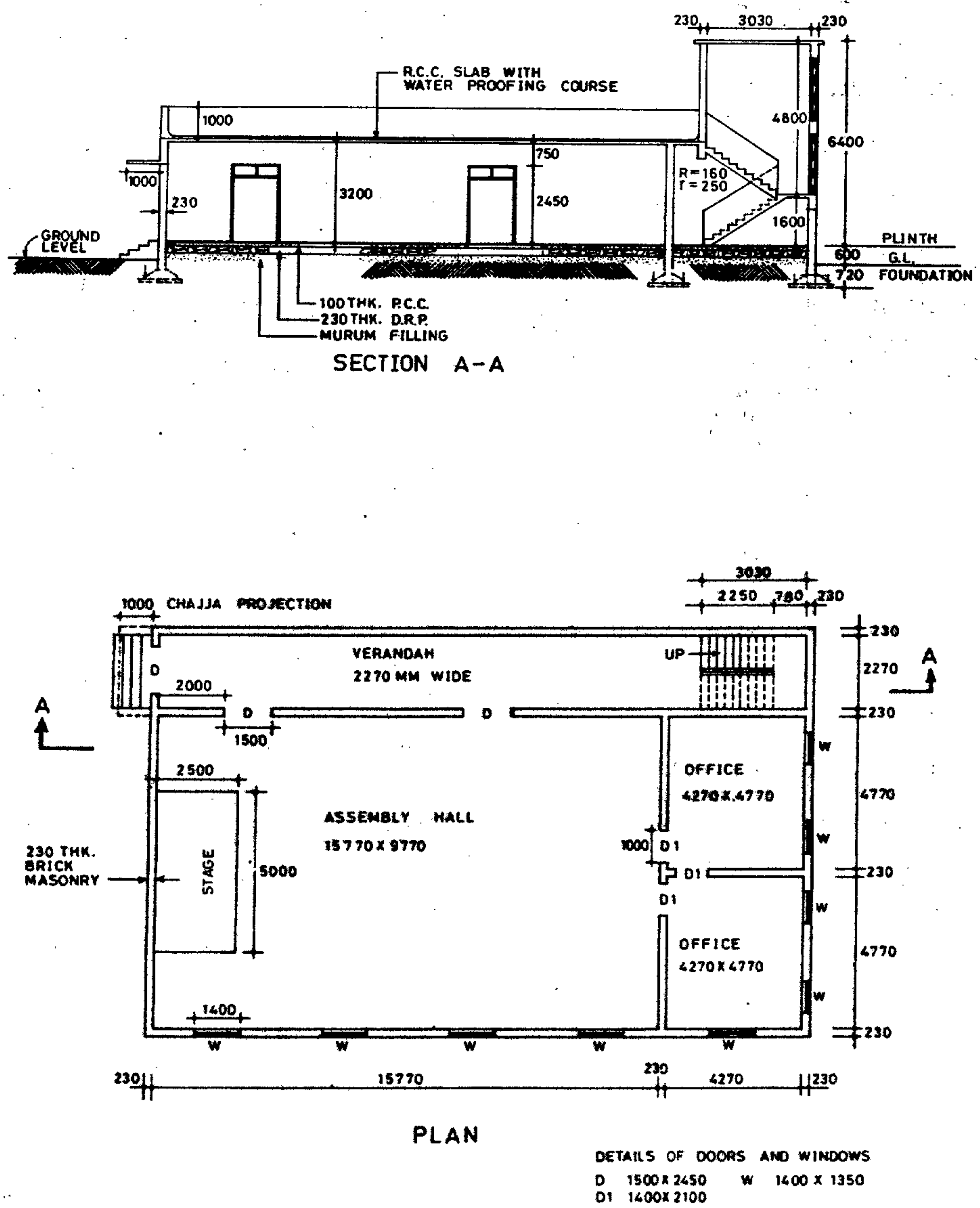


Fig. 6.1 Single Storey Public Building

6.1.3 Preliminaries

As explained in Sect.5.1, the structural planning is to be done for the building plan shown in Fig.6.1. 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 requirement 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 are only along the periphery of the hall, office rooms and staircase. 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 slabs. 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. Providing longitudinal beams at right angles to main beams do not render any help either structurally, architecturally or economically. On the other hand, such beams bring point loads on main beams which are already long and make them heavy and uneconomical. Therefore, appropriate 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 slab over last span of 4.5 m is concerned, one-way continuous slab also will be uneconomical, and 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 automatically get fixed under main beams (C7 to C12 and C15 to C20) and at junctions of cross beam with main beams (C13, C14). 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. 6.2. Alternatively, a grid floor is also possible for such a hall. However, it is beyond the scope of this book.

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 in private sector. See Sect.1.4.3.

6.2 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.

6.2.1 Slab - S1

Step No. (1)	Design Calculations (2)	Reference (3)	Expl. Note No. (4)
1.	Type : One-way Cantilever	*References with star mark refer to IS Code.	
2.	Span : $L = 1\text{m} = 1000$ mm		

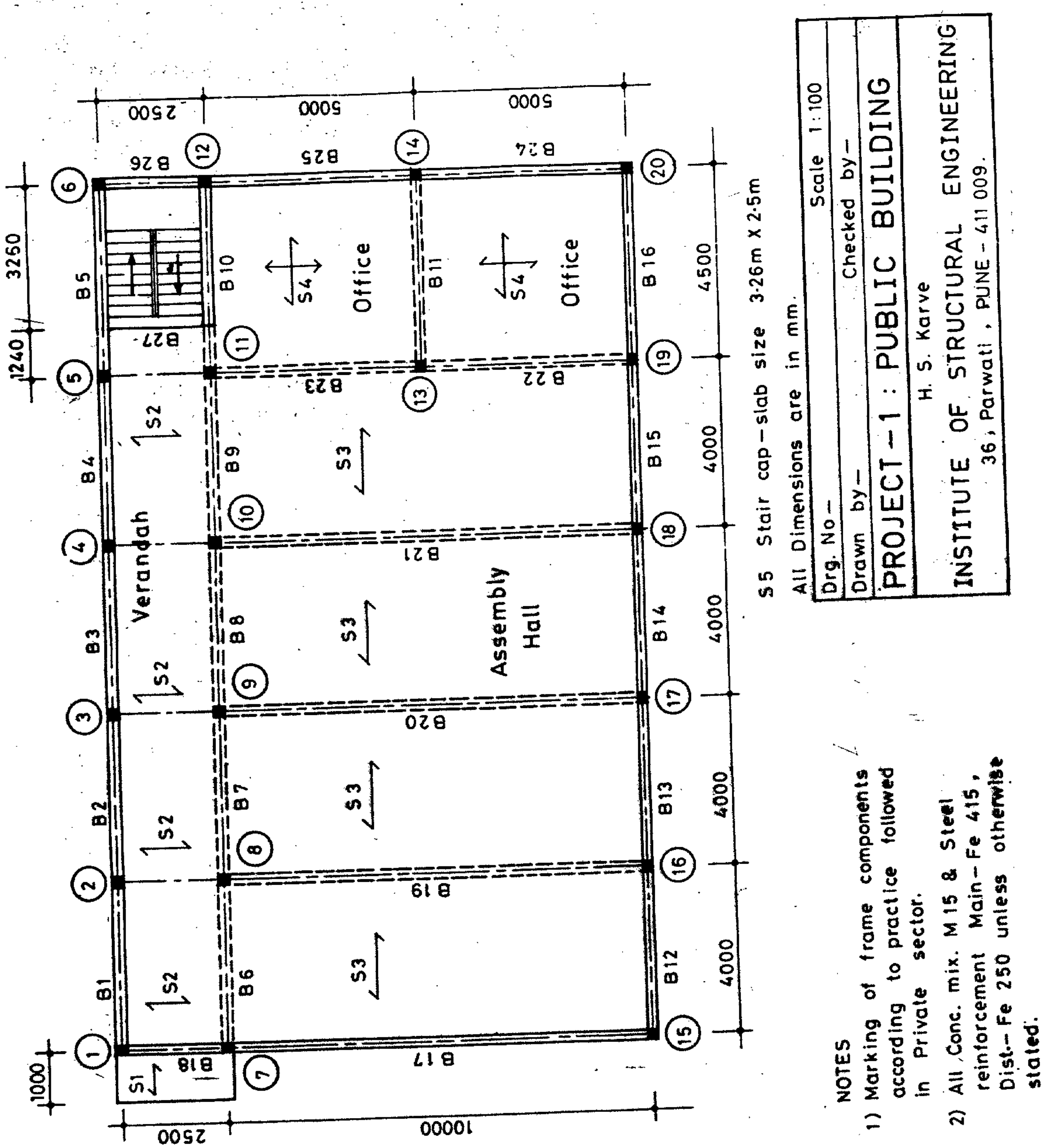


Fig. 6.2 Structural Plan of Single Storey Public Building

Design of slab S1 Continued

Step No.	Design Calculations	Reference	Note No.
3.	Imposed Load : Floor finish = 1.75 kN/m² Live load = 0.75 kN/m² Total imposed load = 2.50 kN/m²	*References with star Mark refer to I.S. Code. Others refer to this book.	(1)
4.	Trial Depth : It is decided by serviceability criteria. Basic L/d ratio r_b = 7 for cantilever Assume p_t=0.2% for M15-Fe415 as load is very light Modification factor α_1 = 1.6 Allowable L/d ratio r_a = 1.6x7 =11.2 Required eff. depth d = span/r_a=1000/11.2=say 90mm Assuming eff. cover d'= 20 mm for Fe415 Required Total depth D = 90 + 20=110 mm	*22.2.1(a) Fig.4.4.1	(2) (3) (4)
5.	Loads: Consider a strip of slab of width b=1 m = 1000 mm Self weight = 25 D where D is in metres = 25x110/1000 = 2.75 kN/m Imposed load = 2.50 kN/m Total load w =2.75+2.50= 5.25 kN/m Design ulti. load w_u = 1.5w = 1.5x5.25 = 7.875 kN/m	*35.3.2 *35.4.1	(5)
6.	Design Moment : $M_{u.max} = w_u L^2/2$ = 7.875x1x1/2 = 3.94 kN.m		
7.	Check for Concrete Depth : $M_{ur.max} = R_{u.max} bd^2$. For slab, b = 1000 mm $R_{u.max}$ = 2.07 N/mm² for M15 - Fe415 $M_{ur.max} = 2.07 \times 1000 \times 90^2 \times 10^{-6} = 16.77$ kN.m > $M_{u.max}$ ∴ O.K.		(6)
8.	Main Steel : Required $p_t(\%) = (.5f_{ck}/f_y)[1-\sqrt{1-(4.6M_{u.max}/(f_{ck}.bd^2)}]\times 100$ $p_t=(0.5\times 15/415)[1-\sqrt{1-4.6\times 3.94\times 10^6/(15\times 1000\times 90^2)}]\times 100=0.14\%$		

Explanatory Notes to Design of Slab S1 :

(1) Since parapet wall extends only over beam B18 (See Fig.6.1), the cantilever slab panel S1 is inaccessible, and therefore, live load is taken as .75 kN/m².

(2) The value of p_t satisfying requirements of both strength and serviceability normally lies between 0.20% to 0.45% for M15-Fe415. Assume smaller percentage for light loads and small span, and vice-versa. Better the judgement, lesser are the trials.

(3) Values of effective depth d and total depth D are generally rounded off to multiples of 10 mm.

(4) Maximum effective cover d' is obtained by assuming maximum bar diameter equal to 10 mm for Fe415 grade and 12 mm for Fe250 grade. Thus,

Effective cover d' = 15+10/2 = 20 mm for Fe415,

and d' = 15+12/2 = 21 mm for Fe250.

(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 = 1.5xw.

(6) The value of M_u is normally expressed in kN.m. Since $R_{u.max}$ is in N/mm² 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.

Eventhough, $M_{ur.max}$ is very large compared to actual maximum design moment the depth cannot be reduced due to requirements of serviceability. 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).

Design of Slab – S1 Continued

Step No.	Design Calculations	Reference	Note No.
8.	Main Steel : Continued		
	Required A_{st} = 0.14x1000x90/100 = 126 mm^2		
	Required $A_{st.min}$ = 0.12 % of gross area = 0.12x1000x110/100 = 132 mm^2	*25.5.2.1	
	Assuming bar diameter = 8mm, Area of bar a_{st} = 50 mm^2		
	Required spacing s= 1000xa_{st}/A_{st}=1000x50/132 = 378 mm		
	Maximum permissible spacing = 3d or 450 mm whichever is less = 3x90 = 270 mm.Provide #8 at 270mm	*25.3.2(b)(1)	
9.	Check for Serviceability :		
	Since required p_t = 0.14% < assumed p_t = 0.2 %. .·.O.K.		(7)
10.	Distribution Steel : Assuming steel of grade Fe250,		(8)
	Required A_{st} = 0.15 % of gross cross-section = 0.15x1000x110x/100 = 165 mm^2	*25.5.2.1	
	Assuming 6mm bars,a_{st}= 28.27mm, Spacing s = a_{st}X1000/A_{st}		
	.·. s =28.27X1000/165=171mm. Provide s = 160 mm		
	Maximum permissible spacing= 5d or 450mm whichever is less = 5x110 or 450mm whichever is less i.e.450mm > 160mm	*25.3.2(b)(2)	
	.·. O.K. .·.Provide ⌀ 6 at 160 mm		
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.		

Explanatory Notes to Design of Slab-S1 Continued

(7) When required p_t works out to be less than p_t assumed in the beginning, the modification factor α_1 (and hence allowable L/d raio) increases thereby requiring depth less than assumed trial depth. Therefore, check for serviceability is automatically satisfied. If required p_t > assumed p_t,increase the depth and revise calculations from first step.

(8) Alternatively, if Fe415 grade of bars are to be used for distribution steel also, the smallest bar diameter available in market for this grade of steel being 8 mm,one may adopt #8 mm with minimum percentage 0.12% for Fe415 grade In this problem,the main steel is also minimum giving A_{st} =132 mm^2 and spacing of bars 378 mm as calculated in Step No.8. Maximum permissible spacing for distribution steel is 450 mm as calculated in Step No.10. Therefore, #8 at 375 mm can be provided theoretically. In practice, however, maximum spacing is not allowed to exceed 300 mm. Therefore, practical spacing of 300 mm will be provided instead of 375 mm.

Explanatory Notes to Design of Slab-S2 :

(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 one-way along the short span. Even if effect of B18 and B27 is considered, the ratio of long span to short span is greater than 2. Therefore,the slab shall be designed as one-way slab.

(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. The slab S-2, will, therefore, be considered as simply supported over 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.

6.2.2 Slab – S2

Step No.	Design Calculations	Reference	Note. No.
1.	Type : One-way, Simply Supported, Single Span.		(1,2)
2.	Span : L = 2.5 m = 2500 mm.		(3)
3.	Imposed Load : Live load LL = 1.50 kN/m² Floor finish FF = 1.75 kN/m² Total imposed load wi = 3.25 kN/m²		
4.	Trial Depth : This is decided by serviceability. Basic L/d ratio r_b = 20 for simply supported Assuming p_t = 0.3 % for M15-Fe415, α_1 = 1.4. Allowable L/d ratio $r_a = \alpha_1 x r_b$ = 1.4x20 = 28. Required eff. depth d = span/r_a=2500/28=say 90mm. Assuming eff. cover d'= 20 mm for Fe415 Required total depth D = 90+20 = 110 mm	*22.2.1(a)	
5.	Loads:Consider one metre width of slab (b= 1m = 1000mm). Self weight = 25x0.11 = 2.75 kN/m Imposed load w_i = 3.25 kN/m Total working load w = 6.00 kN/m Design ultimate load w_u= 1.5w = 9 kN/m		
6.	Design Moment: $M_{u.max}$=w_u $L^2/8$= 9x2.5²/8 = 7.03 kN.m		
7.	Check for Concrete Depth : $M_{ur.max}$= $R_{u.max}$ bd². For slab, b = 1000 mm. $R_{u.max}$ = 2.07 N/mm² for M15-Fe415. ∴ $M_{ur.max}$= 2.07x1000x90²x10⁻⁶ = 16.77 kN.m > $M_{u.max}$ ∴ O.K.		
8.	Main Steel : Required pt(%)= $(0.5f_{ck}/f_y)[1-\sqrt{1-4.6xM_{u.max}/(f_{ck}\ bd^2)}]x100$ $p_t=(.5x15/415)[1-\sqrt{1-4.6x7.03x10^6/(15x1000x90^2)}]x100=0.26\%$. Required A_{st} = p_t bd/100 = 0.26x1000x90/100 = 234mm² Provide #8 mm (a_{st} = 50 mm²) at spacing s = 1000xa_{st}/A_{st} = 1000x50/234=213mm say 200 mm. Permissible s = 3d or 450 mm whichever is less. = 3x90 or 450 mm whichever is less = 270 mm > provided s(=200mm). ∴ O.K. ∴Provide #8 mm at 200 mm as required.		
9.	Check for Serviceability : Required p_t = 0.26 % < assumed p_t = 0.3 %. ∴ O.K.		
10.	Distribution Steel:Since D=110mm equal to that of slab S1, provide distribution steel also as that of S1. Provide ø 6 mm at 160 mm.		

Explanatory Notes for Slab S2 : Continued

3. The effective span depends upon the type of connection between the slab and 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:21.2.(c) of Code. Here in this problem, the connection between slab and beam is not rigid. Therefore, Cl.21.2(a) of the Code applies. Theoretically, effective span = 2500- 230 + eff.depth = 2270 + 90 = 2360 mm i.e. 2.36 m. However, in practice,centre to centre distance is, normally, taken as effective span, on safer side, as it gives span greater than theoretical.

Slab - S2 Continued

Step No.	Design Calculations	Refereneće	Note No.
11.	Check for Shear: Design shear V_{uD} = 9x2.5/2=11.25 kN		
	Shear resistance of slab concrete $V_{uc}=kT_{uc}.bd$	Sect.4.2.4	
	T_{uc} depends upon $p_t = 100A_{st1}/(bd)$.		
	Assuming 50% bars only continued to support,		
	A_{st1}= $(1000a_{st}/s)/2=(1000x50/200)/2=125$ mm².		
	p_t = 100x125/(1000x90)=0.14%. T_{uc}=0.272 N/mm²	Table 4.2	
	k = 1.3 for D = 110 mm.	Sect.4.2.4	
	.˙. V_{uc} = 1.3x0.272x1000x90/1000=31.82 kN		
	> 11.25 kN. .˙. O.K.		(4)
12.	Check for Development Length : For M15-Fe415,		
	Required $(A_{st1}/A_{st.max}) = r(L_d-L_{ex})/(1.3\alpha L)$	Sect.5.2.2 Step-12.	
	In this case, L_d=57 Ø=57x8=456mm for M15-Fe415	Sect.4.3.3	
	r =0.5, α=1/8, L_{ex} =230/2-25 =90 mm, L =2500mm		
	.˙. Required $(A_{st1}/A_{st.max})$=0.5(456-90)/(1.3x2500/8)		
	= 0.45. .˙. 50 % bars could be bent-up as usual.		(4)

Explanatory Notes for Design of Slab - S2 : Continued

(4) These two checks are normally satisfied in One-way slabs. The check for development length should however be applied in case of two-way slabs to verify whether steel required to be continued at bottom shall be 50% or 67%. Check for shear can be safely omitted for all slabs.

Explanatory Notes to Design of Slab- S - 3

Design-I : According to I.S. Code Coefficient:

(1) Though every slab panel is suppored on all sides,the ratio long span to short span is greater than 2.Therefore,the slab will be designed as one-way slab.

(2) Since, in continuous slabs,the B.M. coefficients for dead load and live load are different in I.S.Code, DL and FF are added while LL is kept separate.

(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.This leads to adoption of different practices of detailing of bars.

According to new practice of reinforcement detailing seperate reinforcement is provided at bottom in midspan region and at top over support region. The two are independent of one another. The diameter and pitch at the two regions can, therefore,be different according to requirement;and it is not necessary to match the spacing of bars in two adjacent spans as it is many times done in conventional practice.This practice of detailing obviously requires use of chairs for top steel at support to hold the steel at top of slab at the time of casting, but it saves labour of bending of bars.

6.2.3 Slab - S3

This slab is designed by three different approaches.

(I) By use of I.S.Code coefficients,
(II) By Exact Theory without and with Redistribution of moments,and
(III)By Approximate Method ,
to show that design by the last approach is simple as well as safe.

Slab - S3 Design - I : Using I.S. Code Coefficient

Step No.	Design Calculations	Reference	Note No.
1.	Type : One-way Continuous with more than 3 equal spans.		(1)
2.	Span : L = 4.0 metres each = 4000 mm.		
3.	Imposed Load : Live Load LL = 1.50 kN/m² Floor Finish FF= 1.75 kN/m²		(2)
4.	Trial Depth : This is decided by serviceability. Basic L/d ratio for continuous slab r_b = 26 Assuming p_t = 0.4% for M15-Fe415 for large span Modification factor $\alpha 1$ = 1.28 Allowable L/d ratio r_a = 1.28x26 Required d = L/r_a = 4000/(1.28x26) say 120mm Assuming eff.cover d'= 20 mm for Fe415 Required total Depth D = 120 + 20 = 140 mm.	*22.2.1(a) Fig.4.4.1	
5.	Loads :Consider one metre width of slab (i.e.b=1m=1000mm) Dead Load: Self weight + Floor Finish w_d = 25x0.14 +1.75 = 5.25 kN/m Live Load w_l = 1.50 kN/m		(2)
6.	Design Moments : Bending moments at different sections are calculated using I.S.Code coefficients.		

Section		1	2	3	4		
B.M.Coef. for :							
Dead Load	αd	1/12	-1/10	1/24	-1/12	*Table-7 or	
Live Load	αL	1/10	-1/9	1/12	-1/9	Table B-1	(3)

Bending moment at any section is given by

$$M_u = 1.5(\alpha_d\ w_d\ L^2 + \alpha_L\ w_l\ L^2)$$

At middle of outer span : (i.e. at Section - 1)

M_{u1} = 1.5(5.25x4²/12+1.50x4²/10) = 14.10 kN.m

At penultimate support : (i.e. at Section - 2)

M_{u2} =-1.5(5.25x4²/10+1.50x4²/9) =-16.60 kN.m

At middle of inner span : (i.e. at Section - 3)

M_{u3} = 1.5(5.25x4²/24+1.50x4²/12) = 8.25 kN.m

At intermediate support : (i.e. at Section - 4)

M_{u4} =-1.5(5.25x4²/12+1.50x4²/9) =-14.50 kN.m

Absolute maximum B.M. = $M_{u.max}$ = M_{u2} = 16.60 kN.m

7. Check for Concrete Depth :

$M_{ur.max} = R_{u.max}\, b\, d^2$. For slab b= 1000 mm.

$R_{u.max}$ = 2.07 N/mm² for M15-Fe415.

∴ $M_{ur.max}$ = 2.07x1000x120²x10⁻⁶=29.81kN.m>$M_{u.max}$ ∴ O.K.

8. Main Steel : This is obtained at 4 different sections using

$$\text{Required } A_{st} = (0.5f_{ck}/fy)[1-\sqrt{1-4.6\ M_u/(f_{ck}\ bd^2)}]xbd$$

$$A_{st} = (0.5x15/415)[1-\sqrt{1-4.6\ M_u/(15x1000x120^2)}]x1000x120$$

Values of Ast calculated at different sections for moments at those sections are give in Step-11.

9. Check for Serviceability :

Required maximum A_{st} at midspan = A_{st1} = 355 mm²/m

Required maximum p_t = 355x100/(1000x120)

= 0.296 % < assumed 0.4 %. ∴ O.K.

10. Distribution Steel :

For Fe415 grade,A_{st} = 0.12% of b_d=0.12x1000x140/100=168mm²

Provide #8 at 300 mm giving A_{st} = 168mm².

Design of Slab S3 - Design - I : Continued

Step No.	Design Calculations					Reference Note No.
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	355	425	200	366	
	I- New Practice					
	# and s in mm	8-140	8-110	8-250	8-130	(3)
	Provided A_{st} in mm^2	(359)	(457)	(201)	(387)	
	II- Conventional Practice:					
	Diam.& Pitch in mm	8-140	8-280+8-500	8-250	8-500+8-500	
	Provided A_{st} in mm^2	(359)	(280)	(201)	(201)	(4)
	Required extra A_{st} mm^2	---	145	---	165	
	# and s of extra bars	---	8-280	---	8-250	
	Provided extra A_{st} mm^2	---	(179)	---	(201)	

Explanatory Notes on Design of Slab - S3 : Design - I (Continued)

(4) 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 two adjacent spans. However, since areas required in inner and outer spans at midspan are different, spacings of bars in spans on 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 midspan (when no redistribution of moments is done), the bars taken at top by bending of alternate bars are inadequate. Extra steel is, therefore, required to be provided for the shortage. Since,the area of extra steel required is normally very small, it can be of any diameter. However, the pitch of extra steel is normally matched with that of bars bent up from adjacent spans, though theoretically not so necessary. It reduces the labour in laying the bars. Alternatively,it is placed in such a way as to bring uniformity of spacing of bars by placing them in wide gaps. 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, for matching their spacing. However, from requirements of ease of construction, use of different diameters of bars 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 for uniformity of spacing in all spans for ease of construction. This, however, leads to use of 20% to 30% more steel and hence uneconomical and not recommended.

Explanatory Notes to Design of Slab - S3 : Alternate Design - II.

(1) 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 the minimum load is DL alone for which load factor is 0.9. (Refer to Chap. 3 of Authors' Textbook).

(2)According to Cl.21.4.1 of IS Code,maximum load will be on adjacent spans AB and BC only and not on DE. However,theoretically,there should be maximum load on DE also. Taking minimum load on DE, according to Code,causes an error in B.M. by 4 % on unsafe side.

(3)Analysis for determining the design moments can either be done by moment distribution method or by use of standard moment coefficients for standard typical loadings available in design aids if the calculations are to be done manually else moments may be obtained by exact stiffness method when computer is available.

Slab S3 Continued

Alternative Design - II : Using Exact Analysis and Allowing Redistribution of Moments

Step No.	Design Calculations	Reference Note No.

1. to 5 : Same as those in Design - I.
6. Design Moments : These are obtained by exact analysis allowing redistribution of moment. According to exact analysis, different loading cases are required to be considered for obtaining maximum bending moments at outer support, inner supports, outer midspans and inner midspans, neccessitating n+1 number of loading cases where n is number of spans. This is for obtaining the maximum moment diagram or what is known as bending moment envelope. However, when redistribution of moments is done, it necessitates consideration of only two loading cases, one for maximum B.M. at penultimate support and the other for maximum B.M. at intermediate support. (Refer to Ex.3.6-4 Chap.3 of Authorçs Textbook). Every loading arrangement requires adoption of maximum and minimum design loads.

Maximum design load $w_{u1} = 1.5(w_d + w_1)$
$= 1.5(5.25+1.5) = 10.125$ kN/m (1)

Minimum design load $w_{u2} = 0.9w_d = .9x5.25 = 4.725$ kN/m

Let $w_{u3} = w_{u1} - w_{u2} = 10.125 - 4.725 = 5.4$ kN/m

$w_{u2} L^2 = 4.725x4x4 = 75.6$ kN.m

$w_{u3} L^2 = 5.4x4x4 = 86.4$ kN.m

Loading Case-I : For Maximum B.M. at Penultimate Support (at Section 2), the loading is shown to the left below.

Section		1	2	3	4	Ref.
B.M. Coef.						(2)
$\alpha 2$		.077	-.107	.036	-.071	(3)
$\alpha 3$		.072	-.121	.061	-.018	
$M_{u2} = \alpha 2\ w_{u2}\ L^2$	kN.m	5.82	-8.09	2.72	-5.37	
$M_{u3} = \alpha 3\ w_{u3}\ L^2$	kN.m	6.22	-10.45	5.27	-1.55	
$M_{EU} = M_{u2} + M_{u3}$	kN.m	12.04	-18.54	7.99	-6.92	(5)
δM 30% at 2	kN.m	2.78	5.56	2.78		(4)
M_{Du} =	kN.m	14.82	-12.98	10.77	-6.92	

Explanatory Notes and Design of Slab S3 - Design-II.

The former approach is little laborious and time consuming while the third may not be possible to those who have no access to computer. The analysis has, therefore, been done by second approach here. The given loading is divided into two standard loading cases shown in figures to the left. Bending Moment coefficients are taken from Table 2.7, page 26, of Author's Handbook.

(4) Maximum of 30 % redistribution has been done as allowed by Cl.36.1.1(c) of the Code. This means reduction of support moment at 2 by 30 % (Thus, $\delta M_2 = 0.3x18.54 = 5.56$ kN.m) and increasing the midspan moment by $\delta M/2 = 2.78$ kN.m in both adjacent spans. This helps to get span moment greater than support moment thereby avoiding requirement of any extra steel at support.

Design of Slab – S3 : Design – II : Continued

Loading Case-II : For Maximum B.M. at Inner Support
(Section 3-3), the loading is shown to the left below.

Loading	Section		1	2	3	4
	B.M. Coef.	$\alpha 2$	.077	-.107	.036	-.071
		$\alpha 3$	-.018	-.036	.056	-.107
$M_{u2} = \alpha 2\ w_{u2}\ L^2$	kN.m		5.82	-8.09	2.72	-5.37
$M_{u3} = \alpha 3\ w_{u3}\ L^2$	kN.m		-1.55	-3.11	4.84	-9.24
$M_{Eu} = M_{u2} + M_{u3}$	kN.m.		4.27	-11.20	7.56	-14.61
M = 30 % at 4	kN.m.				2.19	4.38
M_{Du} for Case-II	kN.m.		4.27	-11.20	9.75	-10.23
M_{Du} for Case-I	kN.m.		14.82	-12.98	10.77	- 6.92
Final Des.moment M_u	kN.m.		14.82	-12.98	10.77	-10.23
Governing Load Case No.			I	I	I	II

7. Check for Concrete Depth : Same as that in Design - I.
8. Main Steel : It is calculated at each of the four sections by the usual formula for Ast shown earlier in Design I. The calculated values are shown below.

			1	2	3	4	
	M_u	kN.m.	14.82	12.98	10.77	10.23	
	Required Ast	in mm²	380	324	265	251	
I New Practice -	# and s	in mm	8-130	8-150	8-190	8-200	(5)
	Provided Ast	in mm²	(386)	(335)	(265)	(251)	
II Conventional Practice-	# and s	in mm	8-130	8-260+8-380	8-190	8-190	(5)
	Provided Ast	in mm²	(386)	(325)	(265)	(265)	
Or	# and s	in mm	10-190	10-380+8-380	8-190	8-190	(6)
	Provided Ast	in mm²	(413)	(339)	(265)	(265)	(7)

9. Check for Serviceability: Required p_t at middle of outer span = 380x100/(1000x120) < assumed 0.4 % ∴ O.K
10. Distribution Steel : #8 mm at 300 mm as in Design-I.

Explanatory Notes on Design of Slab S3 : Alternative Design-II : Continued

(5) The redistribution of 30 % is helpful for conventional practice of detailing in which steel at support is obtained by bending alternate bars in adjacent spans and taking them to top over supports. Since, steel at middle of inner span is always less than that at outer span, the average steel at penultimate support is less than steel at middle of outer span. Therefore, in such case, it is convenient to have B.M. at support less than that at middle of outer span. This is achieved by 30 % redistribution.

However, for new practice of detailing, it is advisable to equalise steel and hence B.M. at supports and at midspan for economy. For this, reduce the moment at support by a-value = 2/3rd of absolute difference between the support moment and the span moment, and increase the span moment by 1/3rd of this difference. Thus, in above problem, the absolute difference = 18.54-12.04 = 6.50 kN.m. Therefore, reduce support moment by (2/3)x6.50 = 4.33 kN.m and increase the span moment by (1/3)x6.50 = 2.17 kN.m. After redistribution, moments at midspan as well at support will be 12.04+2.17=14.21 kN.m and 18.54-4.33=14.21 kN.m. This requires Ast =358 mm². Provide #8 at 140 mm at both - midspan and support sections instead of #8 at 130 at midspan and #8 at 150 at support section as shown in the Table of calculations. In comparison of the three methods, therefore, #8 at 140 mm has been considered. In this case, #8 at 200 mm is now sufficient at both sections 3 & 4.

Explanatory Notes on Design of Slab S3 - Design - II Continued

(6) When #8 mm bars are adopted in both the spans, spacings of bars in the two spans are different. Cranking of bars having different spacings in adjacent spans leads to non-uniform distribution of steel over support. To avoid this, some designers try to match the spacings in two spans by adopting different bar diameters in the two spans. In the above case, 10 mm bar is used for outer span and 8 mm bar in inner span with spacing same in two spans.

(7) For seviceability check in case of fixed or continuous beams/slabs, required p_t is that at midspan and not at support. Hence, A_{st} at middle of outer span has been taken for serviceability check.

6.2.3 Slab - S3

Alternative Design-III : Design using Approximate Method

Step No.	Design Calculations	Reference	Note No.

1. to 5 : Same as in Design - I.

6. Design Moments : It is common practice with many designers to take the design moment of $w_uL^2/10$ in outer spans and $w_uL^2/12$ in inner span for simplicity. This gives moment and A_{st} as shown below.

W_u = 10.125 kN/m

w_uL^2 = 10.125×4^2 = 162 kN/m

	1	2	3	4	Note No.
B.M. Coefficient α	1/10	1/10	1/12	1/12	
M_u in kN.m	16.2	16.2/14.9*	13.5	13.5	(1)
Required A_{st} in mm².	414	414/376*	337	337	
I New Practice : # and s in mm	8-120	8-120/8-135*	8-150	8-150	
Provided A_{st} in mm².	(419)	(419)/377*	(335)	(335)	
II Conventional Practice: #-s mm	8-120	8-240+8-300*	8-150	8-150	
Provided A_{st} in mm².	(419)	(377)	(335)	(335)	(1)

Explanatory Note for Designed Slab S3 - Design -III :

(1)Values marked with asterisk(*) are for steel at support equal to average of the values at Sections 1 and 3, 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.

Comparison of Designs :

Section			1	2	3	4
I - New Practice :						
Design-I	:# and s	in mm	8-140	8-110	8-250	8-130
	Provided A_{st}	in mm².	(359)	(457)	(201)	(387)
Design-II	:# and s	in mm	8-130	8-150	8-190	8-200
	Provided A_{st}	in mm²	(386)	(335)	(265)	(251)
Design-III	:# and s	in mm	8-120	8-120	8-150	8-150
	Provided A_{st}	in mm²	(419)	(419)	(335)	(335)
II Conventional Practice:						
Design-I	:# and s	in mm	8-140	8-280+8-500	8-250	8-250
	plus extra			+8-280		+8-250
Design-II	:# and s	in mm	8-130	8-260+8-380	8-190	8-190
	Provided A_{st}	in mm²	(386)	(325)	(265)	(265)
	or # and s	in mm	10-190	10-380+8-380	8-190	8-190
	Provided A_{st}	in mm²	(413)	(339)	(265)	(265)
Design-III	:# and s	in mm	8-120	8-240+8-300	8-150	8-150
	Provided A_{st}	in mm².	(419)	(377)	(335)	(335)

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, and with the advent of computers it can be used easily. However, for hand computation, Design-III based on approximate method of analysis is simple. It saves time, computational efforts, and errs on safer side.

6.2.4 Slab - S4

Step No.	Design Calculations	Reference Note No.
1.	Type : Two-way Continuous with Corners restrained.	
2.	Spans: Short Span L_x = 4.5 m, Long Span L_y = 5 m. Aspect ratio L_y/L_x = 5.0/4.5 = 1.11.	
3.	Imposed Load : w_i = 3.25 kN/m² as for Slab S2.	
4.	Trial Depth : In case of a two-way slab, L/d ratio for serviceability criteria is related to short span. Since short span L_x in this case = 4.5 m is greater than 3.5m (even though live load is less than 3 kN/m²), serviceability requirement is governed by Cl.22.2.1.	*23.1 *Note-1. *23.2 *Note-2.
	In case of two-way slabs, the design moments are very small compared to those in one-way slabs, percentage of steel required in two-way slabs is, in general, very low (less than .25% for M15-Fe415). In this case, since even the live load is also small, only 0.2 % steel will be assumed.	
	For p_t = 0.2 %, $\alpha 1$ = 1.6 for Fe415.	Fig.4.4.1
	Basic L/d ratio r_b = 26 for continuous slab.	Sect.4.4.1(b)
	Alowable L/d ratio r_a = 1.6x26	
	Required d = L_x/r_a=4500/(1.6x26)= say 110 mm	
	Assuming d' = 20 mm for Fe415,	
	Required D = 110+20 = 130 mm.	
	However, since adjacent large floor slab S3 for the hall being 140 mm, provide same thickness for ease of casting. It will also reduce area of steel.	
	Effective depth to outer layer of bars d_o = 120mm,	
	Effective depth to inner layer of bars d_i = 110mm.	
5.	Loads: Consider one metre width of slab. w_u = 1.5(25x.14+1.75+1.5) = 1.5x6.75 =10.125 kN/m².	
6.	Design Moments : Boundary Condition No. = 4. Discontinuous on two adjacent edges. $w_u L_x^2$ = 10.125x4.5² = 205 kN.m	Table B-2 (1)

Span	Position	B.M.Coef.(α)	$m_u=\alpha w_u L_x^2$ kN.m
Short	-Support	0.0537	.0537x205 =11.01 kN.m.
	-Midspan	0.0405	.0405x205 = 8.30 kN.m.
Long	-Support	0.047	.047 x205 = 9.64 kN.m.
	-Midspan	0.035	.035 x205 = 7.18 kN.m.

(Reference: Table B-2)

Explanatory Notes on Design of Slab S4 :

(1) It may be noted that the middle panel near the stair is discontinuous at the edge of beam B10, there being staircase well on other side of B10, its boundary condition corresponds to that of Case-4 of Table B-2.

Design of Slab - S4 : Continued

Step No.	Design Calculations	Reference	Note No.

7. Check for Concrete Depth :

$M_{ur.max} = R_{u.max} bd^2$. For slabs, b = 1000 mm
$R_{u.max}$ = 2.07 N/mm² for M15-Fe415
For outer bars, $M_{ur.max} = 2.07 \times 1000 \times 120^2 \times 10^{-6}$
= 29.8 kN.m > 11.01 kN.m. ∴ O.K.
For inner bars, $M_{ur.max} = 2.07 \times 1000 \times 110^2 \times 10^{-6}$
= 25.05 kN.m ∴ O.K.

8. Main Steel : Use equation,

Required $A_{st} = (0.5 f_{ck}/f_y)[1-\sqrt{1-4.6M_u/(f_{ck}\, bd^2)}]\,.bd$

Span Position	M_u kN.m	d mm	Req. A_{st} mm²	Diam. Spacing mm mm	Prov. A_{st} mm²	Note No.
Short-Support	11.01	120	271	#8 - 180	279	
-Midspan	8.30	120	201	#8 - 250	201	(2)
Long -Support	9.64	120	236	#8 - 200	251	
-Midspan	7.18	110	190	#8 - 250	201	(3)

Distribution Steel : Provide #8 at 300 mm as for slab S3 (3)

9. Check for Serviceability : (4)

Required p_t at midspan-short = 201x100/(1000x120) = 0.170 %
Since required $\%p_t$ is less than assumed $\%p_t$, check for serviceability is satisfied. (5)

10. Check for Shear: B = L_y/L_x = 5.0/4.5 = 1.11

(a)(i)Long Edge-Continuous: $V_{u.max} = 1.2xq_u\, L_x[B/(2B+1)]$ Sect.5.2.3
$V_{u.max}$ = 1.2x10.125x4.5x[1.11/(2x1.11+1)] = 1.2x15.71 = 18.85 kN. Step-11.
A_{st1} = 279 mm² p_t = 100x279/(1000x120) = 0.23 %. k = 1.3 Table 4.2(2)

Explanatory Notes for Design of Slab S4 : Continued

(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. 251 mm² for S4 and for S3, it is 387 mm² according to Design -I, 251 mm² according to Design - II, and 335 mm² according to Design - III for new practice of bar detailing. In 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.

(3) The diameter-spacing shown is for new practice of detailing. In conventional practice of bar detailing, steel at support will be obtained by bending alternate bars (#8 at 500 mm) from spans adjacent to common support B11 to give 208 mm² and balance 251-201 = 50 mm² provided by way of extra steel. If #8mm bars are used, just one extra bar is required per metre width (i.e. #8 at 1000mm). If instead of #8, 06mm bar is to be used of mild steel, its required area = 50x415/250 = 83 mm². Provide ϕ6 mm at 600 mm giving area 84 mm².

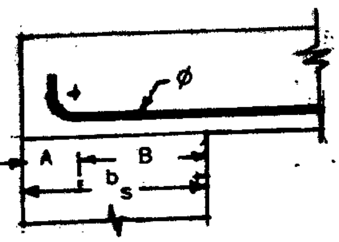
In practice, many times, the steel requied at support i.e. #8 at 200 mm is provided at midspan also so that the same steel can be obtained at supports by cranking of alternate bars from two adjacent spans abd accordingly #8 at 200mm have been provided . For details see schedule of slabs.

(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.

(5) For serviceability, p_t shall be the steel at middle of short span.

Design of Slab – S4 Continued

Step No.	Design Calculations	Reference Note No.
10.	Check for Shear : Continued	
	T_{uc} = 0.338 N/mm², V_{uc}=k T_{uc} bd. k=1.3 for D=140mm	Table 4.2
	V_{uc} = 1.3x0.338x1000x120/1000=52.73kN>18.85kN. O.K.	Sect.4.2.4
	(a)(ii)Long Edge-Discontinuous:$V_{u.max}$=.9x15.71=14.14 kN	Sect.5.2.3
	A_{stx} = 201 mm² at midspan. Assuming 50% bent up to resist moment due to partial fixity,	
	A_{st1} = 100 mm² P_t = 100x100/(1000x120) = 0.084 %.	
	T_{uc} = 0.218 N/mm².k=1.3 (See Sect.4.2.4)	Eq.4.2.3
	V_{uc} = 1.3x0.218x1000x120/1000=34 kN>14.14kN ∴O.K.	
	(b)(i)Short Edge-Continuous : $V_{u.max}$ = 1.2q_u L_x/3	Sect.5.2.3
	$V_{u.max}$= 1.2x10.125x4.5/3=1.2x15.19 = 18.23 kN.	
	A_{st1} = 251 mm². ∴ V_{uc}=50.85 kN > 18.23 kN.∴O.K.	Step-10(a)(i)above
	(b)(ii)Short Edge-Discontinuous:$V_{u.max}$=.9x15.19=13.67kN	Sect.5.2.3
	A_{sty} = 201 mm² at midspan. assuming 50% bent up to resist moment due to partial fixity,	
	A_{st1} = 100 mm² as for Long edge-Discontinuous.	
	T_{uc} = 0.218 N/mm², see Step 10(a)(ii) above.	
	V_{uc} = 1.3x0.218x1000x110/1000 = 31.17 kN > 13.67 kN. ∴ O.K.	
11.	Check for Development Length:	
	(a)(i) Long Edge – Continuous : Required L_d=57x8=456mm	
	Available L_d=L/4 of slab S3 =4000/4=1000 mm.∴O.K.	
	a(ii) Long Edge-Discontinuous : L_d = 57x8 = 456 mm.	
	Assuming 50% bars bent up, M1 = 8.3/2 = 4.15 kN.m. V = $V_{u.max}$ =14.14 kN.	
	Now, L_{ex}=>L_d-1.3M1/V=456-1.3x4.15 x1000/14.14 = 75 mm. = 75+230/2 = 190 mm.from inner face of support.	
	Straight length available inside inner$_o$ support= B =b_s-A=230-(5x8+25)=165mm, Using 90 bend, available anchorage length = 8φ+165=64+165= 229mm>165mm. ∴O.K.	
	(b)(i) Short Edge – Continuous: Required L_d=57x8=456mm	
	Available L_d = L_y/4 = 5000/4 = 1250 mm. ∴ O.K.	
	b(ii) Short Edge-Discontinuous : L_d = 57x8 = 456 mm.	
	Assuming 50% bars bent up , M1 = 7.18/2 = 3.59 kN.m. V = $V_{u.max}$ = 13.67 kN.	
	Now, L_{ex} => L_d-1.3M1/V=456-1.3x3.59x1000/13.67=114 mm. = 114+230/2=229 mm from inner face of support	
	Straight length available inside inner support= B =b_s-A=230-(5x8+25)=165mm. Using 90° bend, available anchorage length =8φ+165 = 229mm.= L_{ex} ∴ O.K.	
12.	Torsion Steel :	
	(a) At corners near columns C12 and C20, since slab is discontinuous over both the edges, full torsion steel equal to 0.75A_{stx} = 0.75x201 = 150 mm² will be required in both directions at right angles in each of the two meshes, one at top and other at bottom for a length = L_x/5 = 4500/5 = 900 mm.	(6)
	(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.	(5)



b_s = Breadth of Support
A = > of (2φ or 25mm) + 5φ
Hook Anchorage value = 8φ

Explanatory Notes for Design of Slab S4 : Continued :

(6) Torsion steel is provided at the corners in one of the following ways.
(i) Two meshes of size 900mm x 900mm, with #8 bars at 300mm (giving A_{st} = 167 mm^2 > 0.75x201) laid othogonally in each of the two layers, shall be provided at each corner making use of whatever main bars are available at top or at bottom in the region common to torsion region $L_x/5$ and middle strips (i.e. between $L_x/5$ and $L_x/8$ in one direction and between $L_x/5$ and $L_y/8$ in the other direction).
(ii) Provide main steel in both the spans - long and short -throughout the width (instead of providing only in the middle 3/4th width) and continue all bars at bottom, within a strip of width $L_x/5$ each way, right upto the slab end and then bending them back through 180o 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 another direction in each of the two meshes at top and bottom instead of $0.75A_{stx}$ 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.

6.2.5 Design of Slab - S5 : Cap Slab Over Staircase :

Step No.	Design Calculations	Reference Note No.
1. Type :	Two-way, Simply Supported, Single Panel, Corners free to lift.	(1)
2. Spans:	Short L_x = 2.5 m, Long Span L_y = 3.26 m. Aspect ratio = L_y/L_x = 3.26/2.5 = 1.3	(2)
3. Imposed Load :	Live Load = 0.75 kN/m^2 (no access) Roof finish= 1.75 kN/m^2 w_1 = 2.50 kN/m^2	
4. Trial	Depth : Since short span is less than 3.5 metres and live load is less than 3kN/m^2, depth will be governed by serviceability requirements of Note-2 of Cl.23.1. Allowable L/D ratio = 35 for simply supported slab for steel of grade Fe250 and 0.8x35 = 28 for steel of grade Fe415. ∴ required 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_i = 80-10 = 70 mm.	
5. Load :	w_u = 1.5(25x0.1+2.5) = 1.5 x 5.0 = 7.5 kN/m	

Explanatory Notes for Design of Slab - S5

(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.

(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 width on one side = 2250 + 1010 = 3260 mm = 3.26 metres.

Design of Slab – S5 : Continued

Step No.	Design Calculations	Reference	Note No.

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	0.093x46.875 = 4.36 kN.m	Table B-2
Long	0.055	0.055x46.875 = 2.58 kN.m	

7. Check for Concrete Depth :

$M_{ur.max} = R_{u.max}\ bd^2$. For slabs, b = 1000 mm.

$R_{u.max} = 2.07$ N/mm² for M15-Fe415.

$\therefore M_{ur.max} = 2.07 \times 1000 \times 80^2 \times 10^{-6} = 13.25$ kN.m for short span > 4.36 kN.m. $\therefore$ O.K.

$M_{ur.max} = 2.07 \times 1000 \times 70^2 \times 10^{-6} = 10.14$ kN.m for long span > 2.58 kN.m. $\therefore$ O.K.

8. Main Steel:This is obtained for both spans using formula

$$A_{st} = (0.5f_{ck}/f_y)[1 - \sqrt{1 - 4.6M_u/(f_{ck}\ bd^2)}]\ bd$$

Span	M_u kN.m.	d mm	Req.A_{st} mm²	Diam. mm	Spacing mm	Prov.A_{st} mm²	
Short	4.36	80	160	#8	240	209	(3)
Long	2.58	70	107	#8	210	239	

$A_{st.min}$ for Fe415 = 0.12 % bD = .12x1000x100/100 = 120 mm²

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 : Continued

(3) The steel required for resisting moment for both the spans is too less. The spacing is governed by maximum permissible spacing of 3d. Theoretically,even 6mm bars of grade Fe250 would have been sufficient. However,they are not used for main steel in practice as they are not economical ($A_{st.req.}$ = 266 mm² for short span and 178 mm² for long span). Besides, 6mm bars do not have any rigidity for working and hence not favoured as main steel.

Explanatory Notes for Staircase Design:

(1) 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 to 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, one metre strip of slab S2 which acts as supporting wide beam for the flight will be required to be designed to carry the load from flight-II besides the load of the cross wall at the end of flight-II. This will require very heavy reinforcement in that strip since the depth of slab is always less than the depth of beam. The solution is 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 x 3260 mm to 2270 mm x 4270 mm resulting in increase in length of side long walls and of size of slab S5. The 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.

6.2.6 Design of Stair

Step No.	Design Calculations	Reference Note No.
Data :	Staircase room 4270mmx 2270 mm.	
	Floor to floor height H = 3.2 meters = 3200 mm	Table 2.2
	Live Load for public buildings = 5 kN/m^2.	
Functional Design:	Let Rise R=160 mm and Tread T= 250 mm.	
	Sec ϕ = $\sqrt{250^2 + 160^2}/250$ = 1.187	
	No. of risers required = H/R = 3200/160 = 20	
	No. of risers ineach of the two flights = 10	
	No. of Treads per flight = 10-1 = 9.	
	Going = 250x9 = 2250 mm. Width of landing at end = 3030-2250= 780 mm	
	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 continuous over 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 metres horizontally = 3260 mm.	
3. Imposed Load :	Live Load = 5.0 kN/m^2. Finish = 1.0 kN/m^2-assumed	
4. Trial Depth :	(Waist thickness of slab)	
	Basic L/d ratio = 20 for simply supported slab.	
	Assumin p_t = 0.45 % since the load is heavy on stairs.	
	Modification factor α = 1.23 for steel of grade Fe415	Fig.4.4.1
	Allowable L/d ratio r_a = 1.23x20 = 24.6.	
	Required effective depth d=L/r_a=3260/24.6=say 140 mm.	
	Assuming d'=20 mm for Fe415, provide D=140+20=160 mm.	
	(Normally, trial depth for stairs may be taken between L/20 to L/25).	
5. Loads :	Self Weight w_d = 25DxSec ϕ = 25x.16x1.187 = 4.75 kN/m^2	
	Weight of steps=25R/2=25x.160/2= 2.00 kN/m^2	
	Imposed load = 1.00 + 5.00 = 6.00 kN/m^2	
	Total working load w =12.75 kN/m^2	
	Total design load w_u=1.5x12.75 =19.125kN/m^2	
6. Design Moment :	$M_{u.max} = w_u L^2/8$ = $19.125 \times 3.26^2/8 = 25.41$ kN.m	(2)

Explanatory Notes on Design of Staircase Continued

(2) Theoretically, load on landing portion 1010 mm long of the flight is only the weight of horizontal slab excluding the weight of step =1.5(25x.16+1+5) = 15 kN/m^2 instead of 19.125 kN/m^2. The load distribution along the length of the flight is non-uniform 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 by hardly 5 % to 10 % and in support moment only upto 5 %. This saving is not enough to justify the cost of extra time and efforts required for rigorous calculations. Simplicity accompanied by additional safety is always preffered to apparent marginal economy in practical design.

Design of Staircase Continued

Step No.	Design Calculations	Reference Note No.

7\. Check for Concrete Depth : $M_{ur.max} = 2.07 \times 1000 \times 140^2 \times 10^{-6}$
$= 40.57$ kN.m > $M_{u.max}$ (=25.4 kN.m). ∴ O.K.

8\. Main Steel:Required
$p_t = (0.5 \times 15/415)[1 - \sqrt{1 - 4.6 \times 25.41 \times 10^6 / (15 \times 1000 \times 140^2)}].100$
$= 0.405$ % <assumed 0.45 %. ∴ O.K. for serviceability.
$A_{st} = 0.405 \times 1000 \times 140/100 = 567$ mm²
Provide #10 at 130 mm giving A_{st} = 604 mm².

9\. Distribution Steel : For Fe415, $p_{t.min}$ = 0.12 %.
∴ $A_{st} = 0.12 \times 1000 \times 160/100$ = 192 mm².
Provide #8 at 260 mm giving Ast = 193 mm².

Design of Flight-II

1\. Type : One-way, Simply supported at one end (B26) and Continuous at the other end (B27).

2\. Span : 3.26 metres horizontally. (3260 mm)

3\. Imposed Load: w_i = 6 kN/m² as for Flight-I.

4\. Trial Depth : Since this flight is continuous at one end, the design moment will be $w_u L^2/10$ instead of $w_u L^2/8$ (in case of flight-I) and hence required depth for this flight will be less than that required for flight-I. However, since two different depths cannot be provided for two flights which have common landing slab, provide depth same as that required for flight-I. ∴ D = 160 mm.

5\. Load : Same as that of flight-I. w_u = 19.125 kN/m.

6\. Design Moment : $M_{u.max} = 19.125 \times 3.26^2/10 = 20.33$ kN.m.

7\. Check for Concrete Depth : Since $M_{u.max}$ is less than that of flight-I, and the depth is safe for flight-I, it is safe for flight-II as well.

8\. Main Steel : Required
$p_t = (0.5 \times 15/415)/[1 - \sqrt{1 - 4.6 \times 20.33 \times 10^6 / ((15 \times 1000 \times 140^2)}] \times 100$
$= 0.315$ % < 0.45% assumed for trial depth.
∴ O.K. for serviceability.
Required $A_{st} = 0.315 \times 1000 \times 140/100 = 441$ mm².
Provide #10 at 175 mm giving Ast = 448 mm².

9\. Distribution Steel : Same as that for flight-I i.e.# 8 at 260 mm.

Table 6.2.7 : Schedule of Slabs

Slab	Depth mm	Reinforcement Short Span Diam Spacing		Reinforcement Long Span Diam Spacing		Span m	Remarks Type	End Condition
S1	110	#8	270	ø 6	160	1.0	One-way	Cantilever
S2	110	#8	200	ø 6	160	2.5	One-way	Simply Supp.
S3-outer	140	#8	120	#8	300	4.0	One-way	Continuous
-inner	140	#8	150	#8	300	4.0	One-way	Continuous
S4	140	#8	200	#8	250	Short 4.5	Two-way	Continuous
S5	100	#8	240	#8	210	Short 2.5	Two-way	Simply Supp.
Stair Fl.I	160	#10	130	#8	260	3.26	One-way	Simply Supp.
" Fl.II	160	#10	175	#8	260	3.26	One-way	Continuous

6.3 DESIGN OF BEAMS

6.3.1 Categoring and Grouping of Beams

As explained in Sect. 5.3.2, 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, since the analysis of a 5 span continuous beam B1-B2-B3-B4-B5, B6-B7-B8-B9-B10, and B12-B13-B14-B15-B16 with unequal spans and unequal 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) left end span beam (e.g.B1,B6,and B12) as independent beam, simply supported at one 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) right end beam (e.g. B5,B10,B16) to be continuous at one end and simply supported at the other. Eventhough, spans and end conditions of these beams are same, beam B16 does not carry any point load but only triangular load which can be converted into equivalent UD load. It will be considered in a separate group while beams B5 and B10 will be categorised separately as they carry point loads as well. The beams B17 and B18 cannot be treated as separate beams because the difference between the spans of the two is very large. It will be treated as a two-span continuous beam with unequal spans.

Beams B22-B23 and B24-B25 are both continuous beams with two equal spans and spans for both of them are also same. 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 and further the loading and section type as shown in table below.

Catagory No.	EC No.	Group No.	Beam Mark	Load Type	Section Type	Span m	*Code No.	Explanatory Note No.
I	1	(a)	B26	UDL	Rectangular	2.5	1111	1,2,3
		(b)	B27	UDL	Rectangular	2.5	1112	3
		(c)	B11	UDL	Flanged	4.5	1123	4
		(d)	B19,B20,B21	UDL	Flanged	10.0	1124	4
II	2	(a)	B1	UDL	Rectangular	4.0	1211	1,5
		(b)	B6	UDL	Rectangular	4.0	1212	1,5
		(c)	B12	UDL	Rectangular	4.0	1213	1,5
		(d)	B16	UDL	Flanged	4.5	1214	1
III	3	(a)	B2, B3, B4	UDL	Rectangular	4.0	1311	1,6
		(b)	B7, B8, B9	UDL	Rectangular	4.0	1312	1,6
		(c)	B13,B14,B15	UDL	Rectangular	4.0	1313	1,6
IV	4	(a)	B22-B23	UDL	Flanged	5.0	2421	7
		(b)	B24-B25	UDL	Flanged	5.0	2422	8
		(c)	B17-B18	UDL	Flanged	10/2.5	2423	9
		(d)	B5	UDL+PTL	Rectangular	4.5	1414	10
		(e)	B10	UDL+PTL	Flanged	4.5	1425	10

Code numbering has been done so as to facilitate computer solution. In the code number
- first digit refers to Number of spans,
- second digit refers to End Condition No.,
- third digit refers to Type of Section - 1 for rectangular, and - 2 for flanged,
- fourth digit referes to Serial No.of the beam in that category.

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 :

(1) All beams bearing this Note No.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 (d), since spans here are small and loading is also not very heavy, there is little advantage in utilizing the flange action. Therefore, these beams are designed treating them as rectangular beams. (The validity of this statement has been shown by calculations at the end of design of beams B7).

(2) Beam B26 is at midlanding. 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.

(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 that 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 parallel to 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 across B27 to avoid cracking, a load over a triangular area with central ordinate $= q_u L_x/4$ is assumed to be transferred to beam B27, where qu is the intensity of load on S2 per sq.m and L_x is the span of S2.

(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 can not be designed as rectangular beam like B26 or B27.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 B20 being identical in all respects, they have been grouped together.

(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 triangular load from S3. Eventhough, slab S3 spans parallel to B6, a load over a triangular portion having central ordinate equal to $q_uL_x/4$ 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 as 2(a).

(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.

(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.

(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 transversly. It is, therefore, designed as a 2 span continuous beam acting as rectangular beam at intermediate support and L beam at midspan.

(9) Beam B17 is continuous with beam B18. However, since span and the load on beam B17 is very large compared to that on B18,no fixity (rotation restraint) will be obtained at continuous end of B17. Therefore, beam B17 can practically be assumed to be simply supported at both ends, though steel will be required to be provided at top of intermediate support common to both beams to resist negative moment that may develop over there due to continuity.

(10) There is no slab at floor level connecting B5 because of stair room except for 1 m portion of landing slab. The beam, therefore, acts as rectangular beam with UDL of wall above and self weight plus load over the portion of S2 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 can, therefore, be designed as L beam.

6.3.2 Common Data for Design of Beams

(1) Trial Section : The basic principle in assuming trial sections is that variety of sections should be minimum to facilitate centering. This reduces the cost also. 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 (750 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 upto 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. 35 mm is considered adequate for beams of size 230x380 mm and 65 mm for section 230x750 mm.

(3) Effective depth d = D - d' = 380 - 35 = 345 mm for beams 380 mm deep,
= 750 - 65 = 685 mm for 750 mm deep beams.

(4) Depth of flange D_f = Depth of slab supported. These are as follows:
Slabs S1 and S2 - 110 mm; Slabs S3 and S4 - 140 mm; Slab S5 - 100 mm.
Stair slab - 160 mm.

(5) Loads: (a) Loads from roof/floor slab (self weight+finish+live load):

Slab	S1	S2	S3	S4	S5	Stair
Total Working load	5.25	6.00	6.75	6.75	5.00	12.75 kN/m².

Total ultimate load = 1.5 times the working load.

(b) Wall : 250 mm thick 1.0 m high. w_w = 20X.25X1.0 = 5 kN/m
250 mm thick 3.2 m high w_w = 20X.25X3.2 = 16 kN/m
(Wall thickness includes thickness of plaster)
Grill 75 mm thick 1.0 m high *2/3X25X.075X1 =1.25kN/m.
(* 1/3 reduction for openings)

(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. D - total depth of beam. Weight of beam adopted in this design is as follows:
230x380 mm 25X.23X(.38-.1) = 1.60 kN/m

(6) Design Moment Coefficients : Allowing Redistribution of Moments.

End Con. No.			Support	Midspan
1	Beam Simply Supported at both ends:	UD Load	0	1/8
		Central Point Load	0	1/4
2	Beam Simply Supported at one end and Continuous at the other :	UD Load	1/10	1/10
		Central Point Load	1/6	1/7
3	Beam Continuous at both ends :	UD load	1/12	1/12
		Central Point Load	1/8	1/8

(7) Design Constants for a balanced section : For M15-Fe 415

% Redistribution of moments	0	20	30	Reference
	$k_{u.max}$ = 0.48,	$k_{u.limit}$ = 0.40	0.30	Eq.4.1-8
	$R_{u.max}$ = 2.07,	$R_{u.limit}$ = 1.80	1.42	Eq.4.1-9a
	% $p_{t.max}$ = 0.72,	$p_{t.limit}$ = 0.60	0.45	Eq.4.1-10a
For beam 230x380mm,	$M_{ur.max}$ = 56.67,	$M_{ur.limit}$ = 49.27	38.87	kN.m.

(8) Minimum Stirrups : For b = 230 mm, for 6 mm diameter two legged M.S. stirrups (A_{sv} = 2x28 = 56 mm²), required spacing is
$s = A_{sv}\, f_y/(0.4b)$ = 56x250/(0.4x230) = say 150 mm.
Thus minimum stirrups are Ø6 at 150 mm for b = 230 mm.

(9) Shear Resistance of Minimum Stirrups : $V_{usv.min}$ = 0.35bd kN.
For beam 230x380 mm (d=345mm), $V_{usv.min}$ = 0.35x230x345/1000 = 27.77 kN.
For beam 230x750 mm (d=685mm), $V_{usv.min}$ = 0.35x230x685/1000 = 55.14 kN.

(10) Required Development length for M15-Fe415 is L_d = 57 #.
For # = 12 mm, required L_d = 57x12 = 684 mm,
For # = 16 mm, required L_d = 57x16 = 912 mm.
Width of support bs is assumed = 230 mm.
Required $L_{ex} = L_d/3 - b_s/2$ = 684/3 - 230/2 = 113 mm for # = 12 mm, and
L_{ex} = 912/3 - 230/2 = 189 mm for # = 16 mm.

Table - 6.1 : Properties of Beam Section
230 mm x 380 mm (d = 345 mm) used in This Design

No.Diam. mm	A_{st} mm²	p_t %	M_{ur} kN.m	V_{uc} kN	$V_{usv.min}$ kN	$V_{ur.min}$ kN	Min.Stir Ø-s(mm)	α1	Req.L_d mm	L_{ex} mm
2-#12	226	0.285	25.93	29.44	27.77	57.21	Ø6-150	1.43	684	113
3-#12	339	0.427	37.23	34.76	27.77	62.53	Ø6-150	1.25	684	113
2-#16	402	0.507	43.05	36.72	27.77	64.49	Ø6-150	1.17	912	189
4-#12	452	0.570	47.43	38.72	27.77	66.49	Ø6-150	1.13	684	113
5-#12	565	0.712	56.51	42.24	27.77	70.01	Ø6-150	1.06	684	113

6.3.3 Design of Typical Beams with Detailed Theoretical Calculations

The beams have been designed according to the procedure given in Sect.5.3.3

Category -I : Beams Simply Supported at Both Ends :

Beam - B26 :

Step No. Design Calculations — Reference

1. Beam Mark : B26 at mid-landing level.
2. End Condi.: Simply Supported at Both Ends. EC = 1.
3. Span : L = 2.5 metres = 2500 mm.
4. Section : Assumed b =230mm, D =380mm, d_c=35mm, d =345mm.
5. Loads : Supported slabs
   - Left : Landing of stair flights - I and II. q_1 = 12.75 kN/m², L = 3.26 m.
   - Right : Nil.

   Load from : Stair Flights-I & II 12.75x3.26/2 = 20.8 kN/m
   Grill 4.8m high, 4.8x1.25 = 6.0 kN/m — Sect.6.3.2(5)
   Self, 25x0.23x(0.38-0.1) = 1.6 kN/m
   Total Working load w = 28.4 kN/m
   Design Ultimate load w_u = 1.5x28.4 = 42.6 kN/m

6. (a) Design Moment : $M_{u.max} = w_u.L^2/8 = 42.6\text{x}2.5^2/8 = 33.28$ kN.m
   (b) Maximum Ultimate Moment of Resistance of Rect. Section,
   $M_{ur.max} = R_{u.max}\ bd^2$. For M15-Fe415, $R_{u.max}$=2.07N/mm²
   $= 2.07\text{x}230\text{x}345^2\text{x}10^{-6}$ =56.67 kN.m > $M_{u.max}$.
   ∴ The beam is singly reinforced and the section is under-reinforced.
7. Main Steel:
   (a) Req.$A_{st} = (0.5f_{ck}/f_y)[1-\sqrt{1-4.6\ M_u/(f_{ck}\ bd^2)}]\ bd$

   $=(0.5\text{X}15/415)[1-\sqrt{1-4.6\text{x}33.28\text{x}10^6/(15\text{x}230\text{x}345^2)}]$ x230X345 = 298 mm²

   Alternatively, $M_{u.max}/M_{ur.max}$=33.38/56.67=0.59 for which
   j_u = 0.892 — 4.1.2(d)
   $A_{st} = M_{u.max}/(0.87f_y\ j_u d)$
   $= 33.28\text{x}10^6/(0.87\text{x}415\text{x}0.892\text{x}345)$ = 299 mm².
   (b) No.Dia. : Provide 3-#12 (A_{st}=3x113=339 mm²).
   Provide Bottom St. 2-#12 + Bottom Bt. 1-#12.
   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_1xØ+(n_1+1) x25, n_1 = 3
   = 3x12 + 4x25 = 136 mm < 230 mm provided. ∴O.K.
   (e) Check for Effective Cover d' : Required
   d_c = Clear cover+#/2 = 25+12/2 = 31 mm < 35 mm ∴O.K.
8. Design for Shear:
   $V_{u.max}$ = w_u L/2 = 42.6x2.5/2 = 53.25 kN.
   V_{uc} = τ_{uc}bd , τ_{uc} depends upon p_t% at support.
   P_t = 100A_{st1}/bd where A_{st1} is area of tension steel at support
   A_{st1} = 2 bars of #12mm = 2X113 = 226 mm²
   P_t = 100x226/(230x345) = 0.285 %
   τ_{uc} = .35+(.285-.25)/(.30-.25)X(.38-.35)= .371 N/mm² — Table-4.2

Design of Beam B26 : Continued Reference

V_{uc} = 0.371x230x345/1000= 29.44 kN.
$V_{usv.min}$= 0.35 bd = 0.35x230x345/1000 = 27.77 kN
$V_{ur.min}$ = 29.44+27.77=57.21 kN>$V_{u.max}$ (= 53.25 kN)
∴ Minimum stirrups are sufficient.
Provide ∅6mm Fe250 2 legged stirrups (A_{st}= 56 mm^2)
Pitch s = A_{sv} f_y/(0.4b) = 56x250/(.4x230) = 152 mm
say 150 mm <.75x345 and < 450 mm ∴ O.K.
∴ Provide ∅6mm at 150 mm

9. Check for Development Length :
Required L_d = 57 ∅ = 57x12 = 684 mm
Available L_d= 1.3M_1/V + L_{ex}
M_1 = M.R. of A_{st1} at support.A_{st1}= 2-#12=226 mm^2.
= 0.87f_y A_{st} d[1- (A_{st} f_y)/(f_{ck} bd)]
= 0.87x415x226x345[1-(415x226)/(15x230x345)]x10^{-6}
= 25.93 kN.m
V = $V_{u.max}$ = 53.25 kN.
L_{ex} = L_d/3-b_s/2 where, b_s = width of support
Assuming b_s = 230 mm, L_{ex} = 684/3-230/2 = 113 mm
Available L_d= 1.3x25.93x1000/53.25 +113 = 633+113
= 746 mm > 684 mm ∴ O.K.

10. Check for Serviceability (Deflection):
Required p_t%= Required A_{st}x100/(bd)
= 298x100/(230X345)= 0.375 % say 0.38 %
Modification factor α_1 = 1.30 for Fe415. Fig.4.4.1
Basic L/d ratio r_b = 20 for S.S.beams. 4.4.1(b)
Allowable L/d ratio r_a = 20x1.30
Required d = L/r_a = 2500/(20x1.30) = 96 mm
< 345 mm provided ∴ O.K.

Hint: Modification factor α_1=1 for p_t=0.867% Normally in limit state design, required p_t% hardly ever exceeds 0.867%. Therefore, it is sufficient to verify that p_t < .867%, in which case proceed as follows. Actual L/d = 2500/345 < basic L/d ratio (r_b=20.) ∴ OK.

Comments: It may be felt that for the section 230x380mm provided Mur.max(=56.67kN.m) is much larger than Mu.max(=33.28kN.m) and a reduced section could be economical. Now,for an ultimate moment of 33.28kN.m, the depth of the balanced section works out to 264 mm and hence the minimum section that can be provided will be 230x300mm.For this section,the area of steel required is 433mm^2. Now, if 4-#12mm giving area of 452mm^2 is provided,the section becomes over-reinforced, which is not permitted by the Code. As such adopted under-reinforced section is considered to be appropriate.

Beam – B27:

Step No. Design Calculations

1. Beam Mark : B27
2. End Condi.: Simply Supported at Both Ends, EC = 1.
3. Span L : 2.5 metres = 2500 mm.
4. Section : Assumed b = 230mm, D = 380mm, d'= 35mm, d = 345 mm.
5. Loads :

On left half span length carrying stair flight-II :
Slab Left : S2,D_f = 110 mm, q_1 = 6.00 kN/m^2,L_x = 2.5 m.
Slab load : Triangular,w_{s1}=w_{eqb1} = (q_1 L_x/6) 5.2.2(11)(c)
w_{s1}= 6x2.5/6 = 2.5 kN/m
Slab Right: stair flight-II, q_2 =12.75 kN/m^2,L_x = 3.26 m.
Slab load : UDL w_{s2}=12.75x3.26/2=20.78 kN/m
Wall load : Nil w_w = 0.
Self-beam : w_b = 1.6 kN/m 6.3.2(5)(c)
Total working load w = (slab left+slab right+wall+self)
w = (2.5+20.78+1.6)= 24.88 kN/m

Design of Beam B27 Continued — References

On right half span length without stair flight but with wall carrying cap slab

Slab Left : S2, w_{s1} = 2.5 kN/m;
Slab Right: Nil. w_{s2} = 0
Wall load : 3.2 m high, w_w =5x3.2 =16.0 kN/m — 6.3.2(5)(b)
Slab above wall:S5, L_x=2.5m, q=5kN/m².
Slab load : Triangular for two-way.
$w_{eqb}=qL_x/3= w_{s3}$ =5x2.5/3= 4.17 kN/m — 5.2.3(11)
Self-beam : w_b = 1.6 kN/m
Total working load w =<u>24.27</u> kN/m

For simplicity, greater of the two loads is considered over entire span. ∴ w = 24.88 kN/m and w_u=1.5x24.88=<u>37.32</u> kN/m
For shear , w_{us} = 1.5(1.6+20.78+6x2.5/8) =36.38 kN/m.

6 (a) Design Moment: $M_{u.max} = w_u L^2/8$ = 37.32x2.5²/8 =29.16 kN.m
(b) $M_{ur.max}$=56.67 kN.m (calculated earlier for beam B26) — 6.3.2(7)
> 29.16 kN.m. ∴ Section shall be under-reinforced.

7. Main Steel: Required
A_{st} = (0.5x15/415)[1-√(1-4.6x29.16x10⁶ /(15x230x345²))] x230x345 = 257 mm².
Provide <u>3 - #12</u> in 1 row. nl = 3. A_{st} = 339 mm².
Bottom Bt. 1 - #12, Bottom St. 2-#12,
Top St. 2 - #10 as anchor bars.
Check for width and cover need not be repeated since it is same as that for Beam-B26.

8. Design for Shear :
$V_{u.max} = w_u L/2$ = 36.38x2.5/2 = <u>45.48</u> kN. *
A_{st1} = 2 - #12 mm,
∴ $V_{ur.min}$ = 57.21 kN > $V_{u.max}$ (=45.48 kN.) — Table 6.1
∴ Provide Minimum stirrups. <u>Ø6</u> <u>at</u> <u>150</u> mm as in Beam B26.

* **Remarks** : Rigorously, w_u for shear on beams carrying triangular loads consist of w_{eqb} instead of w_{eqb} as taken in wu above. However, this refinement may be omitted when difference between the two is small and when $V_{ur.min} > V_{u.max}$.

9. Check for Development Length :
Maximum bar diameter # = 12 mm. Required L_d = 684 mm. — Table 6.1
Available $L_d = 1.3M_1/V+L_{ex}$. $V=V_{u.max}$ = 45.48 kN.
A_{st1} = 2-#12 for which $M_1=M_{ur}$= 25.93 kN.m, L_{ex} = 113mm — Table 6.1
Available L_d = 1.3x25.93x1000/45.48+113= 854mm ∴ O.K.

10. Check for Serviceability: Actual L/d ratio = 2500/345 < 20.
Allwable L/d ratio > 20 since α_1 > 1. ∴ O.K. — Table 6.1

Beam - B11: : ***Design in a brief Format***

Step No. Design Calculations
1.Type : Simply Supported at Both Ends. EC = 1.
2.Span : 4.5 metres = 4500 mm.
3.Section: 230x380mm, d=345mm, D_f = 140 mm (Slab- S4).
4.Loads : Slab S4(Tri.) from both sides + self.
q = 6.75 kN/m², L_x = 4.5 m.
$w_u = 1.5(2xw_{eqb} + w_b) = 1.5(2qL_x/3 + w_b)$
= 1.5(2x6.75x4.5/3 + 1.6) = <u>32.78</u> kN/m
For shear wus.= 1.5(2x6.75x4.5/4 + 1.6) = 25.18 kN/m

Reference

5. Design Moment : $M_{u.max} = 32.78x4.5^2/8 = 82.97$ kN.m
Beam acts as T beam at mid span (Slab S4 is on both sides).
It will now be decided whether it is necessary to design it as flanged beam.
$M_{ur.max} = 56.67$ kN.m for a rectangular section < $M_{u.max}$. — Table 6.1
.'. Assistance of the flange will have to be taken.
.'. The beam will be designed as T beam.
$b_f = L_o/6 + b_w + 6D_f$ for T-beams.
L_o = L for simply supported beams = 4500 mm.
$b_f = 4500/6 + 230 + 6x140 = 1820$ mm.
It will now be verified whether the neutral axis lies in flange or web.
For $x_u = D_f$, $M_{ur1} = 0.36 f_{ck}\, b_f\, D_f\,(d - 0.42D_f)$
$M_{ur1} = 0.36x15x1820x140(345-0.42x140)x10^{-6}$
$= 393.8$ kN.m > $M_{u.max}$.
.'. $x_u < D_f$ and beam acts as rectangular beam with $b=b_f$.

6. Required $A_{st} = (0.5x15/415)[1-\sqrt{1-4.6x82.97x10^6/(15x1820x345^2)}]$ x1820x345 = 687 mm².
Provide 4-#16 mm in 1 row. Provided A_{st} = 804 mm².
Provide Bottom St. 4-#16. No bent bars,
Provide Top St. 2-#12 as anchor bars.

7. Design for Shear : $V_{u.max} = w_u L/2 = 25.18x4.5/2 = 56.66$ kN.
$A_{st1} = A_{st} = 804$ mm². $p_t = 100x804/(230x345) = 1.01\%$.
$\tau_{uc} = 0.602$ N/mm² — Table 4.2
$V_{uc} = 0.602x230x345/1000 = 47.77$ kN.
$V_{usv.min} = 27.77$ kN — Table 6.1
$V_{ur.min} = 47.77+27.77 = 75.54$ kN > $V_{u.max}$ (= 56.66 kN).
.'. Minimum stirrups are sufficients. Provide Ø6-150mm. — Table 6.1

8. Check for Development : Required L_d = 912 mm for 16 mm bar — Table-6.1
$M_1 = M_{ur} = 0.87x415x804x345[1-(804x415)/(15x1820x345)]x10^{-6}$
$= 96.60$ kN.m
Available L_{ex} = 189 mm for Ø = 16 mm — Table-6.1
Available $L_d = 1.3M_1/V_{u.max} + L_{ex}$ — Eq.4.3.2
= (1.3x96.60/56.66)x1000 + 189 = 2405 mm. .'. O.K.

9. Check for Serviceability :
Actual L/d ratio = 4500/345 < 20. Basic L/d ratio = 20. — Eq.4.4.1(b)
p_t = 100X804/(1820X345) = .128 for $\alpha_1 = 1.9$ — Fig.4.4.1
$b_w/b_f = 230/1820 < 0.3$ for which $\alpha_3 = 0.8$. — Fig.4.4.3
.'. Allowable L/d ratio = 20x1.9x.8 > 20. .'. O.K.

Beam - B19,B20,B21 :

Step No. **Design Calculations**

1. Beam Mark : B19.
2. End Cond. : Simply Supported at Both Ends. EC = 1.
3. Span : L = 10 metres = 10000 mm.
4. Section : Assumed b = 230 mm, D = 750 mm, d' = 65mm, d = 685 mm, D_f = 140 mm.
5. Loads : Slabs on both sides S3 (One-way)
q = 6.75 kN/m², L_x = 4 m.
Slab load - Left $w_{s1} = 0.60xqL_x = 0.60x6.75x4 = 16.20$ kN/m
- Right $w_{s2} = 0.55xqL_x = 0.55x6.75x4 = 14.85$ kN/m *
Wall - Nil $w_w = 0$

* See Note on Page 129.

Design of Beam B19 : Continued

Self (beam) w_b = 25x.230x(.750-.140) = 3.51 kN/m
Total working load w = 16.20+14.85+0+3.51 = 34.56 kN/m
Total ultimate load w_u= 1.5xw=1.5x34.56 =51.84 =say 52 kN/m.

* **Note** : Though Code gives shear coefficient = 0.55 for dead load and 0.6 for live load, here the coefficient is taken as 0.55 for the total load since the live load is very small compared to dead load. Besides, it may be observed from Sect.3.4.1 that the total load on the penultimate support can even be taken equal to 1.1qLx instead of what is prescribed by the Code because redistribution of moments can be done at this penultimate support which reduces not only moment at this support but also the shear. * Table 8

6. Design Moment : $M_{u.max}$ = 52x10²/8=650 kN.m. Beam acts as T-beam.
L_o = L = 10000 mm.
b_f = $L_o/6+b_w+6D_f$ = 10000/6+230+6x140 = 2736 mm.
For $x_u=D_f$, M_{ur1}= $0.36f_{ck}\, b_f\, D_f\,(d - 0.42D_f)$
= 0.36x15x2736x140(685 − 0.42x140)x10⁻⁶
= 1295 kN.m > $M_{u.max}$ (=650 kN.m).
∴ x_u < D_f. Beam acts as rectangular beam with b=bf.

7. Main Steel : (a) Area :
Required Ast = $(0.5f_{ck}/f_y)[1-\sqrt{1-4.6xM_{u.max}/(f_{ck}\, b_f\, d^2)}]b_f\, d$

= $(0.5x15/415)[1-\sqrt{1-4.6x650x10^6/(15x2736x685^2)}]$ x2736x685 = 2740 mm².

Provide 6-#25 in two rows with maximum of 4 bars in bottom row.
Check for width : Required width = [4+(4+1)]x25 = 225 mm < 230 mm. ∴ O.K.
Check for Cover : Required cover = 25+25+25/2 = 62.5 mm < assumed 65 mm. ∴ O.K.

8. (a) Check for Development Length :

Here, it is thought of illustrating the calculations for curtailment of bars. Therefore, it will first be necessary to ascertain the number of bars required for bond and end anchorage to facilitate determining the maximum number of bars that can be curtailed.

L_d = $1.3M_1/V+L_{ex}$ ∴ Required M_1 = $V(L_d - L_{ex})/1.3$ Eq.4.3.2
V = $V_{u.max}$ = $w_uL/2$ = 52x10/2 = 260 kN.
L_d = 57 ∅ for M15-Fe415. ∴ L_d = 57x25 =1425 mm,
b_s = 230 mm. L_{ex} = $L_d/3-b_s/2$ = 1425/3-230/2= 360 mm.
Required M_1 = 260x(1425-360)/1.3/1000 = 213 kN.m.
Required N_1 = (213/650)x6 = say 2.
Minimum number of bars required for end anchorage
N_1 = $N_{max}/3$ = 6/3 = 2.

(b) Curtailment of bars (If desired)

Assuming only 2 bars to be curtailed and 4 to be continued.
The distance of Theoretical Point of Cutoff (TPC) from the centre of support
x_1 = $(L/2)(1-\sqrt{N'_1/N_{max}})$
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= .211x10=2.11m =2110mm.
The bars to be curtailed will be required to be extended through a minimum distance of effective depth d from TPC.

Design of Beam B 19 : Continued

8. (a) Check for Development Length Continued-

Therefore, the distance of Actual point of cutoff (APC) from the centre of support = $x_1 - d = 2110 - 685 = 1425$ mm.

9. Design for Shear :

$V_{u.max} = 260$ kN calculated earlier in Step-8(a) above.

The critical section for shear occurs at a distance d from the face of support.

$\therefore \quad V_{ud} = V_{u.max} - w_u(b_s/2 + d)$

$= 260 - 52(230/2 + 685)/1000 = 218.4$ 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 all the proposals, namely, design

(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,

with a view of comparison.

Case - I : No Curtailment and No Bent-up Bars :

$A_{st1} = A_{st.max} = 2945$ mm².
$p_t = 100A_{st}/(bd) = 100\text{x}2945/(230\text{x}685) = 1.87$ %.
$\tau_{uc} = 0.71$ N/mm² Table- 4.2
$V_{uc} = \tau_{uc}\, bd = 0.71\text{x}230\text{x}685/1000 = 111.86$ kN.
$V_{usv.min} = 0.35bd = 0.35\text{x}230\text{x}685/1000 = 55.14$ kN.
$V_{ur.min} = V_{uc}+V_{usv.min} = 111.86+55.14 = 167.00$ kN
$\ll V_{ud}$ (= 218.4 kN).

$\therefore$ Minimum stirrups will not be sufficient. They will be required to be designed.

$V_{usv} = V_{us} = V_{uD} - V_{uc} = 218.4 - 111.86 = 106.54$ kN.

Assuming #8 2-legged vertical stirrups of grade Fe415 A_{sv}=100mm²,

Required s= $0.87f_y\, A_{sv}\, d/V_{usv}$
= 0.87x415x100x685/(106.54x1000) = 232 mm.
< 450 mm and < .75x685 as well. $\therefore$ O.K.

Provide <u>#8</u> <u>at</u> <u>230</u> mm (in multiple of 10 mm on lower side).

Zone of these designd stirrups is given by

$L_{s1} = (V_{u.max} - V_{ur.min})/w_u = 1.79$ m = 1790 mm.

Number of stirrups in this zone $N_{s1} = (L_{s1} - b_s/2 - 50)/s_1 + 1$ (since first stirrup will be placed at a distance 50 mm from the inner face of support)

$\therefore N_{s1} = (1790 - 230/2 - 50)/230 + 1$ = say 8

Actual length of region covered upto the 8th stirrup
= 230/2 + 50 + (8-1)x230 = 1775 mm.

(Theoretical region covered by these 8 stirrups is sl/2=230/2 = 115 mm more than actual length = 1775 + 115 = 1890 mm which is greater than theoretical length Ls1 required = 1790 mm..$\therefore$.O.K.).

Beyond this zone, minimum stirrups will be sufficient. However, they are not necessary in the region where V_u is less than $.5V_{uc}$ near midspan. In this region nominal stirrups may be provided just to hold main bars and anchor bars to form a cage.

$\therefore$ Length of zone of nominal stirrups

$L_{s3} = 0.5V_{uc}/w_u = 0.5\text{x}111.86/52 = 1.075$ m.

Design of Beam B 19 : Continued Reference

9. Design for Shear : Continued

Length of zone of minimum stirrups

$L_{s2} = L/2 - L_{s1} - L_{s3}$

$L_{s2} = 10000/2 - 1790 - 1075 = 2135$ mm.

For #8 2-legged stirrups (A_{sv} = 100 mm²) of grade Fe415, the required spacing of minimum stirrups is given by

$s = A_{sv}\, f_y/(0.4b) = 100x415/(0.4x230)$ = say 450 mm.

< .75x685 (= 513 mm). ∴ O.K.

∴ Provide <u>#8</u> <u>at</u> <u>450</u> mm as minimum stirrups.

In the zone of <u>Nomimnal</u> <u>stirrups</u>, provide <u>Ø6</u> <u>at</u> <u>450</u> mm

Number of stirrups in Zone-II of minimum stirrups

N_{s2} = (L_{s1} + L_{s2} - actual region covered by stirrups in Zone-I)/(s of minimum stirrups)

= (1790 + 2135 - 1775)/450 = 5.

Actual region covered by stirrups in 1st and 2nd zone

= 1775 + 5x450 = 4025 mm.

Number of stirrups in Zone - III of nominal stirrups

N_{s3} = (10000/2 - 4025)/450 = 2.5.

It is the number only on one side of the centre line and therefore it can be in multiply of .5)

Total number of stirrups provided

= 2x[8(in Zone-I)+5 (in Zone-II)]= 26 of #8

and = 2x2.5 = 5 (in Zone-III)] of Ø6.

Case - II : No Curtailment but Use of Bent-up Bars :

A_{st1} = 4 of #25 = 4x491 = 1964 mm².

p_t = 100x1964/(230x685)= say 1.25 %, T_{uc}=0.64 N/mm². Table-4.2

V_{uc} = 0.64x230x685/1000 = 100.83 kN,

$V_{usv.min}$ = 55.14 kN as calculated in Case - I above.

∴ $V_{ur.min}$ = 100.83 + 55.14 = 155.97 kN

< V_{uD} (= 218.4 kN) calculated above.

∴ Minimum stirrups will not be sufficient. The shear reinforcement will be required to be designed.

$V_{us} = V_{uD} - V_{uc}$ = 218.4 - 100.83 = 117.57 kN.

Before deciding the scheme of bending of bars, it is advisable to determine the zone of shear reinforcement. The length of this zone is given by

$L_{s1} = (V_{u.max} - V_{ur.min})/w_u$

= (260 - 155.97)/52 = 2.0 m = 2000 mm.

Since this is greater than 2d (= 2x685 = 1370 mm), bending only one bar near the support and then again bending another bar at a distance of 0.75d 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. Theoretically, the first bar can be bent at a distance not exceeding 2d from the centre of support. Since, two bent-up bars are to be used one after the other, they will cover the entire zone of shear reinforcement. Therefore, it is proposed to bend first bar at a distance of 1.5d = 1.5x685 = 1027.5 say 1025 mm from the centre of support instead of maximum distance allowed equal to 2d.

Design of Beam B19 : Continued Reference

9. Design for Shear : continued

Shear resistance of 1 bar of #25mm (A_{st} = 491 mm²) bent at 45° will be

V_{usb} = $0.87 f_y A_{sb} \sin\alpha$ = 0.87x415x491xsin 45°/1000
= 125.33kN > V_{us} (=117.57 kN).

However, stirrups will have to be provided for
V_{usv} => $0.5V_{us}$ = 0.5x117.57 = 58.78 kN.

Since, this value is also greater than $V_{usv.min}$ (=55.14kN), minimum stirrups will not be sufficient and they will have to be designed.

Assuming #8 2-legged vertical stirrups of grade Fe415, as in Case - I above (A_{sv} =100 mm²), required spacing will be
s_1 = 0.87x415x100x685/(58.78x1000) = 420 mm.
for a length of 1025 mm.

No. of stirrups in this Zone-I(a),
N_{s1a} = (L_{s1a} - b_s/2 - 50)/s_1 + 1
= (1025 - 230/2- 50)/420+ 1 = 3.05 say 3.

Actual length of the region covered by these stirrups
L_{s1a} = 230/2 + 50 + (3 - 1)x420 = 1005 mm.

This will do because last stirrup is further effective for a distance of 420/2 = 110 beyond 1005 mm.

At a distance of 1025 mm where the first bar is bent,
V_u = 260 - 52x1.025 = 206.7 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 (= 100.83 kN) for 4 bars of #25. However, this increase is ignored for simplicity.

∴ V_{us} = 206.7 - 100.83 = 105.87 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} that can be utilized is maximum of $0.5V_{us}$. Therefore, stirrups will be required to provide a shear resistance of $0.5V_{us}$ = .5x105.87 = 52.94 kN. This is less than $V_{usv.min}$ = 55.14 kN. Therefore, in this region, minimum stirrups are adequate.

Provide #8 at 450mm of grade Fe415 as calculated in Case-I in a region between point of first bend to point of second bend (L_{s1b} = L_{s1} - 1.5d = 2000 - 1025 = 875 mm).

No. of stirrups in this region
N_{s1b} = (L_{s1}-actual L_{s1a})/s =(2000-1005)/450=2.2 say 2.

Actual length of region covered by these stirrups and former 3 stirrups= 1005 + 2x450 = 1905 mm. This can be allowed because last stirrup is effective for a further distance of 450/2 = 225 mm.

Length of zone of nominal stirrups remains same equal to 1075mm.
Length of zone of minimum stirrups = 10000/2-1905-1075 = 2020mm.

Number of stirrups in the zone of minimum stirrups
N_{s2} = 2000/450 = 4.48 say 5.

Length of the region covered by these stirrups = 5x450 = 2250 mm.
Total length of region covered by stirrups from support
= 1905 + 2250 = 4155 mm.

∴ Number of stirrups required in the zone of nominal stirrups
N_{s3} = (10000/2 - 4155)/450 = 1.9 say 2.

Design of Beam B19 : Continued

9.Design for Shear : Continued

Total number of stirrups will be as follows :

Zone	Dia.& spacing	Number	Region length
I-a(upto first bend)	#8 at 420 mm	3	1005 mm
I-b(from first to second bend)	#8 at 450 mm	2	900 mm
II (Minimum stirrups)	#8 at 450 mm	5	2250 mm
III (Nominal stirrups)	ϕ6 at 450 mm	2	845 mm

Total number of stirrups in a beam = 2x(3+2+5+2) = 24.

Case - III : Curtailment of Bars - No Bent-up Bars :

At the critical section for shear at a distance of 230/2+685 = 800 mm from the centre of support, V_{uD} = 218.4 kN calculated at the beginning of Step-9.

On curtailment of 2 bars, available bars at support are 4 of #25 as in Case -II above, for which V_{uc} = 100.83 kN. It has also seen in Case-II that minimum stirrups are not adequate.

∴ V_{us} = 218.4 - 100.83 = 117.57 kN.

Since no bent-up bars have been provided,

V_{usb} = 0 and V_{usv}=V_{us} = 117.57 kN.

For #8 2-legged stirrups (A_{sv} = 100 mm²), of grade Fe415, required spacing is given by

s = 0.87x415x100x685/(117.57x1000) = 210 mm.

For 4 of #25 at supports,

$V_{ur.min}$ = 155.97 kN and L_{s1} = 2000mm as in Case-II above.

Now, theoretical point of cutoff (TPC) from bending moment consideration is at a distance of 2110mm from centre of support. Since bars are curtailed at this point, one of the three conditions of Cl.25.2.3.2 of the Code are required to be satisfied.

Condition-(a)

According to this condition, shear at TPC shall not exceed 2/3rd the shear strength i.e. shear resistance at that section shall be at least 1.5 times the shear at the section.

At TPC, V_u = 260 - 52x2.11 = 150.28 kN.

∴Required V_{ur} = 1.5x150.28 = 225.42 kN.

V_{uc} = 100.83 kN for 4 bars of #25 as in Case-II above.

Required V_{us} = 225.42-100.83 = 124.59 kN.

For #8 2-L stirrups (A_{sv} = 100 mm²) of grade Fe415, required spacing s = 0.87x415x100x685/(124.59x1000) = say 200 mm.

This will be required between TPC to APC i.e.in the region between points 1425 mm and 2110 mm from the centre of supports.

Thus, spacing of #8 stirrups is required to be 210 mm for a distance of 1425 mm from centre of support to the actual point of cutoff and 200 mm for a distance 685 mm between APC to TPC. For simplicity and ease of fixing, 200mm spacing may be provided right from the support to TPC for a length of 2110 mm. Since length of region of designed shear reinforcement L_{s1} is just 2000 mm (< 2110 mm) (see Case-II),minimum stirrups (#8 at 450 mm) would be provided beyond TPC.

Number of stirrups required in Zone-I

N_{s1} = (2110 - 230/2 - 50)/200 + 1 = say 11.

Actual length of region covered by these stirrups

= 230/2 + 50 + (11 - 1)x200 = 2165 mm.

Reference

Design of Beam B19 : Continued

9.Design for Shear : Continued

Length of zone of nominal stirrups remains same equal to 1075mm.

Length of zone of minimum stirrups = 10000/2-2165-1075 = 1760mm.

Number of stirrups in Zone-II of minimum stirrups

N_{s2} = 1760/450 = say 4.

Actual length covered by stirrups in Zone-I and Zone-II

= 2165 + 450x4 = 3965 mm.

Number of stirrups required in Zone -III of nominal stirrups

= (10000/2 - 3965)/450 = say 2.5.

Total number of stirrups required

= 2x[11(in Zone-I)+4(in Zone-II)]=30 of #8,

and = 2x2.5 (in Zone-III)] = 5 of ø 6.

Alternatively, Condition-(b) :

At TPC,V_u = 150.28kN while $V_{ur.min}$= 155.97kN as in Case-II.

Therefore, minimum stirrups would have been adequate had the bars not been curtailed. Because of curtailment, additional stirrups are required to be provided having area of minimum stirrups. In other words, the amount of stirrups is required to be doubled or that the spacing is required to be halved. This gives #8 at 225 mm. At the same time, it is necessary that the resultant pitch (i.e. 225 mm) shall not exceed d/8B, where B = (No. of bars curtailed/Total No. of bars prior to curtailement) = 2/6 = 1/3. .·. s <= 685/(8x1/3) = 256 mm. s = 225 mm is O.K.

Thus, spacing for #8 stirrups is required to be 210 mm from support to APC for a distance 1425 mm, and for satisfying the requirement of the Condition-(b), the spacing of #8 stirrups is required to be 225 mm for a distance 685 mm from APC to TPC, and 450 mm beyond. For ease of fixing, a constant spacing of 210 mm is suggested from support to TPC.

Number of stirrups required in Zone-I

N_{s1} = (2110 - 230/2 - 50)/210 + 1 = say 10.

Actual length of region covered by these stiorrups

= 230/2 + 50 + (10-1)x210 = 2055 mm.

This can be allowed since last stirrup is effective for a further distance of 210/2 = 105 mm giving total effective zone

= 2055 + 105 = 2160mm > required 2110 mm.

Length of Zone of nominal stirrups remains same = 1075 mm.

Length of Zone of minimum stirrups=10000/2-2160-1075=1765mm.

Number of stirrups required in Zone-II of minimum stirrups

= 1765/450 = say 4.

Actual length of region covered by stirrups in Zone-I & II

= 2055+450x4 = 3855 mm

Number of stirrups required in Zone-III of nominal stirrups

= (10000/2 - 3855)/450 = say 2.5 of ø 6mm.

Total number of stirrups provided

= 2x[10 (in Zone-I) + 4 (in Zone-II)]= 28 of #8 mm

and = 2x2.5 = 5 (in Zone-III)] Of ø 6mm.

Condition - (c) :

This condition is applicable only when area of extended bars beyond the point of cutoff is double the required area. In this case, area of bars continued is just equal to that required beyond point of cutoff. Therefore, shear reinforcement cannot be designed according to the requirements of this condition.

Design of Beam B19 : Continued Reference

9. Design for Shear : continued

Comparison of Reinforcement for 3 Cases :

Case-I : No Curtailment and No bent-up Bars :

For each stirrup, vertical straight length between horizontal legs
E = 230 − 2x25 = 180 mm,
Horizontal straight length between of vertical legs
A = 750 − 2x25 = 700 mm.
Length of #8 stirrup = 2(A + E) + 24 ø
= 2(700 + 180) + 24x8 = 1952 mm.
Length of ø6 stirrup = 2(700 + 180) + 24x6 = 1904 mm. — Table VIII of IS:2502
Unit wt. of #8 bar = .785 x area of bar in cm^2x 1m
= .785x(IIx8^2/4)/100= .395 kg/m
Weight of #8 stirrup = .395x1952/1000 = .77 kg/No
Unit Wt. of ø6 bar = .785x28.27/100 = .22 kg/m
Weight of ø6 stirrup = .22 x1904/1000 = .42 kg/No
Total number of stirrups = 13x2=26 of #8 & 2x2.5 = 5 of ø6.
Total weight of stirrups = 26x.77+5x.42 = 22.12 kg.
Total length of main bar = end to end length+2B(where B=6 ø)
= L+bs−2(end clearance=2 ø) +2x6 ø
= 10000 + 230 − 2x2x25 + 2x6x25
= 10430 mm = 10.43 m.
Unit weight of 25 mm bar =.785x(π x25^2/4)/100 = 3.853 kg/m
Total weight of main bars= No.of barsxunit Wt.x length in m.
= 6 x 3.853 x 10.43 = 241.12 kg
Total weight of main bars and stirrups=241.12+22.12=263.24kg.

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+25+25+25/2 = 87.5 mm from bottom face. The top bars are at adistance 25+25/2 = 37.5 mm from top face. Therefore, vertical distance between centre of top bar and centre of bent bar =750−(87.5+37.5) = 625mm. ∴
Excess length of each bent bar = 625x(1.414−1)x2 = 518mm say 520mm.
Total length of each bent bar = 10430+520 = 10950 mm = 10.95 m.
Total weight of all bars = 3.853x(4x10.43+2x10.95)=245.13 kg.
Total number of stirrups = 20 of #8 and 4 of ø6.
Total weight of all stirrups = 20x.77 + 6x.42 = 17.08 kg.
Total weight of main bars and stirrups= 245.12+17.08 = 262.20 kg.

Case-III : Curtailment and No Bent-up Bars :

Condition-(a):Length of curtailed bars=2(10000/2−1425)=7150mm=7.15m.
Total weight of main bars = 3.853x(4x10.43 + 2x7.15) = 215.85 kg.
Total number of stirrups = 30 of #8 and 5 of ø6.
Total weight of stirrups = 30x.77+5x.42= 25.20 kg
Total weight of main bars and stirrups = 215.85+25.20= 241.05 kg.

Case No.	Weight of main bars	stirrups	Total in kg.
I	241.12	22.12	263.24
II	245.13	17.08	262.21
III	215.85	25.20	241.05

Percentage saving for Case −III in comparison with Case−I = 8.43%.

There is, practically, no difference between results of Case-I and Case-II. In continuous beam, however, bent up bars serve to act as negative moment steel at support.

Design of Beam B19 : Continued Reference

10. Check for Serviceablity :

(a) Allowable L/D ratio Approach: Actual L/d ratio=10000/685<20. Required p_t=100xA_{st}/(b_fd) for T beams =100x2740/(2736x685)=.15%. For p_t = 0.15 %, $\alpha 1$ =1.8 Since, b_w/b_f = 230/2736 <<.3, $\alpha 3$ =.8. Therefore, allowable L/d ratio > 20. ∴ O.K.

(b) Calculation Approach : Since, the span is large (10 m), the seriviceability requirements regarding deflection are checked by calculating actual maximum deflection and comparing it with permissible deflection.

The total deflection consists of short-term deflection (a_i) due to serivce load, and long-term deflection (a_{cc}) due to creep and (a_{cs}) due to shrinkage.

(i) Short-term Deflection :

For a simply supported beam carrying UD load, the short-term deflection is given by $a_i = (5/384)[wL^4/(E_c I_{eff1})]$. Thus, for calculating a_i, it is first necessary to determine w, E_c and I_{eff1}.

$w = 52/1.5 = 34.67$ kN/m.

$E_c = 5700\sqrt{fck} = 5700\sqrt{15} = 22076$ N/mm².

$I_{eff1} = I_{r1}/[1.2 - (M_r/M_o)(z/d)(1 - x/d)(b_w/b_f)] = (I_{r1}/C)$

$\geq I_{r1}$ but $< I_{gr}$.

where $C = [1.2 - (M_r/M_o)(z/d)(1 - x/d)(b_w/b_f)$

For determining I_{eff1}, it is first necessary to calculate following values at service load,

x = depth of concrete in compression (i.e.depth of N.A.),
z = lever arm ,
M_o = maximum moment at the centre of span,
M_r = cracking moment of resisatance and hence
f_r = modulus of rupture,
I_{gr}= moment of inertia (I_{gr}) of gross flanged section @ its centroidal axis ignoring the reinforcement,and
I_r = moment of inertia of cracked transformed section including the reinforcement.

Depth of neutral axis : Assuming neutral axis to lie in flange,

$b_f x^2/2 = mA_{st}(d-x)$, b_f=2736mm, m =E_s/E_c, E_s =200000 N/mm²,

m = 200000/22076 = 9.06, A_{st} = 6-#25 = 2945mm²,

D = 750mm, d = 685mm, D_f = 140 mm, b_w = 230 mm.

2736 x²/2 = 9.06x2945(685 - x). Solving,

x = 106.25 mm << 140 mm. ∴ O.K.

z = d-x/3 = 685 - 106.25/3 = 649.58 mm.

$I_{r1} = b_f x^3/3 + mA_{st}(d-x)^2$

= 2736x106.25³/3+9.06x29545x(106.25)²=10031x10⁶ mm⁴.

$M_o = M_{u.max}/1.5 = 650/1.5 = 433.3$ KN.m

$M_r = f_{cr} I_{gr}/y_t$, $f_{cr} = .7\sqrt{fck} = .7\sqrt{15} = 2.71$ N/mm².

For, determining I_{gr}, depth of centroidal axis below the compression face, of the gross flanged section ignoring the reinforcement will be required.

Taking moment of concrete area about compression face,

$x = [b_w D^2/2 + (b_f - b_w) D_f^2/2]/[b_w D + (b_f - b_w)D_f]$

$$x = \frac{[230 \times 750^2/2 + (2736-230) \times 140^2/2]}{[230 \times 750 + (2736 - 230) \times 140]} = 170.53 \text{ mm.}$$

Design of Beam B19 : Continued

10. Check for Serviceability : continued

I_{gr} = 2736x170.53³/3 – [(2736 – 230)x(170.53 – 140)³/3
+ 230x(750-170.53)³/3 = 19416x10⁶ mm⁴,

M_r = 2.71x19416x10⁶ /(750-170.53)
= 90.80x10⁶ N.mm = 90.8 kN.m

I_{eff1} = I_r1/C_1 but > I_{r1} and < I_{gr}.

C1 = 1.2-(90.8/433.33)(649.58/685)(1-106.25/685)(230/2736)
= 1.1858 > 1

∴ I_{eff1} = I_{r1} = 10031x10⁶ mm⁴ < I_{gr}(=16611x10⁶ mm⁴). ∴O.K.

a_i = (5/384)(34.67x10)x(10³x10¹²)/[22076x10031x10⁶]
= 20.39 mm

(ii) Deflection due to Shrinkage (a_{cs}) :

a_{cs} = k_3 c_s L^2, k_3 = 0.125 for simply supported beam,

cs = k_4 ecs/D, ecs = 0.0003.

k4 = $0.72(p_t-p_c)/\sqrt{p_t}$ <=1 for (p_t-p_c)< 1, p_t=100A_{st}/(bd)
= $0.65(p_t-p_c)/\sqrt{p_t}$ <=1 for (p_t-p_c)>=1, p_c=100A_{sc}/(bd)

p_t = 100x2945/(230x685)=1.87% > 1%, p_c=0,∴(p_t-p_c)=1.87

k_4 = $0.65\times1.87/\sqrt{1.87}$ = 0.89,

cs = 0.89x0.0003/750 = 0.356x10⁻⁶

a_{cs} = 0.125x0.356x10⁻⁶ x(10⁶ x10³)< = 4.45 mm.

(iii)Deflection due to Creep (a_{cc}) :

Creep deflection is always due to permanent load (w_{perm})only. It is given by

$a_{cc.perm}$ = $a_{icc.perm}$ – $a_{i.perm}$

w_{perm} = dead load shears from slabs from two sides + self wt.
= 0.66x5.25x4 + 0.55x5.25x4 + 3.51 = 27.66 kN/m

Moment at the centre of span due to permanent load only= $M_{o.perm}$

$M_{o.perm}$ = $w_{perm}L^2/8$ = 27.66x10²/8 = 345.75 kN.m

$a_{i.perm}$ = (w_{perm}/w)xa_i=(27.66/34.67)x20.39 = 16.27 mm.

For calculating $a_{icc.perm}$, the calculations will be repeated on the lines of calculation of a_i above for revised (long-term) modulus of elasticity (E_{ce}).

E_{ce} = $E_c/(1+\phi)$ where ϕ is creep coefficient.

Assuming 28 days age of loading,

ϕ = 1.6 and E_{ce} = 22076/(1+1.6) = 8491 N/mm².

Long term modular ratio,

m' = Es/E_{ce} = 200000/8491 = 23.55.

Depth of N.A. : Assuming neutral axis to lie in web because increase in modular ratio lowers the neutral axis,

x = $(b_fD_f^2/2 + mA_{st}\ d)/(b_fD_f + mA_{st})$

$$x = \frac{(2736\times140^2/2+23.55\times2945\times685)}{(2736\times140 + 23.55\times2945)} = 164.23\text{ mm} > D_f$$

$\bar{x}$ = $[(3x - 2D_f)/(2x - D_f)](D_f/3)$
= [(3x164.28-2x140)/(2x164.28-140)](140/3) = 52.67 mm.

z = 685 – 52.67 = 632.33 mm

I_{r2} = 2736x164.28³/3 – (2736 – 230)(164.28 – 140)³/3
+ 23.55x2945(685-164.28)² = 22837x10⁶ mm⁴.

I_{eff2} = I_{r2}/C_2 >= I_{r2} but < I_{gr}.

C2 = $1.2-(M_r/M_{o.perm})(z/d)(1 - x/d)(b_w/b_f)$

C2 = 1.2-(90.8/345.75)(632.33/685)(1-164.28/685)(230/2736)
= 1.187 > 1.

∴I_{eff2} = I_{r2} = 22837x10⁶ mm⁴. But I_{eff2} <=I_{gr} (=19416x10⁶ mm⁴)

∴I_{eff2} = I_{gr} = 19416x10⁶ mm⁴.

Design of Beam B19 Continued Reference

10. Check for Serviceability : continued

$a_{icc.perm} = (5/384)(w_{perm}\ L^4)/(E_{ce}\ I_{eff2})$
$= (5/384)(27.66x10)(10^3x10^{12})/(8491x19416x10^6)$
$= 21.85$ mm.

$a_{cc.perm} = a_{icc.perm} - a_{i.perm} = 21.85 - 16.27 = \underline{5.58}$ mm.

(iv)Total Deflection a_T :

$a_T = a_i + a_{cs} + a_{cc.perm} = 20.39 + 4.45 + 5.58 = \underline{30.42}$mm.

Allowable total deflection = span/250 = 10000/250=$\underline{40}$ mm *22.2(a)
> 30.42 mm. ∴ safe.

(v) Partial deflection after the transfer of load :

$a_p = a_{cc.perm} + a_{cs}(part) + a_{i.temp}$

Assuming 50 % shrinkage to have occurred prior to loading at 28 days, a_{cs}(part) that occurs after loading

$a_{cs}(part) = 0.5xa_{cs} = 0.5x4.45 = 2.23$ mm.

$a_{i.temp} = a_i - a_{i.perm} = 20.39 - 16.27 = 4.12$ mm

∴ $a_p = 5.58 + 2.23 + 4.12 = \underline{11.93}$ mm.

Allowable deflection after the transfer of load
= span/350 or 20 mm whichever is less. *22.2(b)
= 10000/350 = 28.57 mm or 20mm ∴ 20mm > 11.93 mm. ∴ O.K.

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ategory – II : Beams Simply Supported at One End and Continuous at the Other.
– Uniformly Distributed Load only .

II(a) Beam – B1 **Reference**

1. Beam Mark : B1
2. End Cond. No.: 2
3. Span : L = 4.0 m
4. Section : Assumed b = 230 mm, D = 380mm. dc = 35mm, d = 345mm
5. Loads : Self + Parapet 1m + S2(UDL)
   $w_u = 1.5(1.6+5.0+6X2.5/2) = 21.15$ kN/m

6. (a) Design Moment : $M_{u.max} = w_u\ L^2/10 = 21.15X4^2/10 = 33.84$ kN.m at midspan as well as at continuous end.
   (b) Maximum M_{ur} : $M_{ur.max} = 56.67$ kN.m > $M_{u.max}$(=33.84 kN.m) Table 6.1
   ∴ The section will be designed as Singly Reinforced Rectangular.
7. Main Steel : (a) Area
   Required $A_{st} = (0.5x15/415)[1-\sqrt{1-4.6x33.84x10^6/(15x230x345^2)}]$ x230x345
   = 304 mm² at midspan as well as at continuous end.
   (b) No. Diam. : Provide $\underline{3}$ $\underline{\text{of}}$ $\underline{\#12}$ in 1 row.(nl = 3). A_{st} = 339mm².
   (c) Check for width: Required b =3x12+(3+1)x25=136mm < 230mm.∴O.K.
   (d) Check for cover: Required d'=25+12/2=31mm < assumed 35mm.∴O.K.
   (e) Effective depth d = 380-31 = 349 mm.
   (f) Detailing : Midspan Continuous end
   Top St. 2-#10(anchor bars) : 2=#10 continuation of anchor bars
   <u>Bottom</u> <u>Bt.</u> <u>1-#12</u>(to both ends):+2-#12 by 1-#12 bent from adjacent
   <u>Bottom</u> <u>St.</u> <u>2-#12</u> : spans.

8. Check for Shear : (a)At continuous end :
   $V_{u.max} = 0.6w_u\ L = 0.6X21.15X4 = 50.76$ kN
   A_{stl} = 2-#12 + 2-#10. $V_{ur.min} = 57.21$ kN even for 2-12#. Table 6.1
   ∴ Minimum stirrups are sufficient.Provide $\underline{\phi 6}$ $\underline{\text{at}}$ $\underline{150}$ mm Table 6.1

Design of Beam B1 : Continued References

8. Check for Shear : (b) At discontinuous end:
$V_{u.max} = 0.45 w_u L = 0.45 \times 21.15 \times 4 = 38.07$ kN
$V_{ur.min} = 57.21$ kN for $A_{st1} = 2$-#12 as seen above $> V_{u\ max}$.
∴ Minimum stirrups ø6 at 150 mm is adequate.

9. Check for Development Length:(a) For Top steel at Continuous end:
Required $L_d = 684$ mm for #12 — Table 6.1
Available L_d = actual length of top steel
= L/4 = 4000/4 =1000 mm..∴O.K.

(b) For Bottom Steel at Simply Supported End :
Required $L_d = 684$ mm for #12 — Table 6.1
Available $L_d = 1.3M_1/V + L_{ex}$
M_1 = M.R. of A_{st1} i.e. of 2-#12 = 25.93 kN.m — Table 6.1
$V = V_{u.max} = 50.76$ kN, $L_{ex} = 113$ mm for #12 bar — Table 6.1
Available $L_d = (1.3 \times 25.93 \times 1000/38.07) + 113 = 998$ mm ∴ O.K.

10. Check for Deflection:
Actual L/d = 4000/349 < 26.
For 2-#12 in a section 230x380mm, p_t =.285% and $\alpha_1 = 1.43 > 1$ — Table-6.1
∴ Allowable L/d ratio > 26. ∴ O.K.

II(b) Beam – B6 : Brief solution.

Span L = 4 m. Section assumed – 230x380 mm.
Loads : self + Slab S2(UDL)+ Slab S3 (Tri. load)
$w_{u'} = 1.5(1.6 + 6 \times 2.5/2 + 6.75 \times 4.0/6) = 20.40$ kN/m. — 5.2.2
$w_{us} = 1.5(1.6 + 6 \times 2.5/2 + 6.75 \times 4.0/8) = 16.81$ kN/m. — Step-11(c)
$M_{u.max} = 20.4 \times 4^2/10 = 32.64$kN.m at midspan and at contiuous end.
Steel: At midspan – 3 of #12 (2 St + 1 Bt.), $M_{ur} = 37.23$ kN.m — Table 6.1
At continuous end : 2-#10 by continuation of top anchor bars
+ 2-#12 by bending of 1-#12 from each adjacent span.
Shear: (a) At Continuous End : $V_{u.max} = 0.6 w_{u} sL = 0.6 \times 16.81 \times 4 = 40.34$ kN.
$V_{ur.min} = 57.21$ kN even for 2-#12 $> V_{u.max}$ Provide ø6 at 150mm. — Table 6.1
(b) At simply supported end: $V_{u.max} = 0.45 \times 16.81 \times 4 = 30.26$ kN.
A_{st1} = 2-#12 for which $V_{ur.min} = 57.21$ kN $> V_{u.max}$ — Table 6.1
Therefore, provide minimum stirrups ø6 at 150 mm.
Load on column : (a) Continuous end = 0.6[1.5(1.6+6x2.5/2)]x4 = 32.76 kN
(b) Discontinuous end = 0.45x13.65x4 = 24.57 kN
Bond : (a) At Continuous End : Required Ld = 684 mm for #12 bars.
Available L_d = L/4 = 4000/4 = 1000 mm > 684 mm. ∴ O.K.
(b) At simply Supported End: Required L_d=684mm for #12 bars.
Available $L_d = 1.3M1/V_{u.max} + L_{ex}$. $V_{u.max}$ 30.26 kN.
$L_{ex} = 113$ mm for #12 bars, and $M_1 = 25.93$ kN.m for 2-#12. — Table 6.1
∴ $L_d = 1.3 \times 25.93 \times 1000/30.26 + 113 = 1227$ mm > 684 mm..∴O.K.

II(c)Beam – B12 : Brief Solution

Span : L = 4 m. Assumed section – 230x380 mm.
Loads: self + parapet + slab-S3 (Tri. from one-way).
$w_u = 1.5(1.6+5+6.75 \times 4/6) = 16.65$ kN/m.
$w_{us} = 1.5(1.6+5+6.75 \times 4/8) = 14.96$ kN/m.
$M_{u.max} = 16.65 \times 4^2/10 = 26.64$ kN.m
at midspan and at contnuous end.
Main Steel:At Midspan : 3-#12. (2 St.+1 Bt). $M_{ur} = 37.23$ kN.m — Table 6.1
Top St. 2-#10 as anchor bars.
At Continuous End:2-#10 by continuation of anchor bars
+ 2-#12 by bending bars from adjacent spans.(A_{st}=383mm²)

Design of Beam* B12 *: Continued

Shear: (a) At Continuous End : $V_{u.max} = 0.6 \times 14.96 \times 4 = \underline{35.90}$ kN. Table 6.1

$A_{st1} = 2$-#12+2-#10 $V_{ur.min} = 57.21$ kN even for 2-#12

∴ Provide Minimum stirrups $\underline{\phi 6}$ at $\underline{150}$ mm

(b) At Simply Supported End : $V_{u.max} = 0.45 \times 14.96 \times 4 = \underline{26.93}$ kN.

$A_{st1} = 2$-#12 for which $V_{ur.min} = 57.21$ kN.

∴ Provide minimum Stirrups $\underline{\phi 6}$ at $\underline{150}$ mm.

Load on column : (a) Continuous end $= 0.6[1.5(1.6+5)] \times 4 = 23.76$ kN

(b) Discontinuous end $= 0.45 \times 9.9 \times 4 = 17.82$ kN

Bond : (a) At continuous End : required $L_d = 684$ mm for #12 bar Table 6.1

Available $L_d = L/4 = 4000/4 = 1000$ mm. ∴ O.K.

(b) At Simply Supported End : Required $L_d = 684$ mm for #12.

Available $L_d = 1.3M_1/V + L_{ex}$. $V = V_{u.max} = 26.93$ kN.

$L_{ex} = 113$ mm, and $M_1 = 25.93$ kN.m for 2-#12 Table 6.1

∴ Available $L_d = 1.3 \times 25.93 \times 1000/26.93 + 113 = 1365$mm. ∴ O.K.

II(d) Beam – B16: Brief Solution.

Span : L = 4.5 m. Assumed Section $\underline{230 \times 380}$ mm.

Loads: self+parapet+slab-S4(Tri. Two-way)

$w_u = 1.5(1.6+5+6.75 \times 4.5/3) = 25.09$ kN/m.

$w_{us} = 1.5(1.6+5+6.75 \times 4.5/4) = 21.29$ kN/m.

$M_{u.max}$ $25.09 \times 4.5^2/10 = 50.81$kN.m at midspan & at continuous end.

Main Steel : At Midspan: Provide $\underline{5\text{-\#}12}$ (3 St.+2 Bt.). $M_{ur} = 56.51$ kN.m Table 6.1

Top St. 2-#12 as anchor bars.

: At Continuous End: $\underline{5\text{-\#}12}$ by continuation of 2-#12 anchor bars + 2-#12 Bt from bottom bars in this beam + 1-#12 bent from adjacent span (B15).

Shear: (a) At Continuous End: $V_{u.max} = 0.6 \times 21.29 \times 4.5 = \underline{57.48}$ kN.

$A_{st1} = 5$-#12 for which $V_{ur.min} = 70.01$ kN. Provide $\phi 6$-150mm. Table 6.1

(b) At Simply Supported End : $V_{u.max} = 0.45 \times 21.29 \times 4.5 = \underline{43.11}$ kN

$A_{st1} = 3$-#12 for which $V_{ur.min} = 62.53$ kN. Provide $\phi 6$-150mm. Table 6.1

Bond: (a) At Continuous End: Required $L_d = 684$ mm for #12

Available $L_d = L/4 = 4500/4 = 1125$ mm. ∴ O.K.

(b) At Simply Supported End : Required $L_d = 684$ for #12.

Available $L_d = 1.3M_1/V + L_{ex}$, $V = V_{u.max} = 43.11$ kN.

$M_1 = 37.23$ kN.m for 3-#12 and $L_{ex} = 113$ mm Table 6.1

Available $L_d = 1.3 \times 37.23 \times 1000/43.11 + 113 = 1235$ mm. ∴ O.K.

Category – III : Beam Continuous at Both Ends

III(a) Beams – B2, B3, B4 :

1. Beam Mark : B2, B3, B4
2. End Cond. : Continuous at Both Ends. EC = 3.
3. Span : L = 4 m = 4000 mm.
4. Section : Assumed b = 230 mm, D = 380 mm, d' = 35 mm, d = 345 mm.
5. Loads : Same as that for Beam B1. $w_u = w_{us} = 21.15$ kN/m.
6. (a) Design Moment: $M_{u.max} = w_u L^2/12 = 21.15 \times 4^2/12 = 28.2$ kN.m. at midspan as well as at supports.
   (b) $M_{ur.max} = 56.67$ kN.m $> M_{u.max}$ ∴ It can be designed as rectangular section.
7. Main Steel: (a) Midspan Section: Provide $\underline{3\text{-\#}12}$, $M_{ur} = 37.23$ kN.m Table 6.1
   Bottom 2 St. + 1 Bt. Top 2-#10 as anchor bars.
   (b) Support Section: Provide $\underline{2\text{-\#}12}$ by 1 bar bent from each adjacent span + $\overline{2\text{-\#}10}$ by continuation of anchor bars.

Design of Beams B2 - B3 - B4 : Continued

8. Shear: At Both Supports : $V_{u.max}$ = 21.15x4/2 = 42.30 kN.
   A_{st1} = 2-#12+2-#10. $V_{ur.min}$ = 57.21 kN even for 2-#12. Table 6.1
   ∴Provide minimum stirrups ø6 at 150mm.
9. Bond : (a) At Supports:Required L_d = 684 mm for #12 bar. Table 6.1
   Available L_d = L/4 = 4000/4 = 1000 mm. ∴ O.K.
   (b) At Point of Contraflexure (PC) for bottom bars.
   Available L_d = M_1/V+L_{ex}. L_{ex} = 113 for #12 bar. Table 6.1
   Astl at PC = 2-#12 for which M1 = Mur = 25.93 kN.m. Table 6.1
   V = V_u at PC = $V_{u.max}$-w_uxL/5=42.30-21.15x4/5=25.38 kN.
   Available L_d = 25.93x1000/25.38+113 = 1134 mm.. ∴ O.K.
10. Check for Serviceability : Actual L/d ratio=4000/345 < 26.
   Allowable $L/_d$ ratio > 26 since α1 > 1. ∴ O.K. Table 6.1

Note: For details of reinforcement of B4 & B9,see comments at the end of B10.

III(b)Beams - B7,B8,B9 : Brief Solution.

Span : L = 4.0 m = 4000 mm. Assumed Section 230x380 mm.
Loads : w_u = 20.40 kN/m and w_{us}=16.81 kN/m as that for slab S6.
$M_{u.max}$= 20.40X4²/12 = 27.2 kN.m < $M_{ur.max}$ = 56.67 kN.m.
Main Steel:(a) At Midspan:Provide 3-#12 (2 St.+1 Bt.)in one row.
Provide 2-#10 at top as anchor bars.
(b) At Both Supports: Provide 2-#12 by bending 1-#12 from adjacent spans+2-#10 by continuation of anchor bars.
Shear: At Supports : $V_{u.max}$= w_{us}L/2 = 16.81X4.0/2 = 33.62kN.
∴ A_{st1}= 2-#12+2-#10. $V_{ur.min}$= 57.21 kN even for 2-#12. Table 6.1
∴ Provide minimum stirrups ø6 at 150 mm.
Load on column = 0.5[1.5(1.6+6x2.5/2)]x4 = 27.30 kN.
Bond : (a) At Supports : Required L_d = 684 mm for #12 bar.
Available L_d = L/4 = 4000/4 = 1000 mm. ∴ O.K.
(b) At Points of Contraflexure (Pc): Required L_d=684 mm.
Available L_d = M_1/V+L_{ex}. L_{ex} = 113 mm for #12 bar.
A_{st1} = 2-#12 at PC for which M_1 = M_{ur} = 25.93 kN.m Table 6.1
V = V_u at PC = $V_{u.max}$- w_uL/5 = 33.62-16.81x4/5=20.17kN.
∴ Available L_d = 25.93X1000/20.17+113 = 1398 mm. ∴ O.K.

III(c) Beams - B13,B14,B15 : Brief Solution **Reference**

Span : L = 4 m = 4000 mm. Assumed Section 230x380 mm.
Loads: w_u=16.65 kN/m and w_{us}=14.96 kN/m -Same as that for Beam B12.
$M_{u.max}$ = 16.65x4²/12 =22.20 kN.m at midspan and at supports.
Main Steel:(a)At Midspan: Provide 2-#12 Bottom St. in one row. Table 6.1
Provide Top St. 2-#10 as anchor bars.
(b)At Supports:Provide 2-#10 by continuing anchor bars from midspan + 1-#10 extra. A_{st} = 235 mm².
Shear:$V_{u.max}$ = w_{us}L/2 = 14.96x4/2 = 29.92 kN.
$V_{ur.min}$ = 57.21 kN even for 2-#12 (< provided 3-#10). Table 6.1
∴ Provide minimum stirrups ø6 at 150 mm.
Load on Column = 0.5[1.5(1.6+5)x4 = 19.80 kN
Bond :(a) At Supports : Required L_d = 57x10=570mm for #10 bars.
Available L_d = L/4 = 4000/4 = 1000 mm. ∴ O.K.
(b) At Points of Contraflexure(PC) : Required L_d = 684 mm Table 6.1
Available L_d = M_1/V+L_{ex}. Lex = 113 mm for #12 bars.
A_{st1} = 2-#12 for which M_1 = M_{ur} = 25.93 kN.m Table 6.1
V = V_u at PC = $V_{u.max}$-wusxL/5 = 29.92-14.96x4/5=17.95 kN.
Available L_d = 25.93x1000/17.95+113 = 1557 mm. ∴ O.K.

Category - IV Miscellaneous Beam

IV(a) Beams B22-B23 :

Step No. Design Calculations

1. Beam Mark: B22-B23
2. End Cond : Continuous Beam with equal spans. Ends simply supported.
3. Span : 5 m = 5000 mm each.
4. Section : Assumed 230x380 mm, d = 345 mm. D_f = 140 mm.
5. Loads : It is proposed to illustrate here analysis and design by exact method. Therefore, maximum load (DL+LL) and minimum load (DL only) will be calculated separately.

-Slab Left -S3:
Type of slab - One-way ; L_x = 4.0 m.
End condition - Continuous at both ends.
End shear factor = 0.5 ; Load Type - UD load.
DL + LL = 6.75 kN/m². ; Only DL = 5.25 kN/m².

-Slab Right-S4:
Type of Slab - Two-way ; L_x = 4.5 m.
End condition- Continuous over B22-B23 and Simply Supported at opposite end.
End shear factor = 5/8*; Load Type- Trapezoidal.
DL + LL = 6.75 kN/m². ; Only DL = 5.25 kN/m².
Equivalence factor for converting trapezoidal load into UD load $C_1 = [1-1/(3B^2)]$.
$B = L_y/L_x = 5.0/4.5 = 1.11$.
$\therefore C_1 = [1-1/(3x1.11^2)] = 0.73$.

-Self (beam) w_b = 1.6 kN/m. Wall load - Nil.
-Maximum load $w_{u.max}$ = 1.5(DL+LL) = 1.5(self+slab1+slab2)
$w_{u.max}$=1.5 [1.6+6.75x4/2+(5/8)x6.75x4.5x0.73]=43.44kN/m.
$w_{u.min}$=0.9 DL=.9 [1.6+5.25x4/2+(5/8)x5.25x4.5x.73]=20.59kN/m.

6. Analysis for Design Moments :

(A) Analysis by Exact Method without allowing any Redistribution of Moments

In this analysis, we will have to consider all possible loading arrangements to get values of (maximum) design moments at midspan and at support.

Case - I : Maximum Bending Moment at Intermediate Support
This is given by maximum loads on both the spans. For this loading, $M_{u.max} = w_{u.max} L^2/8 = 43.44x5^2/8$ = 135.75 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 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 : $M_{FBA} = 43.44x5^2/8$ = 135.75 kN.m,
$M_{FBC} = 20.59x5^2/8$ = 64.34 kN.m.

Moment Distribution :

	A	B (BA)	B (BC)	C
Span in meters		5.00	5.00	
Load in kN/m		43.44	20.59	
Distribution Factors	:	0.5	: 0.5	:
Initital F.E.M.in kN.m:		135.75	: -64.34	:
Dist. Moments in kN.m:		-35.70	: -35.71	:
Final Moments in kN.m:		100.05	:-100.05	:

Design of Beam B22 - B23 : Continued

Maximum Span Moment :

V_{AB} = 43.44x5/2-100.05/5 = 88.59 kN

x_{max} = 88.59/43.44 = 2.04 m, L_o= 2x2.04 = 4.08 m

M_{max1}= 88.59x2.04/2 = 90.36 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 on the left span. By symmetry, M_{max2} = 90.36 kN.m.

Thus, the design moments are 135.75 kN.m at intermediate support and 90.36 kN.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 = .7x135.75 = 95.03 kN.m.

Midspan moment: V_{AB} = 43.44x5/2 - 95.03/5 = 89.59 kN

x_{max} = 89.59/43.44=2.062m, L_o = 2x2.062=4.124m

M_{max} = 89.59x2.062/2 = 92.37 kN.m.

(C) Analysis by Approximate Method

Design Moment at support as well as at midspan=$w_u L^2/10$

M_{max} = 43.44x5²/10 = 108.60 kN.m.

7. Main Steel :

(A) For Exact Analysis without Redistribution of Moments:

$M_{ur.max}$ = 56.67 kN.m for assumed section — Table 6.1

(a) At Intermediate Support : $M_{u.max}$ = 135.75 kN.m > 56.67 kN.m

∴ The section will be required to be Doubly Reinforced.

M_{u2} = $M_{u.max}$ - $M_{ur.max}$ = 135.75-56.67 =79.08 kN.m

Tension Steel : A_{st1} = $p_{t.max}$ bd. For M15-Fe415, $p_{t.max}$=0.72% — Eq.4.1-15a & 6.3.2(7)

= 0.72x230x345/100 = 571 mm².

A_{st2} = M_{u2}/[0.87f_y(d-d'$_c$)] — Eq.4.1-15b

Assuming d'$_c$ = 35 mm,

A_{st2} = 79.08x10⁶/[0.87x415(345-35)] = 707 mm²

A_{st} = 571 + 707 = 1278 mm².

Compression Steel: d'$_c$/d = 35/345 = say 0.1. For which,

A_{sc} = 1.046xA_{st2}= 1.046x707 = 740 mm². — Table 4.1-3

(b) At Midspan : $M_{u.max}$= 90.36 kN.m > 56.67 kN.m.

∴ Assistance of flange will be taken in resisting bending moment. b_f = L_o/6 + b_w + 6D_f

= 4080/6 + 230 + 6x140 = 1750 mm

For x_u = D_f, M_{u1} = 0.36f_{ck} b_f D_f (d-0.42D_f) — Eq.4.1-20

= 0.36x15x1750x140(345-0.42x140)x10⁻⁶

= 378.64 kN.m > $M_{u.max}$ (=90.36 kN.m)

∴ x_u < D_f and beam acts as rectangular section with b = bf.

Also, $x_{u.max}$ = 0.48 d for M15-Fe415 — Table 4.1-1

= 0.48x345 = 165mm > D_f. ∴ O.K.

Required A_{st} = (0.5x15/415)[1- $\sqrt{1-4.6x90.36x10^6/(15x1750x345^2}$] x1750x345 = 752 mm².

(c) Detailing of bars :

Required A_{st} mm²	At midspan	At intermediate support
At top	-	1278
At bottom	752	740
Provide No. Dia. (Provided A_{st} in mm²)		
At top	2-#10 (157)	2-#10+3-#22 (1297)*
At bottom	4-#16 (804)	4-#16 (804)

* See Note on Page 144.

Design of Beams B22-B23 : Continued

***Note:** Alternatively, only 4-#20 with A_{st} = 1256 mm² can also be allowed (since though it falls short of 22 mm², provided steel at midspan is 52 mm² more. This is equivalent to redistribution of moment of 22x100/1278 = 1.72 %.)

(B) For Exact Analysis with Redistribution of Moments :

(a) At Intermediate Support : $M_{u.max}$ = 95.03 kN.m

Since redistribution of 30% is allowed, $M_{ur.limit}$ = 38.87 kN.m 6.3.2(7)

∴ The section will be required to be Doubly Reinforced.

M_{u2} = 95.03 − 38.87 = 56.16 kN.m

Tension Steel : A_{st1} = $p_{t.limit}$ bd=0.45x230x345/100=357 mm² 6.3.2(7)

A_{st2} = 56.16x10⁶/[0.87x415(345−35)]=502 mm²

A_{st} = A_{st1} + A_{st2} = 357 + 502 =859 mm².

Compression Steel: A_{sc} = $[0.87f_y/(f_{sc}-.45f_{ck})]xA_{st2}$ Eq.4.1-15e

For d'_c/d = 0.1 and $k_{u.limit}$ = 0.3, fsc =340 N/mm². Table 4.1-2

∴ A_{sc} = [0.87x415/(340−0.45x15)]x502=544 mm².

(b) At Midspan : $M_{u.max}$ = 92.37 kN.m > 56.67 kN.m.

∴ The assistance of flange will have to be taken.

b_f = $L_o/6 + b_w + 6D_f$, L_o = 4.124m =4124 mm

= 4124/6 +230 +6x140 = 1757 mm.

As seen in solution (A) above, $x_u < D_f$ even for b_f = 1750mm.

∴ The in this case also, $x_u < D_f$ and the beam acts as rectangular beam with b = b_f.

Required A_{st} = $(0.5x15/415)[1-\sqrt{1-4.6x92.37x10^6/(15x1757x345^2)}]$ x1757x345 = 769 mm².

(c) Detailing of Bars :

			At midspan	At support
Required area in mm²				
	At top		---	859
	At bottom		769	544
Provide	− At top	No.Diam.	2-#12(anchor)	2-#12 + 2-#20
		Area	(226)	(854)
	−At bottom	No.Diam	4-#16	3-#16
		Area	(804)	(603)

It will be observed that though steel at midspan has almost remained same as in Solution − A, the steel at support has reduced consideraly at top as well as on bottom face.

(C) For Approximate Analysis ($w_uL^2/10$ at midspan & at support)

(a) At Support : $M_{u.max}$ = 108.60 kN.m. (This is corresponding to 20% redistribution of moment).

For this, $M_{ur.limit}$ = 49.27 kN.m < $M_{u.max}$ 6.3.2(7)

∴ The section will be required to be doubly reinforced.

M_{u2} = 108.60 − 49.27 = 59.33 kN.m.

Tension Steel:

For dM=20%, $p_{t.limit}$ = 0.6 % and $k_{u.limit}$ = 0.4 6.3.2(7)

∴ A_{st1} = $p_{t.limit}$xbd = 0.6x230x345/100 =476 mm².

A_{st2} = (59.33x10⁶)/[0.87x415x(345−35)]=530 mm².

A_{st} = A_{st1} + A_{st2} = 476 + 530 =1006 mm².

Compression Steel :

For d'_c/d = 0.1 and $k_{u.limit}$ = 0.4, f_{sc} = 345 N/mm². Tab.4.1-2

A_{sc} = [0.87x415/(345−.45x15)]x530 = 566 mm².

(b) At Midspan : $M_{u.max}$ = 108.60 kN.m > $M_{ur.max}$ = 56.67 kN.m.

Therefore, assistance will have to be taken of the flange.

L_o = 0.8L (for $w_uL^2/10$) = 0.8x5000 =4000 mm.

b_f = 4000/6 + 230 + 6x140 = 1737 mm.

7. Main Steel : Solution-(C) Continued

In Solution (A) above, M_{ur1} = 378.64 for $x_u = D_f$ & b_f = 1750 mm. M_{ur1} in this case for b_f = 1737 mm will be just slightly less than 378.64 kN.m while Mu.max at midspan is just 108.60 kN.m. Therefore, beam acts as rectangular section with $b = b_f$.

$$\text{Required } A_{st} = (0.5\text{x}15/415)[1-\sqrt{1-4.6\text{x}108.6\text{x}10^6/(15\text{x}1737\text{x}345^2)}]\text{x}1737\text{x}345 = 911 \text{ mm}^2.$$

(c) Detailing :

		At midspan	At continuous end
Required A_{st} in mm²	At top	---	1006
	At bottom	911	566
Provide	At top - No.Diam.(mm)	2-#10(anchor)	2-#10+3-#20
	- Area mm²	157	1099
	At bottom-No.Diam.(mm)	3-#20	2-#20
		942	628

This steel is more than that required for Solution-(B) allowing redistribution of moments but less than that required for Solution -(A) without redistribution of moments.

The calculations beyond this step have been done for Solution-(C). For Details of reinforcement see **Fig 6.3**

8. Design for Shear :

Since the beam is supporting a two-way slab S4, the equivalent uniformly distributed load for shear is different than that considered for bending. It is given by

$w_{u.eqs} = (q_u L_x/2)[1-1/(2\beta)]$ where $\beta = L_y/L_x$.

$= (6.75\text{x}4.5/2)[1-1/(2\text{x}1.11)]$ = 8.35 kN/m.

Since the slab S4 is continuous over beam B22 - B23 and simply supported at the other end, the above load will be increased by 25%. 5.2.3

Total UD load on beam for shear w_{us} = load from slab S4 + load from slab S3 + self.

w_{us} = 1.5(8.35x1.25+6.75x4/2+1.6) = <u>38.31</u> kN/m.

(a) At Continuous End :

$V_{u.max}$ = 0.6 w_{us} L for design moment of $w_uL^2/10$.

= 0.6x38.31x5 = 114.93 kN.

A_{st1} = 1099 mm², p_t = 100x1099/(230x345) = 1.385 %.

τ_{uc} = 0.65+[(1.385-1.3)/(1.4-1.3)]x(0.67-0.65)=0.667 N/mm² Table-4.2

V_{uc} = 0.667x230x345/1000 = 52.93 kN.

$V_{usv.min}$ = 0.35 x230x345/1000 = 27.77 kN.

$V_{ur.min}$ = 52.93 + 27.77 = 80.70 kN.

V_{uD} = 114.93- 38.31(.23/2+.345)=97.31 kN > $V_{ur.min}$.

∴ Stirrups will be required to be designed.

$V_{us} = V_{uD}-V_{uc}$=97.31-52.93= 44.38 kN.

Assuming 06 - 2 legged Fe250 grade stirrups (Asv = 56 mm²),

Spacing s = 0.87x250x56x345/(44.38x1000) = 95 mm.

Length of Shear Zone Lsl = $(V_{u.max} - V_{ur.min})/w_{us}$

Lsl = (114.93 - 80.70)/38.31 = 895 mm.

∴ Provide pitch 95mm for a length 895mm from centre of support i.e. for a length 895-230/2 = 780 mm from face of support. Provide minimum stirrups ø6 at 150 mm beyond.

(b) At Simply Supported End :

$V_{u.max}$ = 0.45 w_{us} L = 0.45x38.31x5 = 86.20 kN.

A_{st1} = 3-#20 = 942 mm².

p_t = 100x942/(230x345) = 1.187 %.

Design of Beams B22-B23 : Continued

8. Design for Shear Continued :

τ_{uc} = 0.6287 N/mm² by linear interpolation. Table-4.2

V_{uc} = 0.6287x230x345/1000 = 49.89 kN Table-6.1

$V_{usv.min}$ = 27.77 kN

$V_{ur.min}$ = 49.89 + 27.77 = 77.66 kN.

V_{uD} = 86.20 - 38.31(0.23/2+.345) = 68.58 kN.

< $V_{ur.min}$. ∴ Minimum stirrups are sufficient.

Provide Ø6 - 2 legged stirrups at 150 mm. Table-6.1

9. Check for Development Length :

(a) At Continuous End : Maximum bar diameter = 20 mm.

Required L_d = 57 Ø = 57x20 = 1140 mm.

Available L_d = L/4 = 5000/4 = 1250 mm. ∴ O.K.

(b) At Simply Supported End : Maximum bar diameter = 20 mm.

Required L_d = 1140 mm as seen above.

Available L_d = $1.3M_1/V + L_{ex}$. A_{st1} = 942 mm².

M_1 = $0.87f_y A_{st}.d\,[1-(f_y/f_{ck})(A_{st}/b_f\,d)]$ Eq.4.1-4b

= 0.87x415x942x345[1-(415/15)(942/(1730x345))]

= 109.68 x 10^6 N.mm = 109.68 kN.m

V = $V_{u.max}$ = 86.20 kN

L_{ex} = $L_d/3 - b_s/2$ = 1140/3 - 230/2 = 265 mm

Available L_d = 1.3x109.68x1000/86.20+ 265 =1919 mm

> Required L_d = 1140 mm. ∴ O.K.

10. Check for Serviceability :

p_t = $100A_{st}/(b_f d)$ = 100x942/(1737x345) = 0.16 %

$\alpha 1$ = 1.76 Fig.4.4.1

b_w/b_f = 230/1737 < 0.3. ∴ $\alpha 3$ = 0.8 Fig.4.4.3

Basic L/d ratio = 26 for continuous beam. Sect.4.4.1(b)

Allwable L/d ratio = 1.76x0.8x26 = 36.6

Actual L/d ratio = 5000/345 = 14.5. ∴ O.K.

IV(b) Beams B24-B25 : Brief Solution.

Span : 5 m. Two Equal Span - Continuous Beam.
Each span will be designed as a beam continuous at one end and simply supported at the other.

Section: 230x380 mm, d = 345 mm, d'= 35 mm.

Loads : Self + parapet wall + S4 (Trapezoidal)

For bending, w_{ub} = 1.5x[1.6+5+0.45x6.75x4.5(1-1/(3x1.11²))]
= 24.86 kN/m.

For shear, w_{us} = 1.5x[1.6+5+0.45x6.75x4.5(1-1/(2x1.11))]
= 21.17 kN/m

Design Moment : $M_{u.max}$ = 24.86x5²/10 = 62.15 kN.m at support as well as midspan.

(a) At Intermediate Support :

$M_{u.max}$ = $w_u L^2/10$ means 20 % redistribution. 6.3.2(7)

For which, $M_{ur.limit}$ = 49.27 kN.m < $M_{u.max}$

∴ The section shall be doubly reinforced.

M_{u2} = 62.15 - 49.27 = 12.88 kN.m

Tension Steel : A_{st1} = pt.limit bd .

$p_{t.limit}$ = 0.6 % for M15-Fe415 & 20 % redistribution 6.3.2(7)

= .6x230x345/100 = 476 mm²

Design of Beams B24-B25 : Continued

$A_{st2} = (12.88 \times 10^6)/[0.87 \times 415(345-35)] = 115\ mm^2$.
$A_{st} = A_{st1} + A_{st2} = 476 + 115 = 591\ mm^2$.

Compression Steel : For $d'_c/d = 0.1$, and $k_{u.limit}$ 0.4,
$f_{sc} = 345\ N/mm^2$. Table-4.1-2
$\therefore A_{sc} = [0.87 \times 415/(345-0.45 \times 15)] \times 115 = 123\ mm^2$.

(b) At Midspan Section :
$M_{u.max} = 62.15$ kN.m > $M_{ur.max} = 56.67$ kN.m.
$\therefore$ Assistance will be taken of the flange.
$L_o = 0.8 \times 5000 = 4000$ mm.
$b_f = 4000/12 + 230 + 3 \times 140 = 983$ mm.
For $x_u = D_f$, $M_{ur1} = 0.36 \times 15 \times 983 \times 140(345-0.42 \times 140) \times 10^{-6}$ kN.m
$= 212.69$ kN.m > $M_{u.max}$ $\therefore x_u > D_f$ and beam acts as rectangular beam with $b = b_f$.

Required $A_{st} = (0.5 \times 15/415)[1-\sqrt{1-4.6 \times 62.15 \times 10^6/(15 \times 983 \times 345^2)}]$
$\times 983 \times 345 = 522\ mm^2$.

(c) Detailing :

			At midspan	At continuous end
Required	A_{st} in mm^2			
	At top		---	591
	At bottom		522	123
Provide	At top	- No.Diam(mm)	2-#12(anchor)	2-#12+2-#16*
		- area in mm^2	226	628
	At bottom	- No.Diam(mm)	2-#10+2-#16*	2-#10
		- area in mm^2	559	157

* Bent-up from midspan to intermediate support only.

Design for Shear :

(a) At Continuous End : $V_{u.max} = 0.6\ w_{us}\ L = 0.6 \times 21.16 \times 5 = 63.48$ kN
$A_{st1} = 628\ mm^2$- $V_{ur.min} = 64.49$ kN even for A_{st1} = 2-#16 Table-6.1
$\therefore$ Minimum stirrups are sufficient. Provide 06 at 150 mm. Table-6.1

(b) At Simply Supported End : $V_{u.max} = 0.45\ w_{us}\ L$
$= 0.45 \times 21.17 \times 5 = 47.63$ 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.

Check for Development Length :

(a) At Continuous End : Maximum bar diameter = 16 mm.
Required $L_d = 57 \times 16 = 912$ mm.
Available $L_d = L/4 = 5000/4 = 1250$ mm. $\therefore$ O.K.

(b) At Simply Supported End : Maximum bar diameter = 16 mm.
$\therefore$ Required $L_d = 912$ mm.
Available $L_d = 1.3M_1/V + L_{ex}$
Since A_{st} at midspan is fully available at simply supported end, assume $M_1 = M_{u.max}$ at midspan = 62.15 kN.m.
$V = V_{u.max} = 47.61$ kN
$L_{ex} = 189$ for #16 Table-6.1
Available $L_d = 1.3 \times 62.15 \times 1000/47.61 + 189 = 1886$ mm. $\therefore$ O.K.

Check for Serviceability :

For flanged beam, p_t at midspan $= 100 \times A_{st}/(b_f\ d)$
$= 100 \times 559/(983 \times 345) = .165\ \% < 0.4\ \%$.
$\therefore$ Allowable L/d ratio > basic L/d ratio=26 for continuous beam.
Actual L/d ratio = 5000/345 < basic L/d ratio = 26. $\therefore$ O.K.

IV(c) Beam B17-B18 **Reference**

1. Beam Mark : B17-B18
2. Type : Two span continuous beam ABC with unequal spans AB and BC, simply supportd 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 x 750mm, d'= 60mm, d = 690mm, D_f = 140mm.
Span BC : 230mm x 380mm, d'= 35mm, d = 345mm, D_f = 110mm.

5. Loads : For bending - w_{max} = 1.5(DL+LL), w_{min} = 0.9DL.
Span AB : Self + parapet wall + Slab S3(UDL) from one side only.
Load of slab S3 : Dead Load =5.25 kN/m², LL=1.5kN/m². 6.2.3(5)
Total load=6.75 kN/m².
Self weight of beam = 25x.23x(.75-.14) = 3.51 kN/m.
Weight of parapet wall = 5 kN/m.
w_{max} = 1.5 (3.51 + 5 + 0.45x6.75x4) = 31.00 kN/m.
∴ w_{min} = 0.9 (3.51 + 5 + 0.45x5.25x4) = 16.16 kN/m.

Span BC : Self + parapet wall + Slab S1(UDL) from one side and + Slab S2(Tri) from other side.
Load of slab S1 : Total load=5.25 kN/m², LL=0.75kN/m². 6.2.1(5)
Dead load =4.50 kN/m².
Load of slab S2 : Total load=6.00 kN/m², LL=1.50kN/m². 6.2.2(5)
Dead load =4.50 kN/m².
Since slab S2 spans in the same direction as the beam B18, only a triangular load with central ordinate $=qL_x/4$ will be transferred to beam B18.The equivalent UD load = $qL_x/6$.
w_{max} = 1.5(1.6 + 5 + 5.25x1 + 6.0x2.5/6)= 21.52 kN/m.
w_{min} = 0.9(1.6 + 5 + 4.50x1 + 4.5x2.5/6)= 11.68 kN/m.

6. **Design Moments :**

According to exact method of analysis, three different loading cases will be required to be considered. Actual moments 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 : $K = (2x230x750^3/12)/10000 = 1617x10^3$ mm³.
Beam BC : $K = (230x380^3/12)/2500 = 421x10^3$ mm³.
Dis.Factors : d_{BA} = 1617/(1617+421) = 0.793, d_{BC} = 1-.793 = 0.207.

Case - I : Maximum Moment at Intermediate Support - B

For this, there shall be maximum load on both spans.
Load on AB w_1 = 31.00 kN/m, Load on BC w_2 = 21.52 kN/m.

Fixed End Moments : M_{FBA}= 31.00x10²/8 = 387.5 kN.m
M_{FBC}= 21.52x2.5²/8 = 16.81 kN.m

Design of Beam B17-B18 : Continued

Moment Distribution :

	A — B	B — C
Span in metres	10.00	2.50
Load in kN/m	31.00	21.52
Distri. Factors	.793	.207
Initial F.E.M.	387.50	-16.81
Distributed Moments	-293.96	-76.73
Final Moments in kN.m	93.54	-93.54

Shears : Maximum shear at intermediate support occurs in this case.

V_{BA}=31.0x10/2 +93.54/10 = 164.35 kN, V_{AB}=31x10/2-93.54/10=145.65kN

V_{BC}=21.52x2.5/2+93.54/2.5= 64.32 kN, V_{CB}=21.52x2.5/2-93.54/2.5=-10.52 kN

Case - II : Maximum Span Moment in AB

For this, there shall be maximum load on span AB and minimum load on span BC.

Load on AB w_1 = 31.00 kN/m, Load on BC w_2 = 11.68 kN.m

Fixed End MOments: M_{FBA} =387.5 kN.m as in Case-I since load is same.

M_{FBC} = 11.68x2.5²/8 = 9.12 kN.m.

Distribution factors remain unchanged.

Moment Distribution :

	A — B	B — C
Span in metres	10.00	2.50
Load in kN/m	31.00	11.68
Distribution factors	.793	.207
Initial F.E.M.	387.50	- 9.12
Distributed moments	-300.06	-78.32
Final Moments in kN.m	87.44	-87.44

Shears : Shear at support a happens to be maximum in this case.

V_{AB}=31x10/2-87.44/10= 146.26 kN. V_{BA}=31x5+87.44/10 = 163.74 kN

x_{max}= 146.26/31= 4.72 m, L_o= 2x4.72 = 9.44m = 9440 mm.

V_{CB} = 11.68x2.5/2-87.44/2.5=-20.38 kN , V_{BC} = 49.58 kN

Maximum Span Moment in AB : M_{max} = 31x9.44²/8 = 345.32 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 = 16.16 kN/m,Load on BC w_2 = 21.52 kN/m.

Fixed End Moments:M_{FBA} = 16.16x10²/8 = 202 kN.m.

M_{FBC} = 21.52x2.5²/8 = 16.81 kN.m

Distribution factors remain unchanged.

Moment Distribution :

	A — B	B — C
Span in metres	10.00	2.50
Load in kN/m	16.16	21.52
Distribution factors	.793	.207
Initial F.E.M.	202.00	-16.81
Distributed Moments	-146.86	-38.33
Final Moments in kN.m	55.14	-55.14

Design of Beams B17-B18 : Continued

6. Design Moments : Case - III : Continued

Shears : Shear at C will be maximum in this case.

$V_{CB} = 21.52 \times 2.5/2 - 55.14/2.5 = 4.84$ kN,

$V_{BC} = 21.52 \times 2.5/2 + 55.14/2.5 = 48.96$ kN

Since shear in this case is very small the positive moment (=0.544 kN.m) is also very small.

Results :		A	Midspan	B	Midspan	C
	Maximum Moment in kN.m :		345.32	93.54	---	:
	Governing Case :		II	I	III	:
	Maximum Shear :	146.26		164.35 : 64.32		4.84
	Governing Case :	II		I : I		III :

. Main Steel :

(a) At middle of Span AB : $M_{u.max} = 345.32$ 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, $L_o = 9.44$ m = 9440 mm.

$\therefore \ b_f = 9440/12 + 230 + 3 \times 140 = 1436$ mm.

For $x_u = D_f$, $M_{ur1} = 0.36 \times 15 \times 1436 \times 140 \times (690 - 0.42 \times 140) \times 10^{-6}$

$= 685.24$ kN.m $> M_{u.max}$ $\therefore x_u < D_f$.

$\therefore$ Beam acts as rectangular beam with $b = b_f$.

Required $A_{st} = (0.5 \times 15/415)[1 - \sqrt{1 - 4.6 \times 345.32 \times 10^6 / (15 \times 1436 \times 690^2)}] \times 1436 \times 690 = 1445$ mm².

(b) (i) At Intermediate Support (Right side) : $M_{u.max} = 93.54$ kN.m .

Since the beam sections on two sides of support are different, the area of steel will be calculated for both side of the sections. (i) For section 230x380 mm of span BC.

For this section, $M_{ur.max} = 56.67$ kN.m $< M_{u.max}$. Table 6.1

$\therefore$ The section shall be doubly reinforced.

$M_{u2} = 93.54 - 56.67 = 36.87$ kN.m.

Tension Steel : $A_{st1} = p_{t.max}\ bd$, $p_{t.max} = .72\%$ for M15-Fe415 6.3.2(7)

$A_{st1} = 0.72 \times 230 \times 345/100 = 571$ mm²

$A_{st2} = 36.87 \times 10^6 / [0.87 \times 415 \times (345 - 35)] = 330$ mm²

$A_{st} = A_{st1} + A_{st2} = 571 + 330 = 901$ mm².

Compression Steel : For $d'_c/d = 0.1$ and $k_u = k_{u.max} = 0.48$, Table 4.1.2

$f_{sc} = 352$ N/m².

$\therefore A_{sc} = [(0.87 \times 415)/(352 - 0.45 \times 15)] \times 330 = 345$ mm².

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 & small increase in area of compression steel.

(ii) At intermediate support (Left side) - For section 230mmx750mm

$M_{ur.max} = 2.07 \times 230 \times 690 \times 690 \times 10^{-6} = 226.67$ kN.m > 93.54 kN.m

$\therefore$ The section is Singly reinforced.

Tension steel :

$A_{st} = (0.5 \times 15/415)[1 - \sqrt{1 - 4.6 \times 93.54 \times 10^6 / (15 \times 230 \times 690^2)}] \times 230 \times 690$

$= 404$ mm² $>$ minimum steel($= .85 \times 230 \times 690/415 = 325$ mm²)

Design of Beam B17-B18 : Continued

(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

$M_{u.max} = -20.38x2.5/2 - 11.68x2.5^2/8 = 34.60$ kN.m.

$< M_{ur.max} = 56.67$ kN.m.

∴ Required A_{st} = $(0.5x15/415)[1-\sqrt{1-4.6x34.60x10^6/(15x230x345^2)}]$ $x230x345 = 311$ mm².

(d) Detailing :

	Span AB Midspan	At support B Left	At support B Right	Span BC Midspan
Required Ast in mm² At top	---	404	901	311
At bottom	1445	---	345	---
Provide At top -No.Di(mm)	2-#12*	1-#20+2-#12	2-#20+3-#12	3-#12
-area mm²	226	540	967	339
At bottom-No.Di(mm)	4-#20+2-#12	2-#12	1-#20+2-#12	2-#12
-area mm²	1482	226	540	226

* to be discontinued at a section where main bars are provided. Bars 4-#20 at bottom may be discontinued beyond point of contraflexure after extending for a length equal to effective depth

8. **Design for Shear:**

(a) Left Support: $V_{u.max}$ = 146.26 kN for Case - II.

A_{st1} = 1482 mm², p_t=100x1482/(230x690)=.93 %

τ_{uc} = .586 N/mm² by linear interpolation. Table-4.2

V_{uc} = .586x230x690/1000 = 93.00 kN

$V_{usv.min}$ = .35x230x690/1000 = 55.54 kN

$V_{ur.min}$ = 93.00 + 55.54 = 148.54 kN > $V_{u.max}$.

∴ Minimum stirrups are sufficient. Assuming #8 2 legged stirrups

Spacing , s = (2x50)x415/(0.4x230)= say 450 mm.

< 0.75x690. ∴ O.K.

(b) Intermediate Support-left side :

$V_{u.max}$ = 164.35 kN for Case-I.

A_{st1} = 540 mm². p_t=100x540/(230x690)=.34%.

τ_{uc} = 0.4 N/mm² by linear interpolation. Table-4.2

V_{uc} = 0.4x230x690/1000 = 63.48 kN

$V_{usv.min}$= 55.54 kN as calculated above.

$V_{ur.min}$ = 63.48 + 55.54 = 119.02 kN.

Since this is less than $V_{u.max}$, V_{uD} will be calculated.

V_{uD}=164.35 - 31x(.23/2+.69)= 139.40 kN. >$V_{ur.min}$.

∴ stirrups will be required to be designed.

V_{us} = V_{uD}-V_{uc} = 139.40 - 63.48 = 75.92 kN

Assuming #8mm-2 legged stirrups of Fe415 grade

Spacing,s = 0.87x415x100x690/(75.92x1000)=328mm say 300mm

L_{s1} =($V_{u.max}$ -$V_{ur.min}$)/w_u=(164.35-119.02)/31 = 1.46m

Provide #8 at 450mm minimum stirrups for remaining part.

(c) Intermediate Support- right side:

$V_{u.max}$ = 64.32 kN for Case-I.

A_{st1} = 967 mm², p_t = 100x967/(230x345)=1.22%.

τ_{uc} = .634 N/mm² V_{uc} = .634x230x345/1000=50.30 kN Table-4.2

$V_{usv.min1}$= 27.77 kN Table-6.1

$V_{ur.min}$ = 50.30 + 27.77 =78.07 kN > $V_{u.max}$

∴ Minimum stirrups are sufficient. Provide φ6 at 150 mm. Table-6.1

(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 throught the span BC. — Reference

9. Check for development length

The method of checking development length for a continuous beam has already been shown. This check is normally satisfied in case of beams because L/d ratio obtained from bending moment consideration is very low in comparison with that required for serviceability criteria.

10 Check for Serviceability :

Ast at mid-span = 1182 mm²

Since beam acts as a flanged beam,

$p_t = 100 x A_{st}/(b_f\ d)$

$= 100x1482/(1436x690) = 0.15\ \% < 0.4\ \%$ — Sect.5.3.3(10)

. . Available L/d ratio > basic L/d ratio (=26) — Sect.4.4.1 (b)

Actual L/d ratio= 10000/690 < 26 .'. O.K.

For details of reinforcement see Fig. 6.3

Comments : From practical point of view it may be felt that the same section of 230x750 mm for both spans would be preferable. The detailed analysis will show that if the section is kept uniform, and taking moment of inertia of larger span as flanged section while that of shorter span as rectangular section (since it is likely to be under tension throughout), the stiffness of the shorter span increases considerably resulting in large support moment. Even with 30% redistribution the support moment is 100% greater than that obtained for stepped section. The comparison of design moment and shear is given below.

a) Design Bending moment in kN.m

		A	Span Moment	B	Span Moment	C
i)	Uniform Sec.		301.27	-185.28	-81.15	0
ii)	Stepped Sec.	0	345.32	-93.54	0.544	0

b) Design Shear force in kN

		A	B		C
i)	Uniform Sec.	136.67	173.53	104.59	-56.55
ii)	Stepped Sec.	146.26	164.35	64.32	4.85

A comparison of quantity of concrete and steel will show that 10% excess concrete is required for uniform section while quantity of steel is almost same. But proper anchoring arrangement needs to be provided to prevent lifting of simply supported end of short span. As such the stepped section provided is considered to be appropriate.

IV(C) Beam - B5

1. Beam mark : B5
2. Beam Type : 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 230m x 380 mm. d' = 35 mm , d = 345 mm.
5. Loads :

I- UD Load: It is different over different lengths.

(a) Upto Beam B27 for a length = 4.5-3.26=1.24m from left support:

w_1 = Self + parapet wall + slab S2 (Trapezoidal)

Since part of triangular load of slab S2 having central ordinate = $qL_x/4$ has been assumed to be transferred to beam B27, the load on beam B5 in this length upto B27 will be reduced by this much amount.

Total Load from S2 over a length 1.24 m will

= 6x2.5x1.24/2 − (1/2)x(6x2.5/4)x2.5/2 = 6.96 kN.

Load from S2 per metre length = 6.96/1.24=5.61kN/m

w_{u1} = 1.5(1.6 + 5 + 5.61) = <u>18.32</u> kN/m.

(b) From Beam B27 to right end for a length 3.26 metres:

w_2 = self + wall of staircase 3.2 metres high above terrace level+load from cap slab S5 coming on this wall length. Load on wall coming from slab S5 is load over trapezoidal area (since Slab S5 is a two-way slab).

= 5x3.26x2.5/2 − 5x(2.5/2)x(2.5/2) = 12.56 kN. This load acts over the length 3.26 metres.

∴ Load from S5 per metre length=12.56/3.26= 3.85kN/m

w_{u2} = 1.5(1.6 + 5x3.2 + 3.85) = <u>32.18</u> kN/m.

II- Point Load = End shear from Beam B27 = 45.48 kN acting at a distance 1.24 metres from left support (Refer to design of Beam B27 Step 8 in Sect. 6.3.3.).

6. Design Moments :

Since the beam is subjected to loads of different intensities over different lengths, the fixed end moments will be obtained by using coefficients given in Table 2.3(b)(page 23) under Case-6 for a UD load over a part of the length and by Case-1 for non-central point load. These coefficients are reproduced here for benifit of readers.The beam is continuous only at left end but simply supported at the right end. Besides, fixed end moment on other side of left support from beam B4 is also different. Therefore,the moments in beam will be obtained by moment distribution method.

Fixed End Moments :

L = 4.5 metres; k_1L = 1.24 metres ∴ k_1 = 1.24/4.5 = 0.275;

k_2L = 3.26 metres ∴ k_2 = 3.26/4.5 = 0.725;

w_{u1}= 18.32 kN/m; w_{u2} = 32.18 kN/m ; W_u = 45.48 kN.

$$M_{FAB} = [k_1(3k_1^2-8k_1+6)/12]x(w_{u1}k_1L)L + [k_2^2(4-3k_2)/12]x(w_{u2}k_2L)L + [k_1(1-k_1)^2]W_uL$$

$$= [0.275(3x.275^2 - 8x.275 + 6)/12]x(18.32x.275x4.5)x4.5 + [0.725^2(4-3x.725)/12]x(32.18x.725x4.5)x4.5 + [0.275x.725^2]x45.48x4.5 = 9.415 + 37.766 + 29.583$$

$$= 76.76 \text{ kN.m}$$

$$M_{FBA} = [k_1^2(4-3k_1)/12]x(w_{u1}k_1L)L + [k_2(3k_2^2-8k_2+6)/12]x(w_{u2}k_2L)L + [k_1^2(1-k_1)]W_uL$$

$$= [0.275^2(4-3x.275)/12]x(18.32x.275x4.5)x4.5 + [0.725(3x.725^2 - 8x.725 + 6)/12]x(32.18x.725x4.5)x4.5 + [0.275^2x.725]x45.48x4.5 = 2.04 + 50.72 + 11.22$$

$$= 63.98 \text{ kN.m}$$

Consider beam B4 to the left of beam B5 as AO of span L=4m.Load on beam B4 as obtained from design of beam B4 (page 140) = 21.15 kN/m

∴M_{FAO} = 21.15x4²/12 = 28.20 kN.m.

Distribution Factors :

It may be noted that Beam AO is a flanged beam for the purpose of stiffness. Its stiffness will, therefore, be taken equal to two times that of a rectangular beam. While for beam AB there is no slab at all and therefore its stiffness willbe that of a rectangular section. K_{AO} = 2x4EI/4, K_{AB} = 3EI/4.5.

∴ d_{AB} = (3EI/4.5)/(3EI/4.5 + 8EI/4) = 0.25; d_{AO} = 0.75.

Design of Beam B5 : Continued Reference

Moment Distribution :

	O–A	A–B		–B
Span in meters	4.00	4.50		
Point Load & dis.from supports		1.24	45.48	3.26
UD Load in kN/m	21.15	18.32		32.18
Distribution factors	0.75	0.25		
Initial F.E.M. in kN.m	28.20	-76.76		63.98
Release B & C.O. to A		-31.99		-63.98
	28.20	-108.75		0
Distributed Moments	60.41	20.14		
Final Moment in kN.m	88.61	-88.61		0

Shears : $V_{u\,BA}$ = [32.18x3.26x(4.5 - 3.26/2) + 45.48x1.24 + 18.32x1.24x1.24/2]/4.5 - 88.61/4.5 = 62.88 kN.

$V_{u\,AB}$ = 32.18x3.26+18.32x1.24+45.48 - 62.88 =110.22 kN.

Assuming point of maximum span moment to lie between the point load and the right support,

x_{max} = 62.88/32.18 = 1.954 m < 3.26 m. ∴ O.K.

$M_{u.max}$ = 62.88x1.954-32.18x1.954x1.954/2= 61.43 kN.m

Redistribution of Moments :

Let redistribution of moments may be done to the extent the moment at midspan and at support are nearly equal. Allowing approximately 20% redistribution,design moment at support will be = 0.8x88.61 = <u>70.89</u> kN.m.

Reduction in support moment = .2x88.61 = 17.72 kN.m

Increase in shear at B = 17.72/4.5 = 3.94 kN.

Revised V_{uBA} = 62.88+3.94 = 66.82 kN.

Revised x_{max} = 66.82/32.18 = 2.076 m.

Revised $M_{u.max}$= 66.82x2.076/2=<u>69.36</u> kN.m.

Revised V_{uAB} =110.22 -3.94 =<u>106.28</u> kN.

7. **Main Steel :**

(a) At Continuous End: For 20% redistribtion, $M_{ur.limit}$=49.27kN.m

$M_{u.max}$ = 70.89 kN.m > $M_{ur.limit}$ = 49.27 kN.m 6.3.2(7)

∴ The section shall be doubly reinforced.

M_{u2} = 70.89 - 49.27 = 21.62 kN.m.

Tension Steel :

A_{st1} = $p_{t.limit}$ bd.

$p_{t.limit}$ = 0.6 for 20 % redistribution of moments. 6.3.2(7)

∴ A_{st1} = 0.6x230x345/100 = 476 mm².

A_{st2} = 21.62x10⁶ /[0.87x415x(345-35)] = 193 mm².

A_{st} = A_{st1} + A_{st2} = 476 + 193 = 669 mm².

Compression Steel :

For d'_c/d = 0.1 and k_u = $k_{u.limit}$ = 0.4, f_{sc}= 348 N/mm². Tab.4.1-2

∴ A_{sc} = [0.87x415/(348 -.45x15)]x193 = 204 mm².

(b) At Midspan :

$M_{u.max}$= 69.36 kN.m > $M_{ur.max}$ = 56.67 kN.m.

Since there is no slab to act as flange, the beam is a rectangular beam. It will, therefore, be doubly reinforced.

M_{u2} = 69.36 - 56.67 = 12.69 kN.m.

Design of Beam B5 : Continued — Reference

Tension Steel : $A_{st1} = P_{t.max}\, bd. = 0.72x230x345/100 = 571\ mm^2$
$A_{st2} = (12.69/21.62)x193 = 113\ mm^2$ proportionately.
$A_{st} = 571 + 113 = 684\ mm^2$.
Compression Steel : $A_{sc} = 0.87x415x113/(352-0.45x15) = 118\ mm^2$

(c) Detailing :

Required area of steel in mm^2	At Continuous End	At Midspan
- at top	669	118
at bottom	204	684
Provide - at top : No.Di(mm)	*2-#12+1-#12 Bt from B4 +2-#16 Bt.from midspan	2-#12
Area mm^2	741	226
- at bottom:No.Di(mm)	2-#12	3-#12+2-#16@
area mm	226	741

* anchor bars continued from top of midspan of this beam.
@ bars bent to top towards continuous end only.

8. Design for Shear :
(a) At Continuous End : $V_{u.max} = 106.28$ kN.
$A_{st1} = 741\ mm^2$. $p_t = 100x741/(230x345) = .93$ %.
$T_{uc} = .586\ N/mm^2$ by linear interpolation. — Tab. 4.2
$V_{uc} = .586x230x345/1000 = 46.50$ kN, $V_{usv.min} = 27.77$ kN — Tab. 6.1
$V_{ur.min} = 46.50 + 27.77 = 74.27$ kN.
Since this is less than $V_{u.max}$, design shear V_{uD} at critical section will be calculated.
$V_{uD} = 106.28 - 18.32x(.23/2+.345) = 97.85\ kN > V_{ur.min}$
∴ Stirrups will be required to be designed.
$V_{us} = 97.85 - 46.50 = 51.35$ kN.
Assuming ø 6 - 2 legged Fe250 grade stirrups ($Asv = 56\ mm^2$),
Spacing s = 0.87x250x56x345/(51.35x1000) = say 80 mm.
$L_{S1} = (V_{u.max} - V_{ur.min})/w_u = (106.28 - 74.27)/18.32$
= 1.75 m>1.24 m the distance of point load from A.
∴ L_{S1} = 1.24 m only.
Shear to the right of point load = 106.28-18.32x1.24 - 45.48
= 38.08 kN < $V_{ur.min}$.
∴ Minimum stirrups are sufficient beyond point load.
Provide ø6 at 150 as minimum stirrups. — Tab. 6.1
(b) Simply Supported End : $V_{u.max} = 66.82$ kN.
$A_{st1} = 741\ mm^2$, $p_t = 100x741/((230x345) = 0.93\%$, $T_{uc} = 0.586\ N/mm^2$
$V_{uc} = .586x230x345/1000 = 46.50$ kN, $V_{ur.min} = 46.50+27.77 = 74.27$ kN
∴ $V_{ur.min} = 74.27\ kN > V_{u.max}$. ∴ Minimum stirrups are sufficient for full length upto point load.

9. Check for Development Length :
(a) At Continuous End : Maximum bar diameter = 16 mm.
Required L_d = 57x16 = 912 mm.
Available L_d = L/4 = 4500/4 = 1125 mm. ∴ O.K.
(b) At Point of Contraflexure(PC) : Maximum bar diameter = 16 mm.
Required L_d = 912 mm. Available $L_d = M_1/V + L_{ex}$.
Since full A_{st} at bottom is available at point of contraflexure, M_1 may be taken equal to $M_{u.max}$ at midspan x $A_{st.prov}/A_{st.req}$
M_1 = 69.36x741/684 = 75.14 kN.m
∴ V = V_u at PC = $V_{u.max}$ (approximately) at simply supported end.
∴ V = 66.82 kN. L_{ex} = d = 345 mm at PC.
∴ Available L_d = 75.14x1000/66.82+345 = 1469 mm > 912 mm ∴ O.K.

Design of Beam B5 : Continued Reference

9. Check for Development Length : Continued
   (c) At Simply Supported End : Maximum bar diameter = 16 mm.
   Required L_d = 912 mm. Available L_d = $1.3M_1/V + L_{ex}$
   Since full A_{st} is also available at simply supported end,
   M_1 = 75.14 kN.m as at PC and V = $V_{u.max}$ = 66.82 kN.
   L_{ex}= 189 at simply supported end for #16mm. Tab. 6.1
   ∴ Available L_d=1.3x75.14x1000/66.82+189=1651mm > 912mm. ∴ O.K.

10. Check for Serviceability :
   p_t at midspan = 100x684/(230x345) = .86 % < 0.87. ∴ α_1 >1. 4.4.1
   Basic L/d ratio = 26 for continuous beam.
   ∴ Allowable L/d ratio > 26
   Actual L/d ratio = 4500/345 < 25. ∴ O.K.

IV(d) Beam - B10

1. Beam Mark : B10
2. Type : Beam of 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 = 4500 mm.
4. Section : Assumed 230mm x 380mm,d'=d'$_c$ =35mm, d=345mm,D_f=140mm.
5. Loads : For a length of 1.24 m from left support, load w_{u1} is same as that for Beam B5 except that there is no parapet load on this beam.
   ∴ w_{u1} = 1.5(1.6+5.61) = 10.82 kN/m.
   Load w_{u2} = 32.18 kN/m same as that for beam B5.
   Pt. Load W_u= 45.48 kN is same as that for beam B5.
   Additional load from two-way slab S4 is triangular.
   Equivalent UD load for bending w_{ub3} = q_u $L_x/3$.
   w_{ub3} = 6.75x4.5/3 = 10.12 kN/m.
   Equivalent UD load for shear w_{us3} = q_u $L_x/4$.
   w_{us3} = 6.75x4.5/4 = 7.60 kN/m.

   For Bending : Besides Point Load of 45.48 kN,
   Total UD load on left 1.24 m =10.82+10.12=20.94kN/m,
   Total UD load on right 3.26 m=32.18+10.12=42.30kN/m.
   For Shear : Besides Point Load of 45.48 kN,
   Total UD load on left 1.24 m = 10.82+7.6 =18.42kN/m,
   Total UD load on right 3.26m = 32.18+7.6 =39.78kN/m.
6. Design Moments :
   Fixed End Moments :
   Fixed end moments due to w_{u2} and W_u are same as those for beam B5. Moment due to w_{u1} will be reduced proportionately & moments due to wub3 will be added.
   M_{FAB} = (10.82/18.32)x9.415 +37.766 +29.583 +10.12x4.5²/12
   = 89.99 kN.m
   M_{FBA} = (10.82/18.32)x2.04 + 50.72 + 11.22 + 10.12x4.5²/12
   = 80.04 kN.m
   To the left of A, beam is B9 of span 4.0 metres continuous at the other hand and carrying a UD load w_u = 20.40 kN/m.
   M_{FAO} = 20.40x4²/12 = 27.20 kN.m

Design of Beam B10 : Continued

6. Design Moments : Continued

Distribution Factors :

Since beam B10 is also now acting as flanged beam, stiffnesses of both the beams will be multiplied by two,

K_{BA} = 2x3EI/4.5, K A0 = 2x4EI/4.

∴ d_{BA} = (6EI/4.5)/(6EI/4.5+8EI/4) = 0.4, d_{AO} = 0.6.

Moment Distribution :

	0		A		B
Span in metres	4.0		4.50		
Point Load & Dis.from Supports			1.24:45.48:	3.26	
UD Load for bending	20.40		20.94	42.30	
UD load for shear			18.42	39.78	
Distribution factors		0.60	0.40		
Initial F.E.M.		27.20	- 89.99	80.22	
Release B & C.O.to A			- 40.11	-80.22	
		27.20	-130.10	0	
Distributed Moments		61.74	41.16		
Final Moments in kN.m:		* 88.94	- 88.94		

Shears :

For calculation of shear equivalent UD load for shear will have to be taken i.e. 18.42 kN/m over left 1.24 m and 39.78 kN/m over right 3.26 kN/m. Taking moments of all loads about A,

V_{uBA} = [39.78x3.26x(4.50-3.26/2) + 45.48x1.24 + 18.42x1.24x1.24/2 - 88.94]/4.5 = 78.62 kN.

V_{uAB} = 39.78x3.26 + 45.48 + 18.42x1.24- 78.62 = 119.38 kN.

Assuming point of maximum span moment(zero shear) to lie within a distance of 3.26 m on the right upto point load,

x_{max} = 78.62/39.78 = 1.976 m < 3.26 m. ∴ O.K.

$M_{u.max}$ = 78.62x1.976/2= 77.68 kN.m.

Redistribution of Moments :

Since moment at continuous end is greater than that at midspan and since the section at support behaves only as rectangular section while that at midspan acts as flanged one, redistribution of moments will be done for economy. Allowing 20 % redistribution,

reduction in support moment = 0.2x88.94 = 17.79 kN.m.

Design moment at support after redistribution

= 88.94 -17.79 = 71.15 kN.m

Increase in shear at B = 17.79/4.5 = 3.95 kN.

Modified shear V_{uBA} = 78.62+3.95 = 82.57 kN.

Modified shear V_{uAB} =119.38-3.95 =115.43 kN.

Modified x_{max} = 82.57/39.78 = 2.076 m<3.26 m. ∴ O.K.

Modified $M_{u.max}$ = 82.57x2.076/2= 85.71 kN.m.

7. Main Steel :

(a) At Continuous End : $M_{u.max}$= 71.15 kN.m.

For 20 % redistribution, $M_{ur.limit}$ = 49.27 kN.m 6.3.2(7)

∴ The section will be required to be Doubly reinforced.

M_{u2} = 71.15 - 49.27 = 21.88 kN.m.

Design of Beam B10 : Continued

7. Main Steel : (a) At Continuous End : Continued

Tension Steel : $A_{st1} = p_{t.limit}$ bd.

For M15-Fe415 & 20 % redistribution, $p_{t.limit} = 0.6$ %. 6.3.2(7)

A_{st1} = 0.6x230x345/100 = 476 mm²

A_{st2} = 21.88x10⁶ /[0.87x415x(345-35)] = 196 mm²

A_{st} = A_{st1} + A_{st2} = 476 + 196 = 672 mm².

Compression Steel :

For M15-Fe415, and 20 % redistribution and d'_c/d = 0.1,

f_{sc} = 348 N/mm². Tab.4.1-2

∴ A_{sc} = 196x[0.87x415/(348 - .45x15)] = 208 mm².

(b) At Midspan : $M_{u.max}$ = 85.71 kN.m > $M_{ur.max}$ = 56.67 kN.m. 6.3.2(7)

∴ The assistance will be required to be taken from the flange.

Assuming L_o=0.7L=0.7x4500=3150mm as recommended by the Code. 4.1.5(a)

The beam acts as L-beam since there is slab S4 only of thickness 140 mm on one side.

∴ b_f = 3150/12 + 230 + 3x140 = 912 mm.

For $x_u = D_f$, M_{ur1} = 0.36x15x912x140x(345-0.42x140)x10⁻⁶ Eq.4.1-20

M_{ur1} = 197.33 kN.m > $M_{u.max}$ ∴ $x_u < D_f$.

∴ Beam acts as rectangular section with b = b_f at midspan.

Required A_{st} = (.5x15/415)/[1- $\sqrt{1-4.6x85.71x10^{6}/(15x912x345^{2})}$]
x912x345 = 736 mm².

(c) Detailing :

Required A_{st} in mm²		At Continuous End	At Midspan
- at top		672	---
- at bottom		208	736
Provide at top	-No.Di(mm)	2-#10+1-#12 from B9 +2-#16 bt.from midspan	2-#10 (anchor)
	-area mm²	672	157
at bottom	-No.Di(mm)	2-#12	3-#12+2-#16(Bt)
	-area mm².	226	741

8. Design for Shear :

(a) At Continuous End : $V_{u.max}$ = 115.43 kN.

A_{st1} = 672 mm². p_t = 100x672/(230x345) = .847 %.

τ_{uc} = .5641 N/mm² by linear interpolation. Table 4.2

V_{uc} = .5641x230x345/1000 = 44.76 kN. $V_{usv.min}$ = 27.77 kN. Table 6.1

$V_{ur.min}$ = 44.76 + 27.77 = 72.53 kN < $V_{u.max}$

∴ Design shear will be calculated at critical section.

V_{uD} = V_{umax} − w_ux(b_s/2+d)

= 115.43− 18.42x(.23/2+.345) = 106.96 kN > $V_{ur.min}$.

∴ Stirrups will be required to be designed.

V_{us} = V_{uD} − V_{uc} = 106.96 − 44.76 = 62.20 kN.

Assuming #8 - 2 legged Fe415 grade stirrups (Asv = 100 mm²),

Spacing s = 0.87x415x100x345/(62.20x1000) = 200 mm.

< 0.75x345 and < 450 mm. ∴ O.K.

Length of Shear Zone = L_{s1} = ($V_{u.max}$ − $V_{ur.min}$)/w_u

= (115.43-72.53)/18.42 = 2.33 m > 1.24 m.

Shear to the right of point load = 115.43-18.42x1.24-45.48

= 47.11 kN < $V_{ur.min}$ ∴ L_{s1} = 1.24 only.

(b) At Simply Supported End : $V_{u.max}$ = 82.57 kN.

A_{st1} = 3-#12 for which $V_{ur.min}$ = 62.53 kN. Tab. 6.1

V_{uD} = 82.57-39.78x(.23/2+.345)= 64.27 kN > $V_{ur.min}$.

∴ Stirrups will be required to be designed.

V_{us} = V_{uD} − V_{uc}, V_{uc} for 3-#12 = 34.76 kN Tab. 6.1

V_{us} = 64.27 − 34.76 = 29.51 kN.

Design of Beam B10 : Continued

8. Design for Shear :(a) At simply Supported End : Continued
Assuming Ø6 - 2 legged Fe250 grade stirrups (A_{sv} =56 mm²),
Spacing s = 0.87x250x56x345/(29.51x1000)= 142 say 140 mm.
Length of shear Zone L_{s1}=(82.57-62.53)/39.78 =.5m =500 mm.
In remaining part provide,minimum stirrups Ø6 at 150 mm.

9. Check for Development length :
(a) At Continuous End : Maximum bar diameter 16 mm.
Required L_d = 57x16 = 912 mm.
Available L_d = L/4 = 4500/4 = 1125 mm. .'. O.K.
(b) At Simply Supported End : Maximum bar diameter = 12 mm.
Required L_d = 57x12 = 684 mm.
Available L_d = 1.3M1/V + L_{ex}. L_{ex} = 113mm for #12 bar. Tab. 6.1
M_1 = M_{ur} for 3-#12 = 37.23 kN.m,V = $V_{u.max}$= 82.57 kN. Tab. 6.1
Available L_d = 1.3x37.23x1000/82.57 + 113 = 700 mm..'. O.K.

10. Check for Serviceability :
Required p_t at midspan = 100 A_{st}/(b_f d) for a flanged beam.
p_t = 100x736/(912x345) = .234 % < 0.4 %. 5.3.3(10)
.'. Allowable L/d ratio > basic L/d ratio(=26)for continuous beam 4.4.1(b)
Actual L/d ratio 4500/345 < 26 . .'. O.K.

Note: Since the moment at A0 has changed cosiderably,the distribution of bending moment and shear force for beam B9 will be different than the one worked out earlier. Even small negative moment of magnitude 11.4 kN.m prevails over the whole span. The effect of this change in moment will not materially affect the bending moment in B8 but the shear reinforcement and steel for negative moment in the region of beam B9 needs to be provided. To cater for this requirement same shear reinforcement and negative steel will be provided in both the spans over the continuous support of B9-B10 junction. The anchor bars will be sufficient to resist the negative moment occuring in the span region. However, the change in reaction at the end of B9 is ignored.

The same type of arrangement of reinforcement will be made over the continuous support of B4-B5 viz.the negative moment steel and shear reinforcement calculated at the support of B5 will be provided at the common support of B4-B5.

Table 6.3 : SCHEDULE OF BEAMS

Cat. No.	Gr.	EC No.	Beam Mark	Span m	Section b x D mm mm	Bottom Bars St. N-Di.	Bottom Bars Bt. N-Di	Top Bars St. N-Di	Stirrups Des. Di-s	Stirrups Lsl mm	Min. Di-s
I	(a)	1	B26	2.5	230x380	2-#12	1-#12	2-#10	-	-	Ø6-150
	(b)	1	B27	2.5	230x380	2-#12	1-#12	2-#10	-	-	Ø6-150
	(c)	1	B11	4.5	230x380	4-#16	-	2-#12	-	-	Ø6-150
	*(d)	1	B19,B20,B21	10.0	230x750	6-#25	-	2-#12	#8-230	1790	#8-450
II	(a)	2	B1	4.0	230x380	2-#12	1-#12	2-#10	-	-	Ø6-150
	*(b)	2	B6	4.0	230x380	2-#12	1-#12	2-#10	-	-	Ø6-150
	(c)	2	B12	4.0	230x380	2-#12	1-#12	2-#10	-	-	Ø6-150
	(d)	2	B16	4.5	230x380	3-#12	2-#12	2-#12	-	-	Ø6-150
III	(a)	3	B2,B3,B4	4.0	230X380	2-#12	1-#12	2-#10	-	-	Ø6-150
	*(b)	3	B7,B8,B9	4.0	230X380	2-#12	1-#12	2-#10	-	-	Ø6-150
	(c)	3	B13,B14,B15	4.0	230X380	2-#12		2-#10+1-#10	-	-	Ø6-150
IV	*(a)	4	B22,B23	5.0	230X380	2-#20	1-#20	02-#10+1-#20	Ø6-90	720	Ø6-150
	(b)	4	B24,B25	5.0	230X380	2-#10	2-#16	2-#12	-	-	Ø6-150
	*(c)	4	B17	10.0	230X750	4-#20+2-#12		2-#12+1-#20	#8-300	1500	#8-450
	*(c)	4	B18	2.5	230X380	3-#12	-	2-#10+1-#20	-	-	Ø6-150
	(d)	4	B5	4.5	230X380	3-#12	2-#16	2-#12+1-#12	Ø6-80	1240	Ø6-150
	*(e)	4	B10	4.5	230X380	3-#12	2-#16	2-#10+1-#12	#8-200	1240	Ø6-150

* For Details see Fig. 6.3

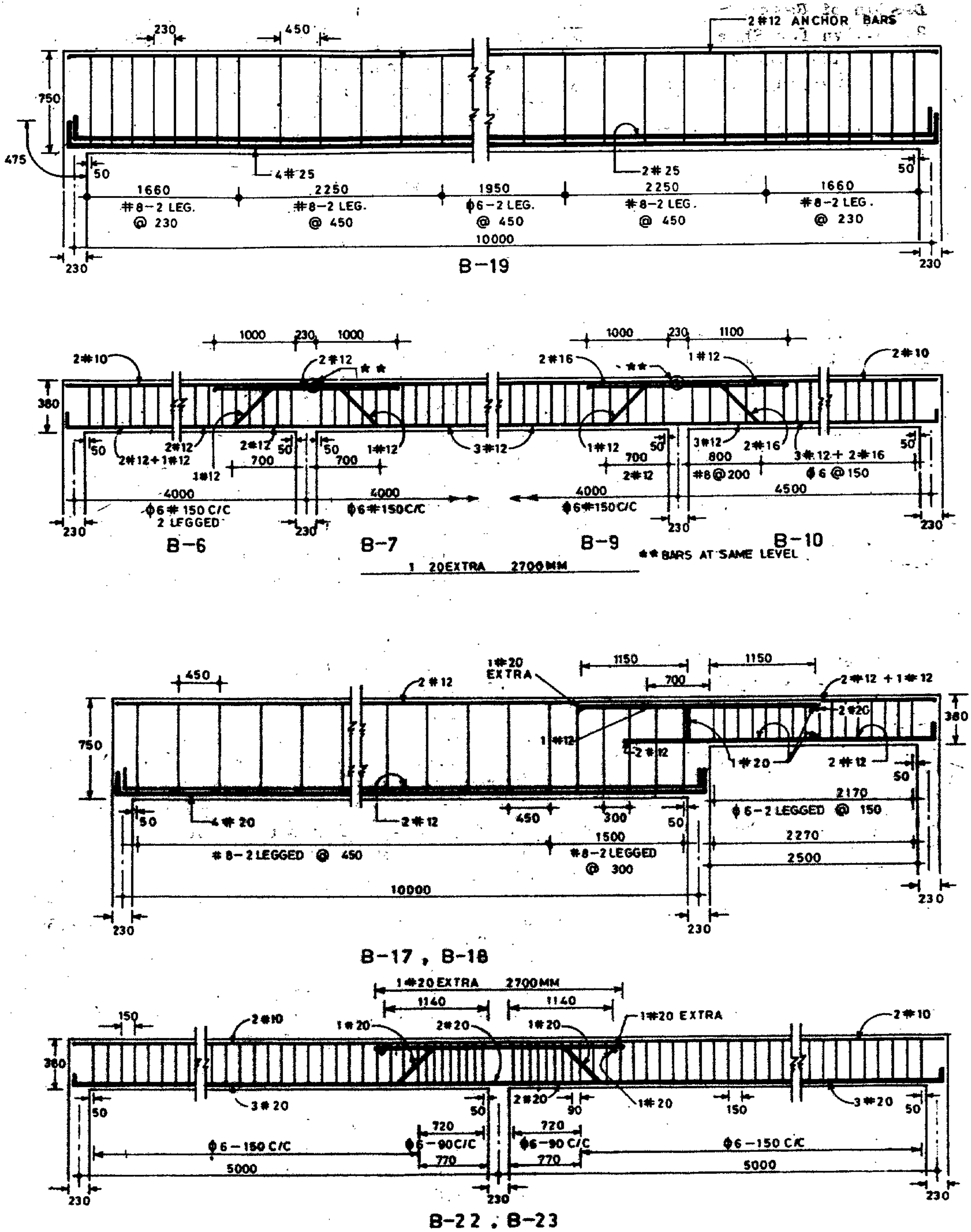


Fig. 6.3 Details of Beams

6.4 DESIGN OF COLUMNS

6.4.1 Categorisation 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 : C8, C9, C10, C16, C17, C18, C13, C14.
(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, C6, 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.

6.4.2 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 loadson 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 loads thus obtained are shown in Table 6.4(a). The beam directions are shown in adajacent figure. The beam shears are shown in Fig. 6.4.

```
      1
      |
4-----C-----2
      |
      3
```

Table 6.4(a) : Loads on Columns

Category	Column Mark	Beam End Shears in kN from directions 1	2	3	4	Total kN	say kN
I	C2	-	42.30	-	50.76	93.06	93
	C3	-	42.30	-	42.30	84.60	85
	C4	-	42.30	-	42.30	84.60	85
II	C5	-	106.28	-	42.30	148.58	149
	C8	-	27.30	260.00	32.76	320.06	320
	C9	-	27.30	260.00	27.30	314.60	315
	C10	-	27.30	260.00	27.30	314.60	315
	C13	114.93	56.66	114.93	-	286.52	287
	C14	63.48	-	63.48	56.66	183.62	184
	C16	260.00	19.80	-	23.76	303.56	304
	C17	260.00	19.80	-	19.80	299.60	300
	C18	260.00	19.80	-	19.80	299.60	300
III	C1	-	38.07	4.84	-	42.91	43
	C6	-	-	53.25	66.82	120.07	120
	C7	64.32	24.57	164.35	-	253.24	253
	C11	-	115.43	86.20	27.30	228.93	229
	C12	53.25	-	47.63	82.57	183.45	183
	C15	146.26	17.82	-	-	164.08	164
	C19	86.20	57.48	-	19.80	163.48	164
	C20	47.63	-	-	43.11	90.74	91

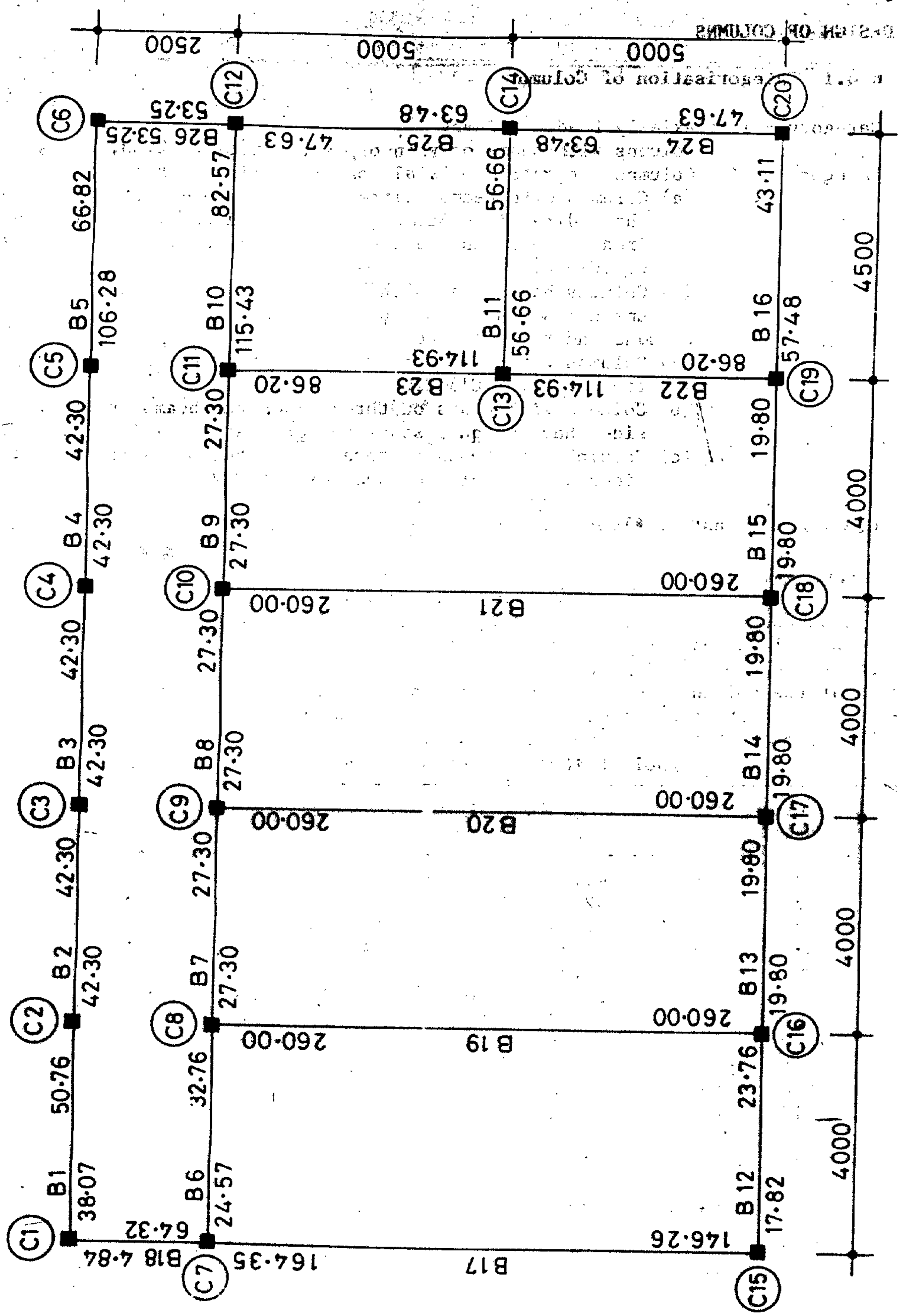


FIG. 6.4 BEAM END SHEARS ON COLUMNS

(b) Grouping of Columns

The columns under each category having loads within a variation of 10 to 20% have been grouped to-gether to reduce calculations. There is single design calulation for each group for maximum load.

Category No.	Group	Column Marks	$P_{u.max}$ kN	-	$P_{u.min}$ kN
I	-	C2,C3,C4	93	-	85
II	(a)	C8,C9,C10,C16,C17,C18	320	-	300
	(b)	C13	287		
	(c)	C5,C14	184	-	149
III	(a)	C7,C11	253	-	235
	(b)	C12,C19,C15	183	-	164
	(c)	C1,C6,C20	120		43*

*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 230x230 mm 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.

6.4.3 Determination of Effective Length and Slenderness

Since even the heaviest load on the column being 320 kN which is less than the axial load carrying capacity of minimum size of 230mm x 230mm with minimum steel (4-#12) for such public buildings, it is proposed to use this single section for all columns.The change in reinforcement will cater for the requirements of bending 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 or 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. Calculatioins 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.
Total height of column above foundation bed = 4520 mm.
Depth of shallowest beam = 380 mm.
Unsupported length of column L = 4520 - 380 = 4140 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. 4.5.1(c)(ii).

∴ Effective length of column L_{eff} = 4140 mm.
Assumed Section b x D (in mm) = 230mm x 230mm.
Slenderness ratio L_{eff}/b = 4140/230 = 18 > 12.
∴ The column is slender.
Allowance for slenderness = $(1/C_r-1)$x100 %.
where, C_r = 1.25- $L_{eff}/(48b)$ = 1.25-18/48= .875.
Allowance for slenderness = (1/.875 -1)x100 = 14.28 % say 14 %.
Factored self weight of column = 1.5x25x.23x.23x4.14 = 8.2 say 8 kN.

6.4.4 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.5.4.8(a)and allowance for slenderness as explained in Sect.5.4.8(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 No.Diameter combinations of bars as explained in Sect.5.4.9(I)(a). Strength in axial compression is obtained by using following equations.

$$P_u = (P_{uc} + P_{us}) \qquad - I$$

where $P_{uc} = \lambda(0.4 f_{ck}\, bD)$ where $\lambda = 0.9$ for b = 230 mm.

$= 0.9 x 0.4 x 15 x 230 x 230/1000 = 286$ kN.

$P_{us} = \lambda(0.67 f_y - 0.4 f_{ck}) A_{sc} = 0.9(0.67x415 - 0.4x15) A_{sc}/1000 = .245 A_{sc}$

Similarly, values of P_{uz} and P_{ub} are required in calculating additional moment in case of slender columns.

$P_{uz} = 0.45 f_{ck}\, Ac + 0.75 f_y\, A_{sc} = 0.45 f_{ck}\, bD + (0.75 f_y - 0.45 f_{ck}) A_{sc}$

$= [0.45x15x230x230 + (0.75x415 - 0.45x15) A_{sc}]/1000$

$= 357 + .3045 A_{sc} \qquad - II$

Ignoring the contribution of steel to strength in axial compression, P_{ub} can be easily obtained using the following equation.

$P_{ub} = 0.36 f_{ck}\; b(7/11)d = 0.36x15x230x(7/11)x[230-(40 + \phi/2)]/1000$

$= 0.79(190 - \phi/2) = 145$ kN for $\phi = 12$ mm

and $P_{ub} = 144$ kN for $\phi = 16$ mm.

The values of P_{uc}, P_{us}, P_u, P_{uz} and P_{ub} are calculated for different No. Di. Combinations using the above relations, and the same are presented in Table 6.4(b).

Table 6.4(b) : Load carrying capacities Pu Puz and Pub in kN of Axially Loaded Columns Concrete Grade - M15 : Steel Grade Fe 415

Section b x D mm mm	P_{uc}(kN)	N-Di(mm) / A_{sc} mm² / P_{us} (kN)	4-#12	6-#12	4-#16	8-#12	6-#16	8-#16	Remarks
		A_{sc} mm²	452	678	804	904	1206	1608	
		P_{us} (kN)	111	166	197	221	295	394	
230x230	286	P_u (Approx)	397	452	483	507	581	680	by Eq.I
		P_u (Exact)	406	457	498	502	590	664	by Theory
		P_{uz}	495	563	602	632	724	847	by Eq.II
		P_{ub}	145	145	144	145	144	144	

Note : Exact value of P_u is obtained corresponding to minimum eccentricity of 20 mm by using the theory of eccentrically loaded columns. (Refer to Sect.9.5.5 of Authors Textbook. The values are obtained by computer and are given in Table 6.11 of Author's Handbook. The same are reproduced here. 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.6.4.5.1 in design of C2.

The Axial Design loads for equivalent short columns are calculated according to the procedure explained in the beginning of this subsection and the same are outlined in tabular form below.

The main reinforcement is arrived by use of Table 6.4(b).

Design of lateral ties is done on following lines.

Though theoretically, minimum diameter of lateral tie is 1/4 the diameter of main bar or 5 mm whichever is less, in practice, minimum diameter adopted is 6 mm. Therfore, provide 6 mm diameter ties of Grade Fe250 at a spacing equal to least of the following.

Least lateral dimension $b = 230$ mm,
48 times diameter of tie $= 48 \times 6 = 288$ mm, and
16 times diameter of main bar $= 16 \times 12 = 192$ mm for $\phi = 12$ mm, and
$= 16 \times 16 = 256$ mm for $\phi = 16$ mm.

i.e. $s = 190$ mm for main bar of 12 mm diameter and
$s = 230$ mm for main bar of 16 mm diameter.

Cat. No.	Group Column Mark	Section b x D	Floor Load	Self Wt.	Total Load	Pa.f %		Eq.axial load	Pa.s 14%	Des. Load	Main N - ø	St. Ties ø - s
I	C2,C3,C4	230x230	93	8	101	-	-	101	14	115	4-#12	ø6-190
II	(a)C8,9,10, C16,17,18	230X230	320	8	328	15	48	376	53	429	6-#12	ø6-190
	(b)C13	230x230	287	8	295	15	43	338	47	385	4-#12	ø6-190
	(c)C14,C5	230x230	184	8	192	15	28	220	31	251	4-#12	ø6-190
III	(a)C7,C11	230x230	253	8	261	33	84	345	48x2	441	6-#12	ø6-190
	(b)C15,12,19	230x230	183	8	191	33	61	252	35x2	322	4-#12	ø6-190
	(c)C6, 20,1	230x230	120	8	128	33	40	168	24x2	216	4-#12	ø6-190

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 for short column.

6.4.5 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.5.4.8(A)(II) or 5.4.8(B)(II). The check for effect of slenderness will be applied according to the procedure explained in Sect.5.4.8(B)(IV). Alternatively, use the procedure given in Table 5.5(c) of Sect.5.9.8.

6.4.5.1 Category -1 : Columns under Axial Loads only : Slender Columns

Columns C2,C3,C4.

1. Axial Load $P_u = 101$ kN excluding allowances for bending and slenderness since the actual effect of both is now seperately considered theoretically.
2. Assumed Section : 230mm x 230mm with main steel 4 - #12 ($A_{sc} = 452$ mm²).
3. Effective Lengh $L_{effy} = 4140$ mm.
4. Slenderness Ratio $L_{effy}/b = 4140/230 = 18 > 12$. ∴ The column is slender.
5. Smaller End Moment $M_{u1} = 0$ at the footing end since the footing is designed as rotation free i.e. for axial load only.
6. 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.
7. Initial Moment M_i is, therefore, equal to 0 due to external moments.
8. Minimum Eccentricity $e_{min.y} = L/500 + b/30 = 4140/500 + 230/30 < 20$ mm. ∴ 20mm.
9. $M_{u.miny} = P_u \times e_{min.y} = 101 \times 20/1000 = 2.02$ kN.m
10. 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.

11. Ideal Axial Strength P_{uz}=495 kN ... ... Table 6.4(b).

12. Axial Strength for Balanced Section :

I - From first principles :

For balanced section, e_{max} = .0035 and

e_{min} = -.002 at tension steel level.

$\therefore$ x_u =[.0035/(.0035+.002)]d = (7/11)d

d' = 40 + ϕ/2 = 40+12/2 = 46 mm, d = 230 - 46 = 184 mm.

x_u = (7/11)x184 = 117 mm

P_{ub} = P_{uc} + P_{us}

P_{uc} = $0.36f_{ck}$ b x_u = 0.36x15x230x117/1000 = 145.31 kN

P_{us} = $(f_{s1}-f_{c1})A_{s1}+f_{s2}xA_{s2}$, f_{s1} and f_{s2} depend upon strains e_1 and e_2.

e_1 = [(117-46)/117]x0.0035 = .002124, e_2 = -0.002.

f_{s1} = 332.29 N/mm2 and f_{s2} = -327.74 N/mm² ... ... Table 2.10

f_{c1} = 0.45x15 = 6.75 N/mm²

A_{s1} = 452/2 = 226 mm² = A_{s2}.

P_{us} = [(332.29 - 6.75)x226 + (-327.74)x226]/1000= -0.5 kN.

$\therefore$ P_{ub} = 145.31 - 0.50 = 144.81 kN. say 145 kN.

II - Using Design Aids :

P_{ub} = $(k_1 + k_2.p/f_{ck})\ f_{ck}$ bD. .. .. ..

f_{ck} bD= 15x230x230/1000 = 793.5 kN

p/f_{ck}= $100xA_{sc}/(f_{ck}$ bD)=(100x452)/(793.5x1000) = 0.057

d'/D = 46/230 = 0.2

k_1 = 0.184 .. .. .. Table E-6

k_2 = -.022 .. .. .. Table E-6

P_{ub} = (0.184 - 0.022x0.057)x793.5 = 145 kN.

III - Using Approximate Method

P_{ub} = 145 kN as calculated earlier in Table 6.4(b) above. It may be observed that results given by approximate method can be accepted.

13. Reduction factor for additional moment due to slenderness

k = $(P_{uz} - P_u)/(P_{uz} - P_{ub})$ but not greater than 1.

= (495 - 101)/(495 - 145) > 1 . . $\therefore$ k = 1.

14. Additional Moment M_{ay} = (P_u b/2000)(L_{effy}/b)².k

M_{ay} = (101x.23/2000)x18² x1 = 3.76 kN.m.

15. Total Moment on Equivalent Short Column

M_u = M_1 + M_{ay} = 2.02 + 3.76= 5.78 kN.m.

16. Check for Combined effect of P_u= 101 kN and M_u = 5.78 kN.m.

I - By Use of Charts :

For d'/D = 0.2, $P_u/(f_{ck}$ bD) = 101/793.5= 0.127, p/f_{ck} = 0.057, $M_u/(f_{ck}.bD)$ = 0.105 (See Chart No.4 Appendix E-3). Therefore, Moment of Resistance of the section M_{ur}=0.105x793.5x.23=19kN.m.

II- By use of Table in Author's Handbbok: Table 6.14, page 228.

For a section 230 mm x 230 mm with 4-#12,

M_{ur} = 19 kN.m for P_u = 103 kN.m.

> 5.78 kN.m acting on the section. Therefore,

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.

6.4.5.2 *Category -II : Columns under Axial Load and Uniaxial Bending :*

Typical detailed calculations are shown only for column in group (a). For other grups only brief calculations are given.

(a) Columns C8,C9,C10,C16,C17,C18 :

(i) Bending @ (major axis of bending) X-axis :

1. Axial Load : P_u = 328 kN (actual axial load excluding allowances).
2. Section : 230 mm x 230 mm with 6-#12. A_{sc} = 678 mm².
3. Effective Length $L_{eff.x}$ = 4140 mm.
4. Slenderness Ratio $L_{eff.x}/D$ = 4140/230 = 18 > 12. .·. Column is slender.
5. Smaller End Moment M_{u1} = 0 at the base as it is designed as rotation free.
6. Larger End Moment = Moment in the column at top due to partial fixity between the beam and the column. It is obtained as under.

Column : $I_c = 230 \times 230^3/12 = 233.2 \times 10^6$ mm⁴ . L_c = 4140 mm,
$K_c = 0.75 \times 233.2 \times 10^6 / 4140 = 42.25 \times 10^3$ mm³. The stiffness is .75I/L since the lower end of the column is rotation free.

Beam : Section 230mm x 750 mm. L = 10000 mm, beam - flanged, w_u = 52 kN/m.
$I_b = 230 \times 750^3/12 = 8086 \times 10^6$ mm⁴ for rectangular section.
$I_b = 2 \times 8086 \times 10^6$ mm⁴ for flanged section. (See sect.3.2.5).
$K_b = 2 \times 8086 \times 10^6 / 10000 = 1617.2 \times 10^3$ mm³.

Distribution Factor $d_{col} = K_c/(K_c + K_b/2) = 42.25/(42.25+1617.2/2) = 0.0497$.
Fixed End Moment from beam due to partial fixity $M_F = w_u L^2/24$.
.·. $M_F = 52 \times 10^2/24$ = 216.7 kN.m.
$M_{col} = d_{col} \times M_F = 0.0497 \times 216.7$ = 10.77 kN.m.
.·. M_{u2} = 10.77 kN.m.

7. Initial Moment $M_i = 0.6M_{u2} + 0.4M_{u1} = 0.6 \times 10.77 + 0.4 \times 0$ = 6.46 kN.m.
8. Minimum Eccentricity e_{min} = 20 mm as calculated earlier.
9. Minimum Moment $M_{u.min} = P_u \times e_{min} = 328 \times 20/1000$ = 6.56 kN.m > M_i.
10. Modified Initial Moment $M_i = M_{u.min}$ = 6.56 kN.m.

11. Ideal Axial Strength P_{uz} = 563 kN from Table 6.4(b).
12. Axial Strength for balanced section P_{ub} = 145 kN from Table 6.4(b).
13. Reduction Factor for additional Moment due to slenderness
 k = (563 - 328)/(563 - 145) = 0.562.
14. Additional Moment $M_{ax} = (P_u D/2000)(L_{eff.x}/D)^2 \times k$
 $M_{ax} = (328 \times .23/2000) \times 18^2 \times 0.562$ = 6.87 kN.m.
15. Total design Moment for equivalent short column $M_{ux} = M_i + M_{ax}$
 M_{ux} = 6.56 + 6.87 = 13.43 kN.m.
16. Moment of Resistance : M_{urx} is obtained from Table E-2.
 For a section 230 mm x 230 mm with 6-#12, M_{urx} = 17 kN.m for P_u = 344 kN.
 Since actual load is 328 kN, M_{ur} can be taken equal to 17 kN.m.
 .·. $M_{urx} > M_{ux}$. .·. the section is safe.

(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 = 20.4 kN/m , L = 4 m = 4000 mm.
M_F = 20.4x4²10 − 20.4x4²/12 = 5.44 kN.m

Beam B6 : 230x380 mm, K_{bL} = 0.75(230x380³ 12)/4000 = 197.20x10³ mm³.
Beam B7 : 230x380 mm, K_{bR} = (230x380³/12)/4000 = 262.93x10³ mm³.
Column C8:230x230 mm, K_{col} = 0.75(230x230³/12)/4140 = 42.25x10³ mm³.
d_{col} = 42.25/(42.25 + 197.20 +262.93) = 0.084
M_{col} = 0.084x5.44 = .458 kN.m = M_{u2}
$M_{u.min}$ = 6.56 kN.m > M_{u2}. ∴ M_i = 6.56 kN.m.
Since column is square and contribution of steel in P_{ub} is ignored,
M_{ay} = 6.87 kN.m as for bending about x − axis.
M_{uy} = 6.56 + 6.87 = 13.43 kN.m.
M_{ury}=14.1 kN.m for P_u = 364 kN > 328 kN from Table E-2. ∴ Safe.

(b) Category − II : Brief Solution for remaining Columns

Group	(b)		(c)	
Column Mark	C13		C5,C14	
1. P_u in kN	287		149*	
2. Section bxD	230x230		230x230	
N-ø	4-#12		4-12#	
A_{sc}(mm²)	452		452	
(i) Bending @ X − axis				
3. $L_{eff.x}$ mm	4140		4140	
4. $L_{eff.x}/D$	18		18	
5. M_{u1} kN.m	0		0	
6. M_{u2} (a)Beam	Left	Right	Left	Right
	−	B11	B4	B5
Type	−	T	L	Rect
(b) w_u kN/m	−	32.78	21.15	
(c) L m	−	4.5	4.0	4.5
(d) M_F kN.m	−	27.66	28.20	108.5
(e) K_b x10³	−	2x233.2	2x262.93	233.7
(f) K_c x 10³	42.25		42.25	
(g) d_{col}	.153		0.053	
(h) $M_{col}=M_{u2}$	4.22		4.25	
7. M_i kN.m	2.53		2.55	
8. e_{min} mm	20		20	
9. $M_{u.min}$ kN.m	5.74		2.98	
10.Revised M_i kN.m	5.74		2.98	

Group		(b)	(c)
Column Mark		C13	C5,C14
11.P_{uz}	kN	495	495
12.P_{ub}	kN	145	145
13. k		594	99
14. M_a	kN.m	6.35	5.50
15. M_{ux}	kN.m	12.09	8.48
16. M_{urx}	kN.m	> 14.0	>18.70
		Safe	Safe
(ii) Bending @ Y − axis			
M_i	kN.m	5.74	2.98
M_{ay}	kN.m	6.35	5.50
M_{uy}	kN.m	12.09	8.48
M_{ury}	kN.m	> 14.0	>18.70
		Safe	Safe

*Note: Load of column C14 (=184kN) is though greater than that (=149 kN) of column C5, it has not been taken here in calculations because M_{col} for C14 is same as that for C13 which is already being tested for higher load = 287 kN.

6.4.5.3 Category − III : Columns under Axial Load and Biaxial Bending

Column Group		---(a)---	-----(b)-----	----(c)----
Column Mark		C7 (C11)	C12 (C15,C19)	C6 (C20,C1)
. P_u	kN	253 (229)	183 (164,164)	120 (91,43)
. Section bxD	mm	230x230	230x230	230x230
N − ø	mm	6-#12	4-#12	4-#12
A_{sc}	mm²	678	452	452

Column Group		---(a)---	---(b)---	---(c)---
Column Mark		C7 (C11)	C12 (C15,C19)	C6 (C20,C1)
		(i) Bending @ X - axis		
3. $L_{eff.x}$	mm	4140	4140	4140
4. $L_{eff.x}/D$		18	18	18
5. M_{ux1}	kN.m	0	0	0
6. $M_{ux2} = M_{col}$	kN.m			
(a) Beam from		Right	Left	Left
Beam Mark		B6	B10	B5
b mm xD	mm	230x380	230x380	230x380
Beam Type		Flanged	Flanged	Rectangular
L	mm	4000	4500	4500
$K_b x10^3$	mm^3	262 93	467.42	*175.28
w_{u1}	kN/m	20.40	UDL+PTL	UDL+PTL
M_{F1}	kN.m	#13.60	#40.02	#16.09
(b) $K_{col} x10^3$	mm^3	42.25	42.25	42.25
$K_{col}+K_{b1}+k_{b2}$		305.18	509.67	217.53
(c) d_{col}		0.138	0.083	0.194
(d) $M_{col}=M_{ux2}$		1.88	3.32	3.12
7. $M_{i.x}$	kN.m	1.13	1.99	1.87
8. $e_{x.min}$	mm	20	20	20
9. $M_{ux.min}$	kN	5.06	3.66	2.40
10.Revised $M_{i.x}$	kN.m	5.06	3.66	2.40
11.P_{uz}	kN	563	495	495
12.P_{ubx}	kN	145	145	145
13. k		0.74	0.594	1.00
14.M_{ax}	kN.m	6.98	4.05	4.47
15.M_{ux}	kN.m	12.04	7.71	6.87
16.M_{urx}	kN.m	21.08	14.94	19.47

(ii) Bending @ Y - axis

		(a)		(b)	(c)
3. $L_{eff.y}$	mm	4140		4140	4140
4. $L_{eff.y}/b$		18		18	18
5. M_{uy1}	kN.m	0		0	0
6. $M_{uy2} = M_{col}$	kN.m				
(a)Beam from		Left	Right	Left	Left
Beam Mark		B17	B18	B25	B26
b mm x D	mm	230x750	230X380	230x380	230x380
Beam Type		Flanged	Flanged	Flanged	Flanged
L	mm	10000	2500	5000	2500
$K_{b1} x10^3$	mm^3	*1212.9	631.0	420.68	841.37
w_u	kN/m	31.00	21.52	24.86	42.60
M_F	kN.m	387.50	16.81	#25.90	#11.09
(b)$M_F=M_{F1}-M_{F2}$		370.69		25.90	11.09
$K_{col} x10^3$	mm^3	42.25		42.25	42.25
$K_{col}+K_{b1}+K_{b2}$		1886.18		462.93	883.62
(c)d_{col}		0.022		0.091	0.048
(d)$M_{col}=M_{uy2}$	kN.m	8.16		2.36	0.53

* k = 0.75 I/L since opposite end is rotation free.
\# $M_F = w_u L^2/24$ for partial fixity.

Category - III : Columns under Axail Load and Biaxial Bending : Continued

Column Group Column Mark		---(a)--- C7 (C11)	-----(b)----- C12 (C15,C19)	----(c)---- C6 (C20,C1)
7. M_{iy}	kN.m	4.90	1.42	0.32
8. $e_{y.min}$	mm	20	20	20
9. $M_{uy.min}$	kN.m	5.06	3.66	2.40
10. Revised $M_{i.y}$	kN.m	5.06	3.66	2.40
11. P_{uz}	kN	563 :	All values same as those for Bending @ X - axis.	
12. P_{uby}	kN	148 :		
13. k		0.747 :		
14. M_{ay}	kN.m	7.04 :		
15. M_{uy}	kN.m	12.10 :		
16. M_{ury}	kN.m	17.73 :		
17. P_u/P_{uz}		0.45	0.37	0.24
18. AlphaN		1.417	1.28	1.07
19. L.H.S. value		1.03	0.85	0.66
20. Remarks		$Unsafe	Safe	Safe

* K = 0.75 I/L since opposite end is rotation free.

$ Though from above calculations the column appears to be unsafe, the assumed section may be allowed if value of the L.H.S. is upto 1.05 because if exact calculations are done for L_{eff} based on end rotation factors β_1 and β_2, the actual effective length reduces which reduces L_{eff}/b ratio, consequently reducing the additional moment M_a. This makes the section safe.

Table 6.4(c) : Schedule of Columns

Cat. No.	Group	Column Mark	Concrete Grade	Section b(mm)xD(mm)	Main Steel No. Di.(mm)	Ties Di.(mm)- s (mm)
I		C2,C3,C4	M15	230 x 230	4 - #12	ϕ6 - 190
II	(a)	C8,9,10,16,17,18	M15	230 x 230	6 - #12	ϕ6 - 190
	(b)	C13,C14,C5	M15	230 x 230	4 - #12	ϕ6 - 190
III	(a)(b)	C7,C11	M15	230 x 230	6 - #12	ϕ6 - 190
	(c)	C1,6,12,15,19,20	M15	230 x 230	4 - #12	ϕ6 - 190

6.5 DESIGN OF COLUMN FOOTINGS

6.5.1 *Categorisaton 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 : Strap Footing

It may be remembered that though basis of categorisation is same for columns and footings, it is not necessary that the categories of a particular column and the 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. Thus, all column footings are under Category - I only.

6.5.2 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 (or with load difference upto say 25 kN)and having 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.

Category - i : Axially Loaded Column Footings

Group No.	Column Mark	Loads kN	Maximum Loads kN	Self Wt. kN	Design Load kN
1	C8, C9, C10 C16,C17,C18	320,315,315 304,300,300	320	8	328
2	C13	287	287	8	295
3	C7, C11	253,229	253	8	261
4	C14,C12,C19,C15	184,183,164,164	184	8	192
5	C5	149	149	8	157
6	C6	120	120	8	128
7	C2,C20,C3,C4	93, 91,85,85	93	8	101
8	C1	43	43	8	51

6.5.3 Design of Footings

Illustrative design calculations according to the procedure explained in Sect.5.10 have been presented here only for footing under Group-3 i.e.for column Nos.C7,C11 for design load of 261 kN because for this group the check for one-way shear is required to be done twice. The calculations for remaining footings are presented in a tabular form. Reader can very easily check the calculations.

6.5.4 *Illustrated Design of Footing in Group - 3 for Column-C7*

Step No.	Design Calculations	Results
	I - Data :	
1.	Column Mark	C7,C11
2.	Bearing Capacity of Soil	$q = 150\ kN/m^2$
3.	Design Load : Maximum load in the group	$P_u = 261\ kN$
4.	Column Section : (a) Width	$b = 230\ mm$
	(b) Depth	$D = 230\ mm$
5.	Material Details: (a) Concrete Grade	M15
	(b) Steel Grade	Fe415
	(c) Design Constants : $R_{u.max}$	$R_{u.max}=2.07\ N/mm^2$
	: $P_{t.min}$	$P_{t.min}=0.205\ \%$
	II - Size of Footing :	
6.	Area of footing is designed for working load since bearing capacity of soil is for service load. Working Load $P_w = P_u/1.5 = 261/1.5$ Assuming self weight of footing to be 10 % of P_u, required bearing area for footing = $(1.1P_u/1.5)/q$ Required $A_f = (1.1x261/1.5)/150 = 1.276\ m^2$	Req.$A_f = 1.276\ m^2$
7.	(a) Required Length of Footing L_f : Assuming equal projections in two perpendicular directions, required length of footing $L_f = (D-b)/2 + \sqrt{(D-b)^2/4 + A_f} = \sqrt{A_f}$ since b = D, $\therefore L_f = \sqrt{1.276} = 1.13m = 1.13x1000 = 1130$ mm.	Req.L_f = 1130 mm.
	(b) Required Width of footing $B_f = A_f/L_f = 1.276/1.13$ $= 1.13\ m = 1130$ mm	Req.B_f = 1130 mm
	(c) Provide Length of footing in multiple of 25 mm on higher side = 1150 mm	L_f = 1150 mm
	(d) Provide Width of footing in multiple of 25 mm on higher side = 1150 mm	B_f = 1150 mm
	(e) Provided area of footing $A_f = L_f \times B_f$ $= 1.15x1.15=1.3225\ m^2$	$A_f = 1.3225\ m^2$
8.	Upward factored reaction (due to P_u) causing bending in footing, $w_u = P_u/A_f = 261/1.3225 = 197.35\ kN/m^2$	$w_u = 197.35\ kN/m^2$
9.	Projection of footing for bending @ X-axis perpendicular to L_f, $X_1 = (L_f - D)/2 = (1150 - 230)/2 = 460mm = 0.46m$	x_1 = 460 mm
10.	Projection of footing for bending @ Y-axis perpendicular to B_f. $Y_1 = (B_f - b)/2 = (1150 - 230)/2 = 460mm = 0.46m$	y_1 = 460 mm
	II - Depth of Footing for Two-way Bending :	
11.	(a) Bending Moment @ column face parallel to X - axis, $M_{ux} = w_u B_f X_1^2/2 = 197.35x1.15x0.46^2/2 = 24.01$ kN.m	M_{ux} = 24.01 kN.m
	(b) Bending Moment @ column face parallel to Y - axis, $M_{uy} = w_u L_f Y_1^2/2 = 197.35x1.15x0.46^2/2 = 24.01$ kN.m	M_{uy} = 24.01 kN.m

Illustrated Design of Footing Continued

Step No. Design Calculations **Results**

12. (a) Width of sloping footing at top, assuming 50mm level projection beyond the face of column on both sides,
$b'_f = b + 50x2 = b + 100 = 230 + 100 = 330$ mm. — $b'_f = 330$ mm

(b) Length of sloping footing at top, assuming 50mm level projection beyond the face of column on both sides,
$L'_f = D + 50x2 = D + 100 = 230 + 100 = 330$ mm. — $L'_f = 330$ mm

13. (a) Required effective depth for bending @ X - axis,

$dx = \sqrt{M_{ux}/(R_{u.max} b'_f)} = \sqrt{24.01x10^6/(2.07x330)} = 188$mm — Req.dx = 188 mm

(b) Required effective depth for bending @ Y - axis,

$dy = \sqrt{M_{uy}/(R_{u.max} L'_f)} = \sqrt{24.01x10^6/(2.07x330)} = 188$mm — Req.dy = 188 mm

14. (a) Assumed bar diameter $\phi = X_1/57$ (since required $L_d = 57\phi$)
= 460/57 say 8 mm. — ϕ = 8 mm

(b) Required cover d'_x from centre of bottom bars
= clear cover + $\phi/2$ = 50 + 8/2 = 54 mm — d'_x = 54 mm

(c) Required cover d'_y from centre of top bars
= clear cover + ϕ + $\phi/2$ = 50 + 8 + 8/2 = 62 mm — d'_y = 62 mm

(d) Required Total depth of footing for bending @ X-axis
= Req.dx + d'_x = 188 + 54 = 242 mm

(e) Required Total depth of footing for bending @ Y-axis
= Req.dy + d'_y = 188 + 62 = 250 mm. — Req.D_f = 250 mm

15. (a) Provided Total Depth D_f = 250 mm. — D_f = 250 mm

(b) Provided effective depth $d_x = D_f - d'_x = 250 - 54 = 196$ mm. — d_x = 196 mm

(c) Provided effective depth $d_y = D_f - d'_y = 250 - 62 = 188$ mm. — d_y = 188 mm

(d) Provided Total Depth at the edge, $D_{f.min}$ = 150 mm. — $D_{f.min}$ = 150 mm

(e) Provided effective depth at the edge to the centre of bottom bars = $d_{x.min} = D_{f.min} - d'_x$ = 150 - 54 = 96 mm. — $d_{x.min}$ = 96 mm

(f) Provided effective depth at the edge to the centre of top bars = $d_{y.min} = D_{f.min} - d'_y$ = 150 - 62 = 88 mm. — $d_{y.min}$ = 88 mm

III - Check for Two-way Shear:

Critical section for two-way shear is a peripheral section at a distance dy/2 from the periphery of column.

16. (a) Length of peripheral critical section at distance dy/2 from column face, $L_2 = D + 2xd_y/2 = D + d_y$ = 230+188= 418mm — L2 = 418 mm

(b) Width of peripheral critical section at distance dy/2 from column face, B2 = b+2xdy/2 = b+dy = 230+188= 418mm — B2 = 418 mm

(c) Effective depth at peripheral section
$d2 = [D_f - (D_f - D_{f.min})x(dy/2-50)/(x_1 - 50)] - d'_y$
= [250- (250- 150) x (188/2-50)/(460- 50)]-62=177.3mm — d2 = 177.3mm

(d) Area resisting shear = A2 = 2(L2 + B2)x d2
= 2(418+418)x177.3=296446 mm² — A2 = 296446 mm²

17. Shear Strength of concrete for two-way bending,
$T_{uc2} = k_s T'_{uc}$

(a) $T'_{uc} = 0.25\sqrt{f_{ck}} = 0.25\sqrt{15} = 0.968$ N/mm² — T'_{uc} = 0.968 N/mm²

Illustrated Design of Footing Continue

Step No.	Design Calculations	Results
17.	(b) $B_c = b/D = 230/230 = 1$	$B_c = 1$
	(c) $k_s = (0.5 + B_c)$ but <= 1. $= (0.5+1) = 1.5.$ $\therefore k_s = 1$	$k_s = 1$
	(d) $T_{uc2} = k_s T'_{uc} = 1.0 \times 0.968 = 0.968$ N/mm².	T_{uc2}=0.968 N/mm²
18.	Shear Resistance of concrete $V_{uc2} = T_{uc2} \times A2 = 0.968x296445/1000 = 286.96$ kN	V_{uc2} = 286.96 kN
19.	Design Shear $V_{uD2} = w_u(L_f B_f - L_2 B_2)$ $= 197.35x(1.15x1.15 - 0.418x0.418)$ $= 226.51$ kN	V_{uD2} = 226.51 kN $\therefore D_f$ = 250 mm.
20.	$V_{ud2} < V_{uc2}$ $\therefore$ O.K.	
	IV - Area of Steel including Check for Development Length:	
21.	(a) Required $A_{stx} = (0.5f_{ck}/f_y)[1- \sqrt{1-4.6M_{ux}/(f_{ck}B'_f dx^2)}]B'_f dx$	
	$A_{stx} = (0.5x15/415)x[1- \sqrt{1-4.6x24.01x10^6 /(15x330x196^2)}] \times 330x196 = 412$ mm².	A_{stx} = 412 mm²
	(b) Required $A_{sty} = (0.5f_{ck}/f_y)[1- \sqrt{1-4.6M_{uy}/(f_{ck}L'_f dy^2)}]L'_f dy$	
	$A_{sty} = (0.5x15/415)x[1- \sqrt{1-4.6x24.01x10^6 /(15x330x188^2)}] \times 330x188 = 440$ mm²	A_{sty} = 440 mm²
22.	(a) Required $A_{sty.min} = p_{t.min} \times L'_f\ dy/100 = 0.205 \times 330x188/100 = 127$ mm²	$A_{sty.min}$=127mm²
	(b) Required $A_{stx.min} = p_{t.min} \times B'_f\ dx/100 = .205 \times 330x196/100 = 133$ mm².	$A_{stx.min}$=133mm²
23.	(a) Diameter of bar ϕ = 8 mm	ϕ = 8 mm
	(b) Required $L_d = 57\phi = 57x8 = 456$ mm	Req.L_d = 456mm
	(c) Available L_d = X1 or Y1 whichever is less = 460 mm > Req. L_d. $\therefore$ O.K. Otherwise A_{st} will be required to be increased. Modified A_{st} = (Req.L_d/Av.L_d)A_{st} but >= A_{st} Alternatively, depth will be increased.	Av. L_d = 460mm
	(d) Modified $A_{stx} = A_{stx}$ in this case.	A_{stx} = 412 mm²
	(e) Modified $A_{sty} = A_{sty}$ in this case.	A_{sty} = 440 mm²
24.	(a) Area of bar $a_{st} = 0.785\phi^2 = 0.785 \times 8^2 = 50.24$ mm².	a_{st} = 50.24mm²
	(b) Requd No.of bars parallel to width of footing for bending @ Y-axis: $Ny = Req.A_{sty}/a_{st} = 440/50.24$ = say 9.	Ny = 9
	(c) Spacing of bars $Sy = (L_f - 50)/(Ny - 1) = (1150-50)/(9 - 1) = 137$ mm.	Sy = 137 mm
	(d) Provided $A_{sty} = Ny \times a_{st} = 9x50.24 = 452$ mm².	Prov.Asty=452mm²
25.	(a) Reqd No.of bars parallel to length of footing for bending @ X-axis: $Nx = Req.A_{stx}/a_{st} = 412/50.24$ = say 9. For square footing, Nx = Ny	Nx = 9
	(b) Spacing of bars $Sx = (B_f - 50)/(Nx - 1) = (1150-50)/(9 - 1) = 137$ mm	Sx = 137 mm
	(c) Provided $A_{stx} = Nx \times a_{st} = 9x50.24 = 452$ mm².	Prov.A_{stx}=452mm²

Illustrated Design of Footing Continued

Step No.	Design Calculations	Results
	V - Check for One-way Shear for Bending @ Y - axis:	
	Since effective depth dy is always less than dx in footing, shear for bending @ y - axis is normally critical as compared to shear for bending @ X - axis. Therefore, it should be checked first. Critical section for one-way shear is taken at a distance dy from face of column. Since the depth of footing varies from face of column to the free edge, the width and depth at critical section will be calculated first.	
26.	(a) Width at top of footing at critical section is given by $L_{y1} = D + 2dy = 230 + 2x188 = 606$ mm	$L_{y1} = 606$ mm
	(b) Effective depth of footing at critical section $d_{y1} = [D_f - (D_f - D_{f.min})(dy - 50)/(Y_1 - 50)] - d'y$ $= [250 - (250 - 150)x(188 - 50)/(460 - 50)] - 62$ $= 154.3$ mm	$d_{y1} = 154.3$ mm
	(c) Area of footing at critical section $A_{y1} = L_f \times d_{y.min} + (d_{y1} - d_{y.min})(L_f + L_{y1})/2$ $= 1150x88+(154.3-88)x(1150+606)/2 = 159411\ mm^2$.	$A_{y1} = 159411 mm^2$
27.	$P_{ty} = 100Asty/A_{y1} = 100x452/159411 = 0.283$ %.	$P_{ty} = 0.283$ %
28.	Shear strength of concrete in one-way shear $T_{ucy1} = 0.37\ N/mm^2$ from Table 4.2 or Eq.4.2-3 .	$T_{ucy1} = 0.37 N/mm^2$
29.	Shear Resistance of Concrete Section in one-way shear $V_{ucy1} = T_{ucy1} \times A_{y1} = 0.37 \times 159448/1000 = 59.00$ kN.	$V_{ucy1} = 59.00$ kN
30.	Design shear $V_{uDy1} = w_u L_f (Y_1 - dy)$ $= 197.35 \times 1.15(0.46-0.188) = 61.73$ kN $> V_{ucy1}$. ∴ The section is unsafe for one-way shear. Since $V_{uDy1}/V_{ucy1} < 1.05$, only steel will be increased and not the depth of footing.	$V_{uDy1} = 61.73$ kN
	Revised calculations for One-way shear @ y - axis :	
26.	L_{y1}, d_{y1} and A_{y1} remain unchanged. Rev.Ny = 9+1=10 bars. Provided $A_{sty} = 10x50.24 = 502.4\ mm^2$.	Rev N_y = 10 Rev.A_{sty}=502mm^2
27.	Revised $P_{ty} = 100x502.4/159411 = 0.315$ %.	Rev.P_{ty}=0.315%
28.	Revised $T_{ucy1} = 0.386\ N/mm^2$ from Eq.4.2-3 .	T_{ucy1}=.386N/mm^2
29.	Revised $V_{ucy1} = 0.386x159411/1000 = 61.57$ kN	V_{ucy1}=61.57 kN
30.	V_{uDy1} remains unchanged = 61.73 kN which is still greater than V_{ucy1} and, therefore, steel will be required to be increased further from 10 bars to 11 bars. On second revision, Ny = 11, $A_{sty} = 552\ mm^2$, P_{ty}=.346%, $T_{ucy1} = 0.401\ N/mm^2$, and $V_{ucy1} = 63.92$ kN $> V_{uDy1} = 61.73$ kN. ∴ O.K.	V_{uDy1}=61.73 kN N_y = 11 A_{sty} = 552 mm^2 P_{ty} = 0.346% T_{ucy1}=.401N/mm^2 V_{ucy1}=63.92 kN.

Illustrated Design of Footing Continued

Step No.	Design Calculations.	Results

VI - Check for One-way Shear for Bending @ X - axis:

Critical section for one-way shear happens to be at a distance dx from face of column. Because of varying depth of footing, the width at top and effective depth at critical section will be calculated first.

31. (a) Width of footing at top at critical section
$B_{x1} = b + 2dx = 230 + 2x196 = 622$ mm. — $B_{x1} = 622$ mm

(b) Effective depth of footing at critical section
$d_{x1} = D_f - (D_f - D_{f,min})(dx-50)/(x_1 - 50) - d'_x$
$= 250- (250- 150)x(196 -50)/(460- 50) - 54=160.4$ mm — $d_{x1} = 160.4$ mm

(c) Area of footing at critical secion
$A_{x1} = B_f \times dx.minx + (d_{x1}-dx.minx)x(B_f+B_{x1})/2$
$= 1150x96 + (160.4 - 96)x(1150 + 622)/2 = 167458\ mm^2$ — $A_{x1} = 167458\ mm^2$

32. (a) For square footing, Nx = Ny for practical reasons. — N_x = 11
∴ Though 9 bars are required theoretically, 11 bars will be provided. $A_{stx1} = 552\ mm^2$. — $A_{stx1} = 552\ mm^2$
$p_{tx} = 100x552/167458 = 0.33$ % — $p_{tx} = 0.33$ %

33. Shear strength of concrete for one-way shear
$T_{ucx1} = 0.394\ N/mm^2$ using Eq. 4.2-3 . — $T_{ucx1} = .394 N/mm^2$

34. Shear resistance of concrete for one-way bending
$V_{ucx1} = 0.394x167458/1000 = 65.98$ kN. — $V_{ucx1} = 65.98$ kN

35. Design shear $V_{uDx} = w_u\ B_f\ (X_1 - dx)$
$= 197.35x1.15x(0.46 - 0.196) = 59.92$ kN — $V_{uDx} = 59.92$ kN
$< V_{ucx1}$. ∴ O.K.

6.5.5 Design of Footing in Tabular Form

Step No. — Footings for Column No.s

I - Data

Step No.			C8	C13	C7	C14	C5	C6	C2	C1
1.	Column Mark		C8	C13	C7	C14	C5	C6	C2	C1
2.	q	kN/m^2	150	150	150	150	150	150	150	150
3.	P_u	kN	342	295	261	192	157	128	101	45
4.(a)	b	mm	230	230	230	230	230	230	230	230
(b)	D	mm	230	230	230	230	230	230	230	230
5.(a)	f_{ck}	N/mm^2	15	15	15	15	15	15	15	15
(b)	f_y	N/mm^2	415	415	415	415	415	415	415	415
(c)	$R_{u.max}$	N/mm^2	2.07	2.07	2.07	2.07	2.07	2.07	2.07	2.07
(d)	$p_{t.min}$	%	0.205	0.205	0.205	0.205	0.205	0.205	0.205	0.205

II - Size of Footing for Bearing

Step No.			C8	C13	C7	C14	C5	C6	C2	C1
6.	Req.A_f	m^2	1.672	1,442	1.276	0.939	0.768	0.626	0.494	0.220
7.(a)	Req.L_f	mm	1294	1201	1130	969	877	792	703	470
(b)	Req.B_f	mm	1294	1201	1130	969	877	792	703	470
(c)	Pro.L_f	mm	1300	1225	1150	975	900	800	725	475
(d)	Pro.B_f	mm	1300	1225	1150	975	900	800	725	475
(e)	Pro.A_f	m^2	1.69	1.50	1.32	.951	.810	.640	.526	.226

Design of Footings Continued

No.	Column Mark	Unit	C8	C13	C7	C14	C5	C6	C2	C1
8.	w_u	kN/m²	202.37	196.58	197.35	201.97	193.83	200.00	192.15	199.45
9.	X_1	mm	535.0	497.5	460.0	372.5	335.0	285.0	247.5	122.5
10.	Y_1	mm	535.0	497.5	460.0	372.5	335.0	285.0	247.5	122.5
	III - Depth of Footing for Two-way Bending									
11.(a)	M_{ux}	kN.m	37.65	29.80	24.01	13.66	9.79	6.50	4.27	0.71
(b)	M_{uy}	kN.m	37.65	29.80	24.01	13.66	9.79	6.50	4.27	0.71
12.(a)	b'_f	mm	330	330	330	330	330	330	330	330
(b)	L'_f	mm	330	330	330	330	330	330	330	330
13.(a)	Req.dx	mm	235	209	188	142	120	98	79	33
(b)	Req.dy	mm	235	209	188	142	120	98	79	33
14.(a)	ø	mm	8	8	8	8	8	8	8	8
(b)	d'_x	mm	54	54	54	54	54	54	54	54
(c)	d'_y	mm	62	62	62	62	62	62	62	62
(d)	Req.D_f	mm	297	271	250	204	182	160	141	95
15.(a)	Prov.D_f	mm	300	275	250	225	200	175	150	150
(b)	Prov.d_x	mm	246	221	196	171	146	121	96	96
(c)	Prov.dy	mm	238	213	188	163	138	113	88	88
(d)	Prov.$D_{f.min}$	mm	150	150	150	150	150	150	150	150
(e)	Prov.$d_{f.minx}$	mm	96	96	96	96	96	96	96	96
(f)	Prov.$d_{f.miny}$	mm	88	88	88	88	88	88	88	88
	IV - Check for Two-way Shear									
16.(a)	L2	mm	468	443	418	393	368	343	318	318
(b)	B2	mm	468	443	418	393	368	343	318	318
(c)	d2	mm	216.6	197.2	177.3	155.7	134.7	112.3	88.0	88.0
(d)	A2	mm²	405587	349470	296393	244720	198229	154087	111936	111936
17.(a)	T'_{uc}	N/mm²	0.968	0.968	0.968	0.968	0.968	0.968	0.968	0.968
(b)	B_s		1	1	1	1	1	1	1	1
(c)	ks		1	1	1	1	1	1	1	1
(d)	T_{uc2}	N/mm²	0.968	0.968	0.968	0.968	0.968	0.968	0.968	0.968
18.	V_{uc2}	kN	392.70	338.37	286.98	236.95	191.93	149.19	108.38	108.38
19.	V_{uD2}	kN	297.78	256.52	160.81	130.75	104.47	81.57	24.83	
20.	D_{fs}	mm	300	275	250	225	200	175	150	150
	V - Area of Steel for Bending including Check for Development Length									
21.(a)	Req.A_{stx}	mm²	514	451	412	253	212	169	141	21
(b)	Req.A_{sty}	mm²	542	477	441	270	228	185	159	23
22.(a)	Req.$A_{stx.min}$	mm²	166	150	133	116	99	82	65	65
(b)	Req.$A_{sty.min}$	mm²	161	144	127	110	94	77	60	60

Design of Footings : Area of Steel for Bending – continued

		Column Mark	C8	C13	C7	C14	C5	C6	C2	C1
23.(a)	Max. Ø	mm	8	8	8	8	8	8	8	8
(b)	Req.L_d	mm	456	456	456	456	456	456	456	456
(c)	Ava.L_d	mm	535	497.5	460	372.5	335	285	247.5	122.5
(d)	Mod.A_{stx}	mm^2	514	451	412	309	287	270	259	78
(e)	Mod.A_{sty}	mm^2	542	477	441	330	311	296	292	86
24.(a)	a_{st}	mm^2	50.24	50.24	50.24	50.24	50.24	50.24	50.24	50.24
(b)	Nx		11	9	9	7	6	6	6	2 4
(c)	Sx	mm	125	146	137	154	170	150	135	425 141
(d)	Prov.A_{stx}	mm^2	552	452	452	351	301	301	301	100 201
25.(a)	Ny		11	10	9	7	7	6	6	2 4
(b)	Sy	mm	125	130	137	154	141	150	135	425 147
(c)	Prov.A_{sty}	mm^2	552	502	452	351	351	301	301	100 201

VI – Check for One-way Shear for Bending @ Y – axis:

			C8	C13	C7	C14	C5	C6	C2	C1
26.(a)	L1	mm	706	656	606	556	506	456	406	406
(b)	d_{y1}	mm	179.9	167.5	154.3	136.7	122.6	106.3	88	88
(c)	A_{y1}	mm^2	206531	182541	159448	123096	103497	81891	63800	41800
27.(a)	Prov.Ny		11	10	9	7	7	6	6	4
(b)	Prov.A_{sty}	mm^2	552	502	452	351	351	301	301	200
(c)	P_{ty}	%	.267	.275	.284	.286	.340	.368	.472	.481
28.	T_{ucy1}	N/mm^2	.361	.365	.370	.371	.398	.411	.454	.457
29.	V_{ucy1}	kN	74.62	66.72	59.00	45.69	41.22	33.68	28.95	19.09
30.	V_{uDy1}	kN	78.13	68.51	61.73	41.26	34.37	27.52	22.22	3.27
	Remarks		Unsafe	Unsafe	Unsafe	Safe	Safe	Safe	Safe	Safe

Steps for revised calculations for columns C8, C13, and C7

Col	Ny	A_{sty} mm^2	P_{ty} %	T_{ucy1} N/mm^2	V_{ucy1} kN	V_{uDy1} kN	Remarks:	Ny	A_{sty} mm^2	P_{ty} %	T_{ucy1} N/mm^2	V_{ucy1} kN	V_{uDy1} kN	Remarks
C8	12	602	.291	.374	77.34	78.13	Unsafe	13	652	.316	.387	79.91	78.13	Safe
C13	11	552	.303	.380	69.38	68.51	Safe	–	–	–	–	–	–	–
C7	10	502	.315	.386	61.60	61.73	Unsafe	11	552	.346	.401	64.02	61.73	Safe

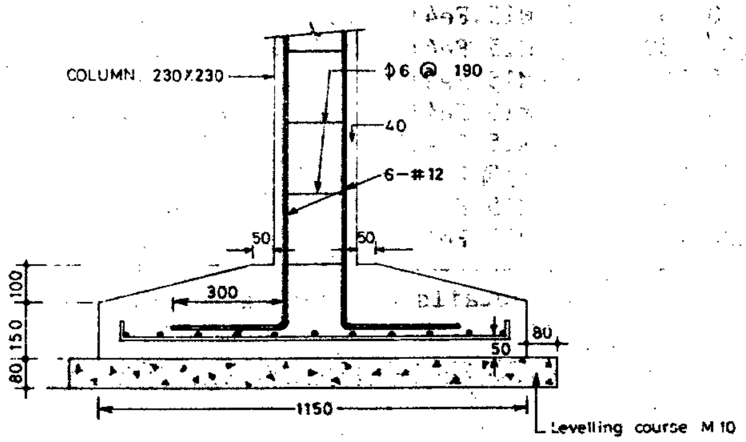
VII – Check for One-way Shear for Bending @ X – axis:

			C8	C13	C7	C14	C5	C6	C2	C1
31.(a)	B1	mm	722	672	622	572	522	472	422	422
(b)	d_{x1}	mm	185.4	173.2	160.4	142.9	129.2	113.4	96	96
(c)	A_{x1}	mm^2	215165	190857	167450	129847	109175	87896	69600	45600
32.(a)	Prov.Nx		13	11	11	7	7	6	6	4
(b)	Prov.A_{stx}	mm^2	653	552	552	351	351	301	301	200
(c)	P_{tx}	%	.304	.290	.330	.271	.320	.343	.433	.441
33.	T_{ucx1}	N/mm^2	.380	.373	.394	.363	.389	.400	.439	.442
34.	V_{ucx1}	kN	81.87	71.23	65.92	47.15	42.74	35.14	30.52	20.14
35.	V_{udx1}	kN	76.03	66.59	59.91	39.68	32.97	26.24	21.11	2.51
36.	Remarks		Safe	Safe	Safe	Safe	Safe	Safe	Safe	Safe

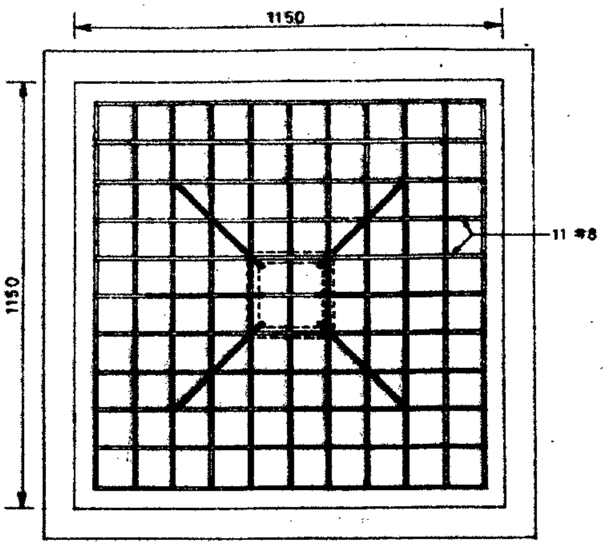
SCHEDULE OF FOOTING

Col. Mark	P_u kN	b mm	D mm	q kN/m²	Grades of Con Steel	L_f mm	B_f mm	D_f mm	$D_{f.min}$ mm	Nx	ϕ mm	Sx mm	Ny	ϕ mm	Sy mm
C8	342	230	230	150	M15 Fe415	1300	1300	300	150	13-#8		104	13-#8		104
C13	295	230	230	150	M15 Fe415	1225	1225	275	150	11-#8		117	11-#8		117
C7	261	230	230	150	M15 Fe415	1150	1150	250	150	11-#8		110	11-#8		110
C14	192	230	230	150	M15 Fe415	975	975	225	150	7-#8		154	7-#8		154
C5	157	230	230	150	M15 Fe415	900	900	200	150	7-#8		141	7-#8		141
C6	128	230	230	150	M15 Fe415	800	800	175	150	6-#8		150	6-#8		150
C2	101	230	230	150	M15 Fe415	725	725	150	150	6-#8		135	6-#8		135
C1	45	230	230	150	M15 Fe415	475	475	150	150	4-#8		141	4-#8		141

For details of footing See Fig. 6.5



SECTION



PLAN

FIG. 6.5 COLUMN FOOTING (Column C7)

Project – II : Design of Multi-storeyed Office Building

7.1 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 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 structure 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. As seen in Chapter-3, that 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 as well as for working on a Personal Computer.

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. Analyses of top storey frame and bottom storey frame have been done using the substitute frame package prepared by the Authors for Personal Computer. Design of members of only one frame that is analysed has been presented. The purpose of this project is not to illustrate the design of entire building. This has been done in Project-3.

7.2 SALIENT FEATURES

A floor plan of a typical public-cum office building is shown in Fig.7.1. 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. Thus, entire building space frame can be divided into a number of vertical plane frames.

2) According to the requirements of the architect, there has to be only one line of internal columns along one side of the corridor. The transverse beams are, therefore, 2 span continuous beams with unequal spans.

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 beams are not only subjected to uniformly distributed load due to floor load but also a point load from the supported longitudinal beam.

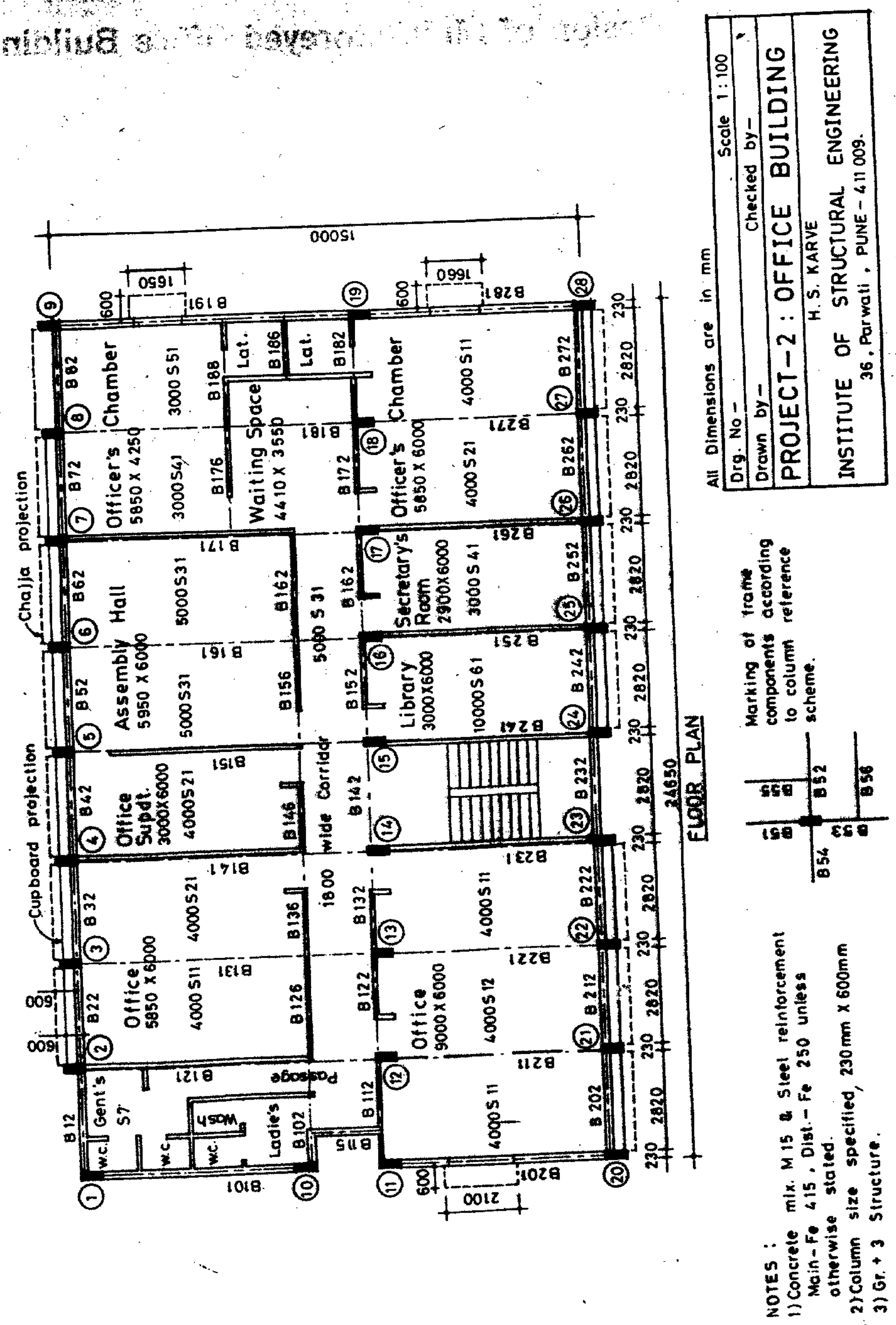


Fig. 7.1 Floor Plan Of Typical Public-cum Office Building

4) According to another requirement of the architect, all the columns are required to be of equal size 230 mm x 600 mm right from bottom to top.

5) The various rooms are used for different functions. Therefore, the live loads for all rooms are not same. (For example, Office Chamber -3 kN/m², Office-4 kN/m², Stairs - Passages and Assembly Hall- 5 kN/m², Library -10 kN/m² etc). Therefore, though practically all frames are geometrically simillar, they are subjected to diffetent 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 library floor load from one side	Internal wall with office floor load on one side and assembly hall load on 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 locatio
28-19-9	External wall	External wall.

This requires analysis to be done seperately, practially, for all frames.

6) 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 0.75DL. If 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.

7.3 DATA

1.	Type of Structure	: Multistorey rigid jointed frame.
2.	Layout	: As shown in Fig. 7.1.
3.	Number of Storeys	: Four. (Ground + 3).
4.	Floor to floor height	: 3.35 metres.
5.	Height of plinth	: 0.5 metre. above G.L.
6.	Depth of foundation	: 1.5 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.	Live Load	: As per Table 2.2.
10.	Materials	: Concrete M15,Steel:Main Fe415,Secondary Fe250.
11.	Design Philosophy	: Limit State Method conforming to IS:456-1978.
12.	Design Assumptions	: All members of the main frames are rigid jointed, Simple connections in case of other components.

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* 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.

7.4 LOADS

(a) Dead Load :

i) Finishes.

Terrace waterproofing	2.5 kN/m^2
Floor finish	1.0 kN/m^2
Sanitary blocks including filling	2.5 kN/m^2

ii) Slab : 25D kN/m^2 where D is depth of slab in metres.

iii) Walls : External 250 mm thick : 20X.25 = 5 kN/m/metre height.
Internal 150 mm thick : 20X.15 = 3 kN/m/metre height.

(b) Live Load.

Roof	1.5 kN/m^2 ,	Office floors	4 kN/m^2,
Library	10 kN/m^2 ,	Officers Chamber	3 kN/m^2,
Assembly Hall	5 kN/m^2 ,	Stairs, Corridors	5 kN/m^2,
Sanitary Blocks	Public 3 kN/m^2 ,	Private	2 kN/m^2.

7.5 DESIGN OF MEMBERS

The design of frame consists of the following :
(a) Design of Slabs
(b) Analysis of Frame,
(c) Design of Beams B22-B13-B3,
(d) Design of Columns C22,C13,C3,
(e) Design of Footings for columns C22,C13,C3.
Each one is pressented in seperate section below.

7.6 DESIGN OF SLABS :

Slab Marks	Roof Slabs S1	S2	Floor Slabs S3	S4	Reference
1. Type	One-way Continuous		One-way Continuous		
2. End Condition No.	2	3	2	3	
Design moment Coefficient α =	1/10	1/12	1/10	1/12	
3. Span L metres	3.05	3.05	3.05	3.05	
4. Imposed Load : FF kN/m^2	2.50	2.50	1.00	1.00	
LL kN/m^2	1.50	1.50	4.00	4.00	
Total w_i kN/m^2	4.00	4.00	5.00	5.00	
5. Depth :					
For serviceability, D mm	110	110	110	110	App.C-1
p_t=.4%, α_1=1.28, r_b =26, r_a =33.28					
6. Loads :					
Self weight = 25D/1000 kN/m^2	2.75	2.75	2.75	2.75	
Dead Load q_d = Self + FF kN/m^2	5.25	5.25	3.75	3.75	
Live Load q_l = LL kN/m^2	1.50	1.50	4.00	4.00	
Total Load q = $q_d + q_l$ kN/m^2	6.75	6.75	7.75	7.75	
Ultimate Load q_u = 1.5 q kN/m^2	10.125	10.125	11.625	11.625	
7. Design Moment: $M_{u.max} = \alpha q_u L^2$ kN.m	9.42	7.85	10.81	9.01	
8. Main Steel: - Fe415	321.89	262.96	376.39	306.25	Eq. 4.1-5
Diameter and spacing mm	#8-150	#8-190	#8-130	#8-160	Table F-3

Slab Marks		Roof Slabs S1	S2	Floor Slabs S3	S4	Reference
9. Distribution Steel – Fe250 Diameter and Spacing	mm	ϕ6-160	ϕ6-160	ϕ6-160	ϕ6-160	Table C-3
10. End Shears :						
Long Edge $q_uL/2$	kN/m	15.44	15.44	17.73	17.73	
Short Edge $q_uL/6$	kN/m	5.15	5.15	5.91	5.91	
11. End Shear due to factored live load on long edge = $1.5q_l\ L/2$	kN/m	3.43	3.43	9.15	9.15	

7.7 ANALYSIS OF FRAME : See Fig.7.2.

7.7.1 Member Data :

(a) Beams :		4-5	5-6	Explanatory Note No.
1) Size : bxD	mm	230x600	230x600	(1)
2) Length L	mm	6000	8400	
3) Moment of Inertia	mm^4			
i) Rectangular section $I=bD^3/12$		4140×10^6	4140×10^6	
ii) Multiplying factor for flanged section		2	2	(2)
iii) I of flanged Section	mm^4	8280×10^6	8280×10^6	
iv) Stiffness $k_b = I/L$	mm^3	1.38×10^6	$.986\times10^6$	

(b) Columns :		Upper (4-1,5-2,6-3)	Lower (4-7 5-8,6-9)	
1) Size. bxD	mm	230x600	230x600	
2) Length L	mm	3350	3350	(3)
3) Moment of Inertia $I=bD^3/12$	mm^4	4140×10^6	4140×10^6	
4) Stiffness $K_c = I/L$	mm^3	1.236×10^6	1.236×10^6	

Explanatory Notes :

(1) Eventhough span lengths of two beams are different, the depth has been taken same for both the spans from practical considerations.

(2) Since, both the beams are flanged, the moment of inertia of flanged section is taken approximately equal to 2 times that of rectangular beam. See Sect.3.2.5.

(3) Length of column for the purpose of calculation of stiffness is to be taken equal to floor to floor height because it may not be possible to determine centre to centre distance between column joints at top and at

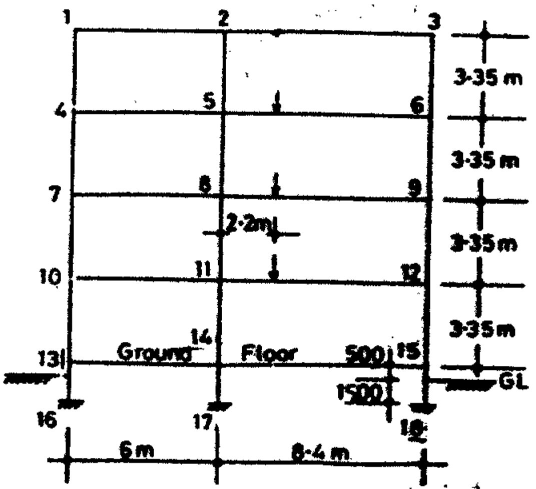


Fig.7.2 Cross-section of plane frame

7.7.2 Load Data :

(A) Roof Level :

Slab : 110 mm thick : Dead Load : q_d = Self Wt. + FF
= 25x.11+2.5 = 5.25 kN/m²
Live Load : q_l = 1.50 kN/m²
Total Working Load : q = 5.25 + 1.50= 6.75 kN/m²
Total Ultimate Load : q_u = 1.5x6.75 =10.125kN/m²
Parapet Wall:250mm thick x 1m high : w_w =20x.25x1= 5.00 kN/m

(1) Longitudinal Beams :

(a)External : Assuming Section 230 mm x 300 mm
DL : Self : 25x.23x(.3 - .11) = 1.10 kN/m
Parapet Wall : = 5.00 kN/m
Slab : $q_dL_x/6$=5.25x3.05/6= 2.67 kN/m*
Total wd = 8.77 kN/m
LL : Slab : $q_lL_x/6$=1.50x3.05/6= 0.76 kN/m*
Total Working Load w = (DL+LL)= 9.53 kN/m
Maximum Ultimate Load for beam design :
w_{ub} =1.5x9.53=14.30 kN/m

* This is the triangular load transferred to longitudinal beam as explained in Sect.3.4.1. It is taken for beam design only and not for column design since it is already fully taken along the span on to the transverse beams which transfers the same to the column. It is, therefore, excluded while computing the load on column.

Maximum 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 = 9.15x3.05/2 = 13.96 kN(from each side)

(b)Internal: Assuming Section 230mm x 300mm
DL : Self (Same as external) = 1.10 kN/m
Slab $2xq_d$ x $L_x/6$ = 2x2.67= 5.34 kN/m
Total dead load w_d = 6.44 kN/m
LL : Slab $2xq_l$ x $L_x/6$ = 2x0.76= 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)=1.5x7.96 =11.94 kN/m
Maximum ulitmate load for column design excluding triangular slab load
= 1.5xself = 1.5x1.1 = 1.65 kN/m only.
Beam End Shear as column load = 1.65x3.05/2 = 2.52 kN(from each side)

(2) Transverse Beam : Section 230mm x600 mm
DL : Self : 25x.23x(0.6-.11) = 2.82 kN/m
Slab : 5.25x3.05 =16.01 kN/m
Total dead load w_d =18.83 kN/m
LL : Slab : 1.50x3.05 w_l = 4.58 kN/m
Total working load w = w_d+w_l =23.41 kN/m
Maximum load w_{max} = 1.5(DL+LL)
= 1.5x23.41 =35.11 kN/m say 35 kN/m
Minimum load w_{min} = 0.9DL
= 0.9x18.83 =16.95 kN/m say 17 kN/m.
Ratio LL/Total Load= 4.58/23.41 = 0.196

(B) Floor Level :

Slab : Dead Load : Self weight + floor finish
q_d = 25x.11 + 1 = 3.75 kN/m²
Live Load : q_l = = 4.00 kN/m²
Total working load q =q_d + q_l= 7.75 kN/m²
Total ultimate load q_u= 1.5xq =11.625kN/m²

Wall : Internal : 150 mm thick(including plaster), 2.2m high
w_{w1} = 20x.15x2.2 = 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} = 20x.25x2.9 = 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 230mm x 300mm
Dead Load : Self weight=25x.23x(.3-.11)= 1.10 kN/m
Wall : W_{w1} = 6.60 kN/m
Slab* : 2X($q_d L_x$/6)=2X1.91= 3.82 kN/m
Total = 11.52 kN/m
Live Load : : 2X($q_l L_x$/6=2X2.03)= 4.06 kN/m
Total working load w = 15.58 say 15.60 kN/m
Total ultimate load w_u = 1.5x15.58 = 23.37 kN/m.

(b) External : Section 230mm x 450mm
Dead Load : Self wt = 25X.23X(.45-.11) = 1.96 kN/m
Wall = W_{w2} = 14.50 kN/m
Slab* =$q_d L_x$/6=3.75x3.05/6= 1.91 kN/m
= 18.37 kN/m.
Live Load : =$q_l L_x$/6=4.00x3.05/6= 2.03 kN/m
Total working load w = 20.40 kN/m
Total ultimate load w_u = 1.5x20.4 = 30.6 kN/m.

(2) Main Transverse Beam : B221 in Fig.7.1 (or 4-5 in Fig.7.2)
Size - 230 mm X 600 mm

(a) Uniformily Distributed Load:
Dead load :Self weight :25X.23X(.6-.11)= 2.82 kN/m
Slab : 3.75x3.05 = 11.44 kN/m
Total dead load = 14.26 kN/m
Live load :Floor :4x3.05 = 12.20 kN/m.
Total Working Load w= DL+LL=14.26+12.20= 26.46 kN/m
Maximum Load : w_{max} = w_1 = 1.5(DL+LL)
= 1.5(14.26+12.2) = 39.69 kN/m say *40* kN/m.
Minimum Load : wmin = 0.9 DL=0.9x14.26 = 12.83 kN/m say *13* kN/m.
Ratio LL/Total load = 12.20/26.46 = 0.46

(b) Point Load : Nil

(3) Main Transverse Beam : B131 in Fig.7.1 (or Beam 5-6 in Fig 7.2)

(a) Uniformly Distributed Load : Same as that for Beam 4-5.
w_{max} = w_1 = = *40* kN/m
w_{min} = w_2 = = *13* kN/m.

(b) Point Load: Shear from Internal Longitudinal Beams B126 and B136
Dead Load = 11.52X3.05 = 35.14 kN
Live load = 4.06X3.05 = 12.38 kN
Maximum Load Pmax = P1 = 1.5(DL+LL)
= 1.5(35.14+12.38)= 71.30 kN say *72* kN
Minimum Load Pmin = P2 = 0.9DL
= 0.9x35.14 = 31.63 kN say *32* kN.

Foot Note-1:Though the slabs S1-S2 are spanning one way across transverse main beams(frames), part of slab load over a Triangular area with central ordinate (1/2)(qL_x/2) is transferred to longitudinal beam as explained in Sect.3.4.1. The eqvivalent uniformly distributed load is qL_x/6=.9x35.14= 31.6 kN say 32kN.

(C) Plinth Level :

(1) Longitudinal Beams :

External : Section 230mm x 450mm.
Self : 25x.23x.45 = 2.60 kN/m
Wall : 250mm thick, 3.85m high
w_w=20x.25x(3.85-0.45)= 17.0 kN/m
Total working load , w = 19.60 kN/m
Total Ultimate Load w_u = 1.5w = 1.5x19.6 = 29.40 kN

Internal : Section 230mm x 300mm.
Self : 25x.23x(.3) = 1.73 kN/m
Wall : 150mm thick,(2.2+.5)= 2.7m high
w_w = 20x.15x2.7 = 8.10 kN/m
Total working load w = 9.83 kN/m
Total ultimate load w_u =14.75 kN/m

(2) Transverse Beams : No transverse beams are provided at plinth level.

7.7.3 Fixed End Moments :

(a) Span 4-5 : L = 6 metres
Maximum : $M_{F45} = w_1L^2/12 = 40X6^2/12 = 120$ kN.m $= M_{F54}$
Minimum : $M_{F45} = w_2L^2/12 = 13x6^2/12 = 39$ kN.m $= M_{F54}$

(b) Span 5-6 : L = 8.4 metres., Point Load at 2.2 metres from support 5.
Maximum : $M_{F56} = 40X8.4^2/12 + 72X2.2X6.2^2/8.4^2 = 321.49$ kN.m
$M_{F65} = 40X8.4^2/12 + 72X2.2^2X6.2/8.4^2 = 265.82$ kN.m
Minimum : $M_{F56} = 13X8.4^2/12 + 32X2.2X6.2^2/8.4^2 = 114.79$ kN.m
$M_{F65} = 13X8.4^2/12 + 32X2.2^2X6.2/8.4^2 = 90.05$ kN.m

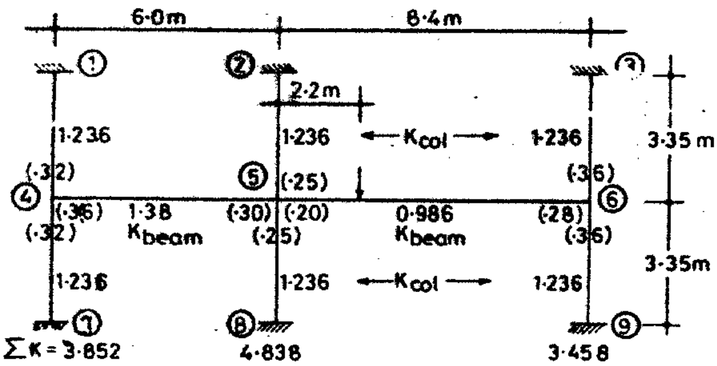
7.7.4 Design Moments and Shears by Analysis of Substitute Frame - I (Floor Frame)

The substitute floor frame has been analysed by hand computation using Moment Distribution Method. The process of distribution has been done only for 3 cycles working upto two places beyond decimal.This much accuracy is considered adequate for analysis of a R.C.building frame.

Distribution factors:(Fig.7.3)

Joint	Columns Upper	Columns lower	Beams Left	Beams Right
4 Kx10^6	1.236	1.236	-	1.38
Σ Kx10^6		3.852		
$d_i=K_i/\Sigma K$	0.32	0.32	-	0.36
for 2 columns		0.64		
5 Kx10^6	1.236	1.236	1.38	0.986
Σ Kx10^6		4.838		
$d_i=K_i/\Sigma K$	0.25	0.25	0.30	0.20
for 2 columns		0.50		
6 Kx10^6	1.236	1.236	0.986	-
Σ Kx10^6		3.458		
$d_i=K_i/\Sigma K$	0.36	0.36	0.28	-
for 2 columns		0.72		

Substitute Frame-I : Floor Frame



Distribution Factors at joints are given in brackets

Fig.7.3 Substitute Frame - I : Floor Frame

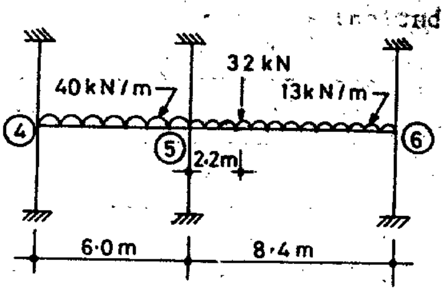
Since live load (4 kN/m²) > 0.75 times Dead load (.75x 3.75 kN/m²) , all loading arrangements will have to be considered to obtain the design moments and shears.

(a) Loading Case - I :

Maximum Midspan moment in span 4-5, and Maximum Support moment at 4

For this, there shall be maximum load on span 4-5, and minimum load on span 5-6 as shown in Fig. 7.3(a).

Substitute Frame-I



Loading Case-I

Fig. 7.3(a)
Loading Arrangement for Maximum Moments

Moment Distribution :

Joint	4		5			6	
	Cols.	4-5	5-4	Cols.	5-6	6-5	Cols.
Distri. factors	0.64	0.36	0.30	0.50	0.20	0.28	0.72
Initial F.E.M.		-120.00	120.00		-114.79	90.05	
Distri. moments	76.80	43.20	- 1.56	-2.60	- 1.05	-25.21	-64.84
Carry over mom.		- 0.78	21.60		- 12.60	- 0.52	
Distri.moments	0.50	0.28	- 2.70	-4.50	- 1.80	0.15	0.37
Carry over mom.		- 1.35	0.14		0.07	- 0.90	
Distri.moments	0.86	0.49	- 0.06	-0.10	- 0.05	0.25	0.65
Final moments	78.16	- 78.16	137.42	-7.20	-130.22	63.82	-63.82
Moment in each column	39.08			-3.60			-31.91

Shears : V_{45} = 40x6/2 - (137.42 - 78.16)/6.0 = 110.12 kN
V_{54} = 40x6 - 110.12 = 129.88 kN
V_{56} = 13x8.4/2+32x6.2/8.4+(130.22-63.82)/8.4 = 86.12 kN
V_{65} = 13x8.4 + 32 - 86.12 = 55.08 kN.

Maximum Span Moments :

Span 4-5 : x_{max} = 110.12/40
= 2.753 m from end 4.
M_{max1} = 110.12x2.753/2-78.16
= 73.42 kN.m

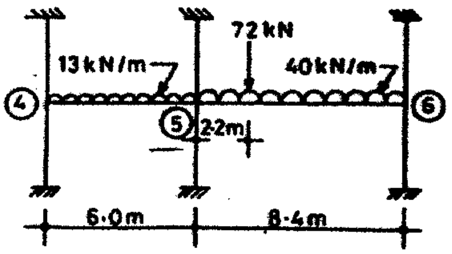
Span 5-6 : x_{max} = 55.08/13
= 4.237 m from end 6.
M_{max2} = 55.08x4.237/2-63.82
= 52.86 kN.m.

(b) Loading Case -II :

Maximum Midspan Moment in Span 5-6 and Maximum support Moment at 6

For this, there shall be maximum load on span 5-6, and minimum load on span 4-5 as shown in Fig. 7.3(b).

Substitute Frame-I



Loading Case-II

Fig. 7.3(b)
Loading Arragement for Maximum Moments

Moment Distribution:

Joint	4		5			6	
	Cols	4-5	5-4	Cols.	5-6	6-5	Cols.
Distri.Factors	0.64	0.36	0.30	0.50	0.20	0.28	0.72
Initial F.E.M.		-39.00	39.00		-321.49	265.82	
D.M.	24.96	14.04	84.74	141.25	56.50	-74.43	-191.39
C.O.M.		42.37	7.02		-37.21	28.25	
D.M.	-27.12	-15.25	9.05	15.10	6.04	-7.91	-20.34
C.O.M.		4.52	-7.62		-3.95	3.02	
D.M.	-2.89	1.63	3.47	5.79	2.31	0.85	-2.17
Final Moments	-5.05	5.05	135.66	162.14	-297.80	213.90	-213.90
Moment in each column	-2.52			81.07			-106.95

Shears : V_{45} = 13x6/2 − (135.66 + 5.05)/6 = 15.55 kN

V_{54} = 13x6 − 15.55 = 62.45 kN

V_{56} = 40x8.4/2+72x6.2/8.4+(297.80−213.90)/8.4=231.13 kN

V_{65} = 40x8.4 +72 − 231.13 =176.87 kN.

Max span moments:

Span 4-5 : x_{max} = 15.55/13

= 1.196 m from end 4.

M_{max1} = 15.55x1.196/2+5.05

= 14.35 kN.m

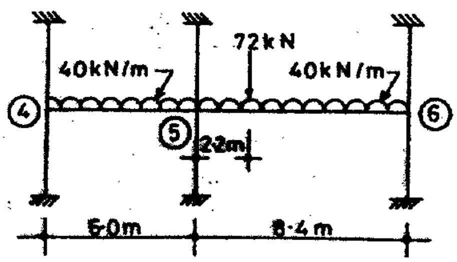
Span 5-6 : x_{max} = 176.87/40

= 4.422 m from end 6.

M_{max2} = 176.87x4.422/2−213.90

= 177.16 kN.m.

Substitute Frame – I



Loading Case – III

Fig. 7.3(c)

Loading Arrangement for Maximum Moments

(C) Loading – III :

Maximum Support Moment at 5

For this, both spans shall carry maximum load. See Fig.7.3(c)

Moment Distribution :

Joint	4		5			6	
Distribution factors:	Cols.	4-5	5-4	Cols.	5-6	6-5	Cols.
Initial F.E.M.		-120.00	120.00		-321.49	265.82	
Distributed moments	76.80	43.20	60.45	100.74	40.30	-74.43	-191.39
Carry over moments		30.22	21.60		-37.21	20.15	
Distributed moments	-19.34	-10.88	4.68	7.81	3.12	-5.64	-14.51
Carry over moments		2.34	-5.44		-2.82	1.56	
Distributed moments	-1.50	-0.84	2.48	4.13	1.65	-0.44	-1.12
Final moments	55.96	-55.96	203.77	112.68	-316.45	207.02	-207.02
Moments in each column	27.98			56.34			103.51

Shears : V_{45} = 40X6/2-(203.77-55.96)/6 = 95.36 kN
V_{54} = 40X6 - 95.36 =144.64 kN
V_{56} = 40X8.4/2+72X6.2/8.4+(316.45-207.02)/8.4=234.17 kN
V_{65} = 40X8.4 +72 - 234.17 =173.83 kN.

Maximum Span Moments : Span 4-5 : x_{max} = 95.36/40 = 2.384 m from end 4.
M_{max1} = 95.36X2.384/2- 55.96= 57.71 kN.m
Span 5-6 : x_{max} = 173.83/40 = 4.346 m from end 6.
M_{max2} = 173.83X4.346/2-207.02=170.71 kN.m.

Results :

Beam and Column Bending Moments in kN.m

Case: No.	Jt. 4 : Cols.	4 : Beam	Max.span moment	5 Beam	5 : Cols.	5 : Beam	Max.span moment	6 Beam	6 : Cols.
I	39.08	78.16	73.42	137.42	3.60	130.22	52.86	63.82	31.91
II	2.52	5.05	14.35	135.66	81.07	297.80	177.16	213.90	106.95
III	27.98	55.96	57.71	203.77	56.34	316.45	170.71	207.02	103.51
Max.	39.08	78.16	73.42	203.77	81.07	316.45	177.16	213.90	106.95

Shears in kN

Case No.	Jt. 4	5		6
I	110.12	129.88	86.12	55.08
II	15.55	62.45	231.13	176.87
III	95.36	144.64	234.17	173.83
Maximum shear	110.12	144.64	234.17	176.87
Maximum column Load	110.12	378.81		176.87

7.7.5 Design Moments and Shears by Substitute Frame -II (Bay Frame)

In this method, separate bay frames will be considered for span 4-5 & 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.

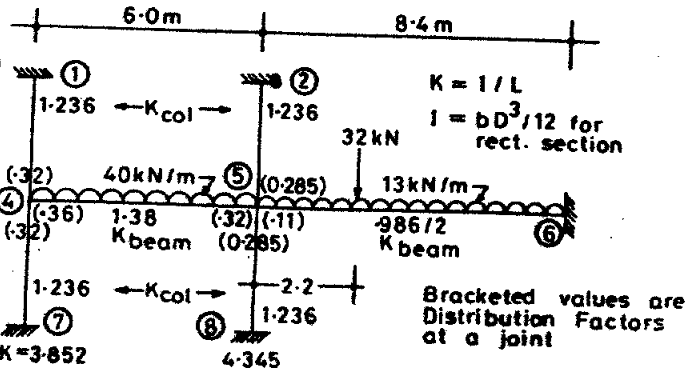


Fig. 7.4(a) Substitute Frame - II (Bay Frame) Loading Case 1

(a) Loading Case - I : Maximum Midspan Moment in Span 4-5, and Maximum Support Moment at 4.

The substitute frame will be as shown in Fig.7.4(a). 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(section 7.7.4a). Since effect of columns at 6 is ignored here, the stiffness of beam 5-6 is reduced to half. The distribution factors are calculated as under.

Jt.		Column Upper	Column Lower	Beam Left	Beam Right	ΣK
4	Stiffness $KX10^{-6}$	1.236	1.236	-	1.38	3.852
	Distri. factors $K/\Sigma K$	0.32	0.32	-	0.36	
5	Stiffness $KX10^{-6}$	1.236	1.236	1.38	0.986/2	4.345
	Distri. factors $K/\Sigma K$	0.285	0.285	0.32	0.11	

Fixed end moments will be same as those calculated for Substitute Frame-I.

Moment Distribution :

Joint	4		5		
D.F.	0.64	0.36	0.32	0.57	0.11
F.E.M.		-120.00	120.00		-114.79
D.M.	76.80	43.20	- 1.67	- 2.97	- 0.79
C.O.M.		- 0.83	21.60		
D.M.	0.53	0.30	- 6.91	-12.31	- 2.38
C.O.M.		- 3.45	0.15		
D.M.	2.21	1.24	- 0.05	- 0.08	- 0.02
Mom. in Beam	79.54	- 79.54	133.12	-15.36	-117.76
Cols.	39.77			- 7.68	

Shears :

$V_{45} = 40 \times 6/2-(133.12-79.54)/6$
$= 111.07$ kN

$V_{54} = 40 \times 6-111.07 = 128.93$ kN

Maximum span moments :

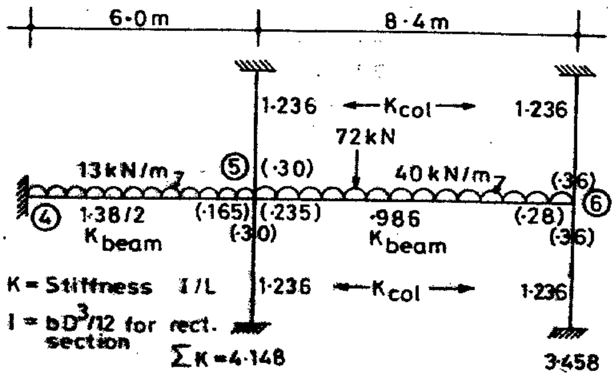
$x_{max} = 111.07/40 = 2.777$ m

$M_{max1} = 111.07 \times 2.777/2-79.54$
$= 74.68$ kN.m.

(b) Loading Case - II :

Maximum Midspan Moment in Span 5-6,
Maximum Support Moment at 6:

The substitute bay frame for this case will be as shown in Fig.7.4(b). In this case, span 4-5 shall carry maximum load while span 5-6 shall carry minimum load as in Case-II of Substitute Frame-I(Sect.7.7.4b). 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.



Bracketed values indicate Distribution Factors at the joint

Fig. 7.4(b) Substitute Frame - II (Bay Frame) Loading Case - II

		Column Upper	Column Lower	Beam Left	Beam Right	ΣK
Joint 5	Stiffness $K \times 10^{-6}$	1.236	1.236	1.38/2	0.986	4.148
	Distribution factor $K/\Sigma K$	0.30	0.30	0.165	0.235	
6	Stiffness $K \times 10^{-6}$	1.236	1.236	0.986	-	3.458
	Distribution factor $K/\Sigma K$	0.36	0.36	0.28	-	

Fixed end moments will be same as calculated for Substitute Frame - I.

Moment Distribution :

Joint	5			6	
Members	5-4	Cols.	5-6	6-5	Cols.
D.F.	0.165	0.60	0.235	0.28	0.72
F.E.M.	39.00		-321.49	265.82	
D.M.	46.61	169.50	66.38	-74.43	191.39
C.O.M.			- 37.21	33.19	
D.M.	6.14	22.23	8.74	- 9.29	- 23.90
C.O.M.			- 4.64	4.37	
D.M.	0.77	2.78	1.09	- 1.22	- 3.15
Moments in Beam,	92.52	194.61	-287.13	218.44	-218.44
Columns		97.30			-109.22

Shears :

$V_{65} = 40 \times 8.4/2+72 \times 2.2/8.4$
$-(287.13-218.44)/8.4$
$= 178.68$ kN

$V_{56} = 40 \times 8.4+72-178.68$
$= 229.32$ kN

$x_{max} = 178.68/40$
$= 4.467$ m.

$M_{max} = 178.68 \times 4.467/2-218.44$
$= 180.64$ kN.m.

(C) Loading Case - III : Maximum Support Moment at 5.

The substitute frame for this case will be as shown in Fig.7.4(c) which will have both spans with ends fixed. In this case, both spans will carry max.load just as in Case - III of Substitute Frame -I (Sect.7.7.4). The distribution factors at Joint 5 are given as under :

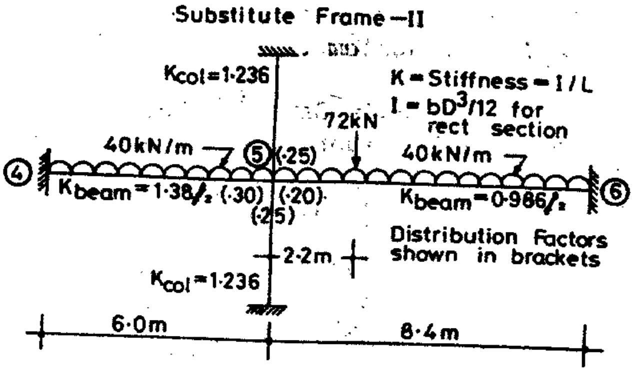


Fig. 7.4(c) Substitute Frame - II (Bay Frame) Loading Case - III

Joint	Column Upper	Column Lower	Beam Left	Beam Right
5 $K \times 10^{-6}$	1.236	1.236	1.38/2	.986/2
$d_i = K_i/\Sigma K$	0.338	0.338	0.189	0.135

Initial Fixed end moments remain same as those for Substitute Frame-I.

Moment Distribution :

Joint	4	5			6
Member :		5-4	Cols.	5-6	
D.F. :		.189	.676	.135	
Initial F.E.M. :	-120.00	120.00		-321.49	262.82
D.M. :		38.08	136.21	+27.20	
C.O.M. :	19.04				13.70
Final Moments :	-100.96	158.08	136.21	-294.29	276.52
Moment in each column:			68.10		

Shears :

V_{54} = (158.08-100.96)/6 +40x6/2
= 129.51 kN

V_{56} = 40x8.4/2+72x6.2/3.4 +(294.29-276.52)/8.4
= 223.25 kN

Results :

Case No.	Beam Moments in kN.m: Jt. 4	Midspan	5		Midspan	6	Moment in each column: 4	5	6
I	79.54	74.68	133.12	117.76	-	-	39.77	7.68	-
II	-	-	92.52	287.13	180.64	218.44	-	97.30	109.22
III	-	-	158.08	294.29	-	-	-	68.10	-
Maximum	79.54	74.68	158.08	294.29	180.64	218.44	39.77	97.30	109.22

Shears in kN.

Case	Joint 4	5		6
I	111.07	128.93		
II			229.32	178.68
III		129.51	223.29	
Maximum	111.07	129.51	229.32	178.68
Max.Column Load	111.07	358.83		178.68

7.7.6 Design Moments and Shear by Substitute Frame Method - III (Beam and Column Systems)

(A) 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 columns neglecting totally the fixity offered by the columns. All possible loading arrangements are considered.

Joint / Member	5-4	5-6
$KX10^{-6}$	1.38X.75=1.035	.986X3/4=0.74
$d_i = K_i/\Sigma K$	0.583	0.417

(Both members meet at joint 5.)

Initial fixed end moments will be same as those calculated for Substitute Frame Method - I (Sect. 7.7.4).

Substitute Frame - III : (a) Beam System

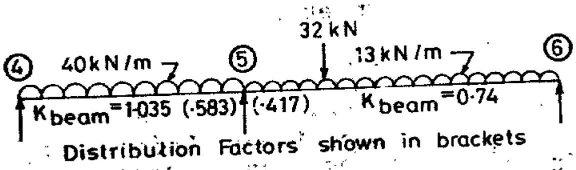


Fig. 7.5(a) Substitute Frame - III Loading Case I

(a) Loading Case - I : Maximum Midspan 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.7.5(a). The Distribution factors are calculated as under.

Moment Distribution :

Joint	4	5		6
D.F.		.583	.417	
Initial F.E.M	-120.00	120.00	-114.79	90.05
D.M	+120.00	-3.04	-2.17	-90.05
C.O.M.	-	60.00	-45.02	
D.M		-8.73	-6.25	
Final Moment	0	168.23	-168.23	

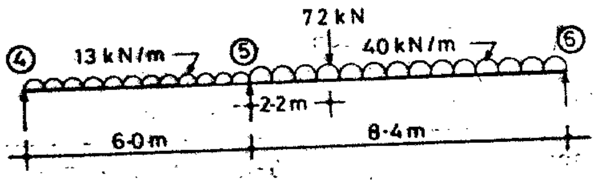


Fig.7.5(b) Substitute Frame- III Loading Case II

V_{45} = 40X6/2-168.23/6 = 91.96 kN

V_{54} = 40X6 -91.96 = 148.04 kN

V_{56} = 13X8.4/2 + 32X6.2/8.4 + 168.23/ 8.4 = 98.25 kN

V_{65} = 13X8.4 + 32 - 98.25 = 42.95 kN

Span 4-5 : x_{max} = 91.96/40 = 2.3 m

M_{max}= 91.96X2.3/2 = 105.75 kN.m.

(b) Loading Case - II : Maximum Midspan 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.7.5(b). Distribution factors remain unchanged.

Moment Distribution :

Joint	4	5		6
D.F		.583	.417	
Initial F.E.M	-39.00	39.00	-321.49	265.82
D.M	39.00	164.69	117.80	-265.82
C.O.M.	-	19.50	-132.91	
D.M.		66.12	47.29	
Final Moments	0	289.31	-289.31	0

V_{45} = 13X6/2-289.31/6 = -9.22 kN

V_{54} = 13X6 + 9.22 = 87.22 kN

V_{56} = 40X8.4+72X6.2/8.4 + 289.31/8.4 = 255.58 kN

V_{65} = 40X8.4+72-255.58= 152.42 kN

Span 4-5 : No positive max. moment

M_{centre} = -9.22X3 - 13X3X3/2 = -86.16 kN.m

Span 5-6 :

x_{max} = 152.42/40 = 3.81 m

M_{max2} = 152.42X3.81/2 = 290.36 kN.m

(c) *Loading Case – III*
Maximum Support Moment at 5.

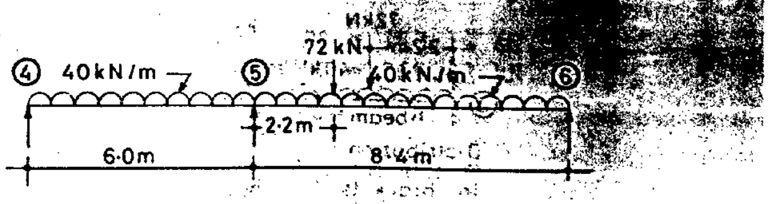


Fig. 7.5(c) Substitute Frame – III
Loading Case III

In this case, maximum load shall occur on both the spans 4-5 and 5-6 as shown in Fig.7.5(c). Distribution factors remain unchanged.

Moment Distribution

Joint	4	5		6
D.F.		.583	.417	
Initial				
F.E.M	-120.00	120.00	-321.49	265.82
D.M.	120.00	117.47	84.02	-265.82
C.O.M.	-	60.00	-132.91	-
D.M.		42.51	30.40	
Final Moments	-	339.98	-339.98	0

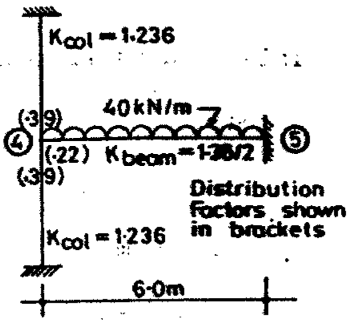
V_{45} = 40X6 – 176.66 = 63.34 kN
V_{54} = 40X6/2+339.98/6 = 176.66 kN
V_{56} = 40X8.4/2+72X6.2/8.4 + 339.98/8.4 = 261.62 kN
V_{65} = 40X8.4+72–261.62 = 146.38 kN
x_{max1} = 63.34/40 = 1.5835 m
M_{max1} = 63.34X1.5835/2 = 50.15 kN.m
x_{max2} = 146.38/40 = 3.6595 m
M_{max2} = 146.38X3.6595/2 = 267.84 kN.m

Results for Beam Moments and Shears without the effect of columns :

Case No.	Beam Moments: Jt.4	Midspan	5	Midspan	6	Shears: 4	5		6
I	0	105.75	168.23	–	0	91.96	148.04	98.25	42.95
II	0	–	289.31	290.36	0	-9.22	87.22	255.58	152.42
III	0	50.15	339.98	276.84	0	63.34	176.66	261.62	146.38
Maximum	0	105.75	339.98	290.36	0	91.96	176.66	261.62	152.42

(B) 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. The relevant column systems with loadings for maximum moments in columns are shown in Fig.7.6.



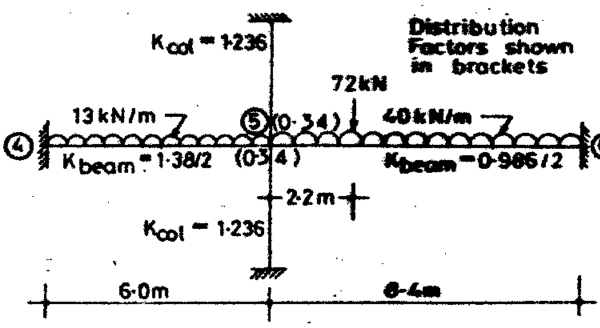
Substitute Frame III
Column System : Joint-4

Joint-4 Case – I :

ΣK = 1.236+1.236+1.38/2 = 3.162
Distribution factor for column:
D_{col} = 1.236/3.162 = 0.39
M_{col} = 0.39x40x6²/12 = 46.8 kN.m.

Joint-5 Case-III :

ΣK = 1.236+1.236+1.38/2+0.986/2=3.655
D_{col} = 1.236/3.655 = 0.34
Unbalanced moment at the joint –
M_e = 72x2.2x6.2² /8.4² + 40X8.4² /12 –13x6² /12 = 282.49 kN.m
M_{col} = 0.34x282.49 = 96.05 kN.m.



Substitute Frame – III Column System : Joint – 5

Fig 7.6.
Loading Conditions for Maximum Moments in Columns.

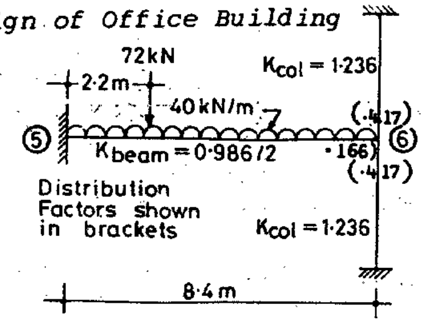


Fig. 7.6 Substitute Frame III Column System : Joint-6

Loading Conditions for maximum moments in Columns

Joint-6 Case-II-

ΣK = 1.236+1.236+0.986/2
= 2.965

D_{col} = 1.236/2.965
= 0.417

M_e = $72 \times 2.2^2 \times 6.2/8.4^2 + 40 \times 8.4^2/12$
= 265.82 kN.m

M_{col} = 0.417X265.82
= 110.85 kN.m.

Results :

	Column Moments		
Joint	4	5	6
Moment	46.8	96.05	110.85 kN.m

The beam moments and shears will now be modified to account for the effect of column moments.

Modification of Beam moments :
Beam System : Case - I

M_{45} in beam = $2 \times M_{col}$ = 2x46.8 = 93.6 kN.m
This reduces the midspan moment by 93.6/2 = 46.8 kN.m

Modified Shear : V_{45} = 91.96 + 93.6/6 = 91.96 + 15.6 = 107.56 kN.
x_{max} = 107.56/40 = 2.689 metres from end 4.
Modified max. span moment = 107.56x2.689/2 - 93.60 = 51.01 kN.m.

Beam System Case-II :

M_{65} in beam = $2 \times M_{col}$ = 2x110.85 = 221.70 kN.m.
This reduces midspan moment by 221.70/2 = 110.85 kN.m.

Modified shear : V_{65} = 152.42 + 221.70/8.4
= 152.42 + 26.39 = 178.81 kN.
x_{max} = 178.31/40 = 4.47 metres from end 6.

Modified max.span moment = 178.81x4.47/2 - 221.70 = 177.94 kN.m.

Beam System Case-III :

Since Case-III of Column system does not match with Case-III of beam system, moments in intermediate columns will be obtained for loading Case-III of beam system instead of that shown in Case-III of column system.

Unbalanced moment at 5 =321.49-120.00=201.49 kN.m
This will cause moment in column = 0.34x201.49 = 68.50 kN.m.(Distribution Factor is taken from Case-III of column system).
Total moment in two columns = 2x68.50 = 137.0 kN.m.

This is now to be distributed in beams as M_{54} and M_{56} in propertion of their stiffnesses.

Reduction in M_{54} = (1.38/2)/(1.38/2+0.986/2)x137.0 = 79.91 kN.m
∴Modified moment M_{54} = 339.98 - 79.91 = 260.07 kN.m
Increase in M_{56} = 137.00 - 79.91 = 57.09 kN.m
Modified moment M_{56} = 339.98 + 57.09 = 397.07 kN.m.

Modified shears : V_{54} = 176.66 − 79.91/6 = 176.66 − 13.32 = 163.34 kN.
V_{56} = 261.62 + 57.09/8.4 = 261.62 + 6.80 = 268.42 kN.
V_{45} = 63.34 + 13.32 = 76.66 kN.
V_{65} = 146.38 − 6.80 = 139.58 kN.

Modified max.span momens : x_{max1} = 76.66/40 = 1.9165 metres from end 4.
M_{max1} = 76.66x1.9165/2 = 73.46 kN.m.
x_{max2} = 139.58/40 = 3.4895 metres from end 6.
M_{max2} = 139.58x3.4895/2 = 243.53 kN.m.

Results of modified moments :

Joint		4		5	5		6
Moments from Beam Sustem							
Case − I	Initial moments	0	105.75				
	Modified moments	93.60	51.01				
Case −II	Initial moments					290.36	0
	Modified moments					177.94	221.70
Case−III	Initial moments		50.15	339.98	339.98	276.84	
	Modification		73.46	260.07	397.07	243.53	
Modified Maximum moments		93.60	73.46	260.07	397.07	243.53	221.70

Results of Modified Shear :

Joint		4	5	5	6
Case − I	Initial shear	91.96	148.04		
	Modification	+15.60	−15.60		
	Final shear	107.56	132.44		
Case −II	Initial shear			255.58	152.42
	Modification			−26.39	+26.39
	Final shear			229.19	178.81
Case−III	Initial shear		176.66	261.62	
	Modification		−13.32	+6.80	
	Final shear		163.34	268.42	
	Maximum shear	107.56	163.34	268.42	178.81

7.7.7 Comparison of Results of Three Methods

(a) Beam moments

Joint / Method	4		5	5		6
I	78.16	73.42	203.77	316.45	177.16	213.90
II	79.54	74.68	158.08	294.29	180.64	218.44
III-Unmodified	0	105.75	339.98	339.98	290.36	0
Modified	93.60	73.46	260.07	397.07	243.53	221.70

Observations for Beam Moments

Methods I and II give practically 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 lower 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 midspan 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). If beam moments are modified and columns designed for moments obtained by column system, it may be observed that modified values of beam moments are still on the higher side.

(b) Beam Shears

Comparison of Joint / Method	4	5		6
I	110.12	144.64	234.17	176.87
II	111.07	129.51	229.32	178.68
III-Modified	107.56	163.34	268.42	178.81

(c) Column Moments

Comparison of Joint / Method	4	5	6
I	39.08	81.07	106.95
II	39.77	97.30	109.22
III	46.80	96.05	110.85

Observations for Beam Shear, Column Moments and Concluding Remarks :

The values of beam shears and column moments obtained by Method-II do not deviate much from those of Method-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 %. Initial values of moments and shears of Method-III deviate too much from the values of Method-I and therefore use of Method- III without modification should definitely be avoided. Modified values of Method-III are all on the higher side in comparison with the values of Method-I. Therefore, it can be concluded that the Substitute Frame Method- I should always be preferred to other two methods. Method - II may be adopted when number of spans greater than 3 as in that case Method -I is likely to be laborious and susceptible to computational errors. However, Method -I can definitely be used for all spans when computer is available. Method -III may be used exceptionally and that too with necessary modifications.

The design of frame in the subsequent sections has been done using results of Method -I. The results for top frame and bottom frame have been obtained by using stiffness method taking only joint rotations as unknowns and neglecting the axial deformations. The results obtained by using computer program prepared by the authors are given for top frame, intermediate frames and bottom frame. The results of moment distribution with three cycles of distribution done by hand calculations are nearly equal to the values obtained from computer program. See Sect.7.7.9. It may not be difficult to work out the moments in top and bottom frames also by hand calculations; however, the same may be done by the reader if he so desires.

7.7.8 Computer Results using Stiffness Method for Substitute Frame – I :

(I) TOP STOREY FRAME

(a) Beam and Column Moments in kN.m

Loading Case No.	Joint 1	Max. Span Moment	2		Max. Span Moment	3
I	50.55	70.07	128.87	124 93	62.11	54.73
II	37.91	58.18	175.68	220.02	138.22	124.32
III	5.68	20.64	132.13	201.95	143.25	129.97
Maxium Moment						
in Beam	50.65	70.07	175.68	220.02	143.25	129.97
in Column	50.65		44.34			129.97

(b) Shear in Beam in kN.

Loading Case No.	Jt. 1	2		3
I	91.96	118.04	79.76	63.04
II	82.04	127.96	158.39	135.61
III	29.92	72.08	155.57	138.43
Maximum Shear	91.96	127.96	158.39	138.43
Load on Column	91.96	286.35		138.43

(II) MIDDLE STOREY FRAME

Beam and Column Moments in kN.m

Loading Case No.	Moments in	4	Max. Span Moment	5		Max.Span Moment	6
I	Beam	78.33	73.13	137.75	130.36	53.01	63.32
	each Column	39.17		3.69			31.66
II	Beam	56.23	58.88	200.45	316.43	170.50	206.57
	each column	28.12		58.99			103.29
III	Beam	4.86	15.16	130.78	297.60	176.49	213.81
	each column	2.43		83.41			106.91
Maximum Moment							
	in Beam	78.33	73.13	200.45	316.43	176.49	213.81
	in column	39.17		83.41			106.91

(b) Shear in Beam in kN

Loading Case No.	Jt. 4	5		6
I	110.10	129.90	86.20	55.00
II	95.96	144.04	234.22	173.78
III	16.39	61.61	231.12	176.88
Maximum Shear	110.10	144.04	234.22	176.88
Max.Column Load	110.10	378.26		176.88

(III) BOTTOM STOREY FRAME

(a) Beam Moments in kN.m

Loading Case No.:	Jt 10	Max. Span Moment	11	11	Max. Span Moment	12
I	69.71	76.34	141.14	134.11	54.14	57.14
II	49.63	61.34	204.20	290.21	159.89	174.80
III	9.71	15.96	147.50	295.16	185.27	198.29
Maximum	69.71	76.34	204.20	295.16	185.27	198.29

(b) Beam Shear in kN

Loading Case No.:	Jt 10	11	11	12
I	108.10	131.90	87.38	53.82
II	94.24	145.76	244.31	163.69
III	12.80	65.20	232.68	175.32
Maximum Shear	108.10	145.76	244.31	175.32
Max. Column Load	108.10	390.07		175.32

(c) Column Moments in kN.m

Loading Case No.	Joint: 10 Column - Upper	10 Lower	11 Upper	11 Lower	12 Upper	12 Lower
I	47.43	22.28	4.78	2.25	38.87	18.26
II	35.76	15.86	58.52	27.49	118.92	55.87
III	6.61	3.10	100.46	47.20	134.90	63.38
Maximum Moment	47.43	22.28	100.46	47.20	134.90	63.38

7.7.9 Comparison of Results of Middle Storey Frame obtained by Moment Distribution method by hand calculation with those obtained by Stiffness Method by Computer

(a) Beam Moments in kN.m

Method	Joint: 4	Max. Span Moment	5	5	Max. Span Moment	6
Moment Distribution	78.16	73.42	203.77	316.45	177.16	213.90
Stiffness Method	78.33	73.13	200.45	316.43	176.49	213.81

(b) Column Moments in kN.m (c) Beam Shears in kN

Method	(b) Joint: 4	(b) 5	(b) 6	(c) 4	(c) 5	(c) 5	(c) 6
Moment Distribution	39.08	81.07	106.95	110.12	114.64	234.17	176.87
Stiffness	39.17	83.14	106.91	110.10	144.04	231.12	176.88

Conclusion : If hand calculations are to be done in case of non-availability of computers, two cycles (in addition to initial) moment distribution is more than sufficient. The design of frame members has been done based on computer results.

7.8 DESIGN OF BEAMS

7.8.1 Design of Middle Storey Transverse Beam 4-5-6

Design is presented here only for floor beam in a compact tabular form.

Step No.	Description	Unit	Joint 4	Midspan	5	5	Midspan	6	Note No.
1.	Span L	mm		6000			8400		
2.	End Condition		Fixed		Continuous			Fixed	
3.	Section (Assumed) b	mm		230			230		
	D	mm		600			600		
	d'	mm		50			50		
	d	mm		550			550		
	D_f	mm		110			110		
	b_s	mm		600			600		
4.	Load : UDL w_u	kN/m		40.00			40.0		
	: PTL P_u	kN		–			72.00		
	Dist. from left	m		–			2.2		
5.	$M_{u.max}$	kN.m	78.33	73.13	200.45	316.43	176.49	213.81	
6.	$M_{ur.max}$	kN.m	144.02	144.02	144.02	144.02	144.02	144.02	(1)
	Section Type		SR	SR/(T)	DR	DR	T	DR	(2)
	L_o	mm		(4200)			5880		
	b_f	mm		(1590)			1870		
	M_{u1} for $x_u = D_f$	kN.m		(476)			560		(3)
	x_u	mm		< Df			< Df		
	b	mm	230	230/(1590)	230	230	1870	230	
	$R_u = M_u/(bd^2)$	N/mm²	1.13	1.05	2.88	4.55	.312	3.07	
7.	Main Steel :								
	p_t	%	.346	.319/(.043)	.966	1.48	.0895	1.025	(4)
	p_c	%			.261	.798		0.32	
	Required A_{st}	mm²	438	404/(373)	1222	1872	921	1297	
	Required A_{sc}	mm²			330	1010		406	
	Top St. N-ø	mm	4-#12	2-#12*	4-#20	6-#20	*2-#12(@2+2)#20		(5)
	Bottom Bt. N-ø	mm	–	–	–	–	@2-#20	–	
	Bottom St. N-ø	mm	2-#16	2-#16	2-#16	@2#20+2#16	2-#16	2-#16	
	Provided A_{st}	mm²	452	402	1256	1884	1030	1256	
	Provided A_{sc}	mm²			402	1030		402	
8.	Shear :								
	$V_{u.max}$	kN	110.10		144.04	234.22		176.88	(6)
	Loading Case	No.	I		III	III		II	
	N_1 – ø	mm	4-#12		4-#20	6-#20		4-#20	
	A_{st1}	mm²	452		1256	1884		1256	
	V_{uc}	kN	51.30		76.90	85.77		76.90	(7)
	$V_{usv.min}$	kN	44.27		44.27	44.27		44.27	(8)
	$V_{ur.min}$	kN	95.57		121.17	130.04		121.17	
	V_{uD}	kN	76.10		110.04	200.22		142.88	(9)
	$V_{us} = V_{uD} - V_{uc}$	kN	–		–	114.45		65.98	(10)
	Des.Stir. N-ø	mm	–		–	#8-175		#8-300	(11)
	L_{s1} and N_1	mm	–		–	2200/14		1395/6	(12)
	Dist. of PI x_1	mm	–		–	1558		1445	(13)
	Min. Stir. N-ø	mm	#8-400		#8-400	–	#8-400	–	(14)

Design of Beam 4-5-6 continued

Step No.	Description	4	PI LEFT	PI RT.	5		PI LEFT	PI RT.	6	Note No.
9.	Bond									(15)
	Max. Bar diam ϕ mm	12	16	16	20	20	20	20	20	
	Req. $L_d = 57\phi$ mm	684	912	912	1140	1140	1140	1140	1140	
	$L_{ex} = d$ mm		550	550			550	550		
	$N_1 - \phi$ mm		2-#16	2-#16			2-#16 +2-#20	2-#16 +2-#20		
	A_{st1} mm²		402	402			1030	1030		
	Available M_1 kN.m		73.13	73.13			196.52	196.52		
	V_1 kN		76.52	76.52			171.90	119.08		
	Available L_d mm	1500	1505	1505	1500	2100	1693	2200	2100	
10.	Deflection									
	p_t at midspan						0.0887			
	α_1						1.9			
	b_w/b_f						.12			
	$\alpha 3$						.8			
	Basic L/d ratio						26			
	Allwable L/d ratio						39.5			
	Actual L/d ratio						15.2			

Explanatory Notes and Calculations :

(1) $M_{ur.max}$ for rectangular section $=R_{u.max}\,bd^2$. For M15-Fe415, $R_{u.max} = 2.07 N/mm^2$. Therfore, $M_{ur.max} = 2.07\ bd^2 \times 10^{-6}$ kN.m.

(2) Since $M_{u.max}$ is far less than $M_{ur.max}$ of rectangular section, there is marginal advantage in designing the section as a flanged section. If so desired, the calculations give the values shown in brackets.

(3) In order to determine whether the neutral axis lies in the flange or web, M_{ur1} is required to be calculated for $x_u = D_f$ using Eq.4.1-20.

(4) Values of required p_t are obtained by using Table D-4 for Singly reinforced. Rectangular section(SR) and by using Table D-5 for Doubly reinforced Rectangular (DR) section. For flanged section, it is obtained using Eq. 4.1-17b.

(5) Number Diameter (N-ø) combination is selected for required A_{st} using Tables D-6(a),(b),(c).

(6) $V_{u.max}$ is taken directly from results of frame analysis[Sec.7.7.8 Table(b)].

(7) V_{uc} corresponding to available N1-ø combination of tension steel at critical section is obtained by using Eq.4.2-3

(8) Value of $V_{usv.min}$ is obtained by using Table D-9.

(9) V_{uD} is calculated at a distance $(b_s/2+d)$ from the centre of support using Eq.(4.2-1a) only when it is found that $V_{ur.min} < V_{u.max}$.

(10) $V_{us} = V_{uD} - V_{uc}$.

(11) Design of stirrups is done using Table D-8.

(12) $L_{s1} = (V_{u.max} - V_{ur.min})/w_u$ for beams carrying UD Loads. Number of stirrups, $N1 = (L_{s1}/s)+1$ rounded to integer value on higer side.

(13) If $L_{s1} > x_1$ - the distance of Point of Inflection (PI), shear design is required to be done afresh beyond PI because the tension steel beyond PI changes and it is the bottom steel. In this case, for span 5-6, since L_{s1} works out to be equal to 2604mm but limited to 2200 mm (i.e. upto point load) which is greater than the distance of PI from the support, the calculations for distance of PI and shear in the positive moment region are given seperately below.

(13)(a) Distance of left PI from left support (Jt. 5) : Loading Case - III.
V_{56} = 234.22 kN, M_{56} = 316.43 kN.m, w_u = 40 kN/m. Point load = 72 kN at a distance 2.2 m from left support.

Let x_1 be the distance of PI from left support.

$234.22\, x_1 - 40\, x_1^2/2 - 316.43 = 0$. On solving, $x_1 = 1.558$ m.

V_1 at PI = 234.22 - 40x1.558 = 171.90 kN > $V_{ur.min}$ of top bars.

.'. Designed stirrups will be applicable only upto PI. Design for shear will have to be done afresh beyond PI taking bottom bars as tension steel.

At PI, A_{st} = 2-#16 + 2-#20 = 1030 mm² (bottom bars).

V_{uc} corresponding to this = 70.71 kN using Eq.4.2-3.

V_{us} = 171.9 - 70.71 = 101.19 kN.

From Table D-8, required stirrups would be #8 at 200 mm. Thus, spacing of #8 stirrups is required to be 175 mm upto PI and 200 mm beyond PI. However, for simplicity same spacing of 175 mm may be provided right upto point load.

For bottom bars, $V_{ur.min}$ = V_{uc} + $V_{usv.min}$ = 70.71 + 44.27 = 114.98 kN.

V_u just to the left of point load (at x = 2.2 m from left support)
= 234.22 - 40x2.2 = 146.2 kN.

V_u just to the right of point load = 146.22-72.00 = 74.22 kN < $V_{ur.min}$.

.'. Minimum stirrups are adequate beyond the point load.

(b) Distance of right PI from right support (Jt.6) : Loading Case = II.
V_{65} = 176.88 kN, M_{65} = 213.81 kN.m, w_u = 40 kN/m.
$x_{max} = V/w_u$ = 176.88/40 = 4.422 m.

Distance of PI from right support,

$$x_2 = x_{max} - \sqrt{x_{max}^2 - 2M_A/w_u} = 4.422 - \sqrt{4.422^2 - 2\text{x}213.81/40}$$
$$= 1.445 \text{ m} = 1445 \text{ mm.}$$

At PI, V_2 = 176.88 - 40x1.445 = 119.08 kN.

Since, L_{s1}= 1547 mm > x_2 (=1445 mm), entire shear zone does not lie in the negative moment region. Eventhough the part of shear zone extends in the positive moment region , it is not necessary to calculate $V_{ur.min}$ for positive region again because minimum steel is taken for computation of $V_{ur.min}$.

At PI, A_{st1} = same as that at left PI.

.'. $V_{ur.min}$ = 114.98 kN < V_2 (=119.08 kN).

.'. Minimum stirrups are not sufficient beyond PI.

V_{us} = 119.08 - 70.71 = 48.37 kN.

Provide #8 at 400 mm as obtained by Table D-8.

It may be noted that in fact requirement of minimum stirrups is #8 at 450 mm (See Table D-7). But the spacing has to be limited to 0.75d = .75x550 = 412 mm which gives practical spacing of 400 mm. This spacing is adequate in the region beyond PI also as seen above.

(14) Minimum stirrups are obtained using Table D-7. Actual spacing shall not exceed 0.75d = 0.75x550 = 412 mm say 400 mm.

(15) In span 5-6, at left PI, available A_{st1} = 1030 mm².

Required A_{st} at midspan = 925 mm² for $M_{u.max}$ = 176.49 kN.m.

.'. Available M_1 = (1030/925)x176.49 = 196.52 kN.m.

Since check for development length is required to be applied to bottom (positivce moment) steel at PI, their positions are required to be determined.

$V_{u.max}$ = 110.10 kN, $M_{u.max}$ = 78.33 kN.m, w_u = 40 kN/m - Loading Case - I.

x_{max} = 110.10/40 = 2.7525 m from Joint 4.

$$x_1 = 2.7525 - \sqrt{2.7525^2 - 2\text{x}78.33/40} = 2.7525 - 1.913 = 0.8395 \text{ m.}$$
$$x_2 = 6.00 - (2.7525 + 1.913) = 1.3345 \text{ m.}$$

V_u at PI = V1 = 110.10 - 40x0.8395 = 76.52 kN.

7.8.2 Design of Middle Storey Longitudinal Beams

Step No.	Details	Floor Beams		Reference or Note No.
		External	Internal	
1.	Span m	3.05	3.05	
2.	End Copndition No.	3	3	
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	35	35	
	Effective depth d mm	415	265	
4.	Loads : w_u kN/m	30.60	23.37	Sect.7.7.2(B)
5.	$M_{u.max} = w_u L^2/12$ kN.m	23.72	18.12	
6.	Main Steel :			
	(a) At Midspan Top St.N − ø mm	2-#10*	2-#10*	*Anchor bars
	Bottom Bt.N − ø mm	1-#10	1-#10	Table D-1
	Bottom St.N − ø mm	2-#10	2-#10	Table D-1
	Provided A_{st} mm²	236	236	
	Provided d mm	420	270	
	Provided M_{ur} kN.m	33.24	20.51	
	(b) At Support Top St.N − ø mm	3-#10	3-#10	Note No.1
	Bottom St.N − ø mm	2-#10*	2-#10*	
	Provided A_{st} mm²	236	236	
	Provided d mm	420	270	
	Provided M_{ur} kN.m	33.24	20.51	Note No.2
7.	Shear : $V_{u.max} = w_u L/2$ kN	46.67	35.64	
	A_{st1} N − ø mm	3-#10	3-#10	
	V_{uc} kN	33.21	24.80	Table 4.2
	$V_{usv.min}$ kN	33.81	21.73	Table D-9
	$V_{ur.min}$ kN	67.02	46.53	
	Stirrups-Minimum ø-s mm	ø6-150	ø6-150	Table D-7
8.	Bond : Maximum bar diam. ø mm	10	10	
	Required $L_d = 57\,ø$ mm	570	570	
	Available L_d −			
	(a) At Supports: L/4 for top bars	762	762	∴ O.K.
	(b) At Point of Inflexion (PI) for bottom bars: $L_d = M_1/V_1 + L_{ex}$			
	Available N_1 − ø mm	2-#10	2-#10	
	Available $M_1 = M_{ur}$ kN.m	22.45	14.23	Eq.(4.1-4b)
	Assuming distance of PI from adjacent support $x_1 = L/5$ mm	610	610	
	$V_1 = V_{u.max} - w_u x_1$	28.00	28.00	
	$L_{ex} = L_d/3 - b_s/2$	175	175	
	Available L_d mm	977	683	∴ O.K.

Note : - 1 : At support, 2-#10 are obtained by continuation of top anchor bars from midspan, and 1-#10 is obtained by single bar bent up from left span and and again going to bottom in right span.

Note :- 2 : For external beam, though provided M_{ur} appears to be very 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.)

Design of Longitudinal Beams continued

Step No.	Details	External Beam	Internal Beam	Reference
9.	Deflection : p_t %	.23	.365	
	Allowable L/d ratio	> 26	> 26	
	Actual L/d ratio	< 26	< 26	.. O.K.
10.	End Shear for column load excluding triangular slab load kN	[30.6 − 1.5x (1.91+2.03)] x3.05/2=37.65	[23.37 − 1.5x (3.82+4.06)]x 3.05/2=17.61	
	Total load on column due to end shear fron two beams kN	75.30	35.22	

7.8.3 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.7.7.2(A)(3) and Sect.7.7.8 respectively as the same are required for design of columns.

7.9 DESIGN OF COLUMNS

Column in Frame		Left	Middle	Right	Reference
Column Mark		C22	C13	C3	
7.9.1 Category of Column		II	I	II	Sect.5.4.3

7.9.2 Calculation of Column Loads in different storeys :

(A) Exact Method

Column between		Left	Middle	Right	Reference
(a) *Roof to 3rd Floor (R-3)* :					
Assumed Section b (mm) x D (mm)		230x600	230x600	230x600	
Floor to floor Height m		3.35	3.35	3.35	
Factored self weight = 1.5x25x.23x.6x3.35 = 17.34	kN	17.34	17.34	17.34	
Max.shear from Transverse beam	kN	91.96	286.35	138.43	Sect.7.7.8-I(b)
Shear from Longitudinal beams	kN	27.91	5.03	27.91	Sect.7.7.2(A-1)
Total	kN	137.21	308.72	183.68	
Roof Load : P_r say	kN	137	309	184	
Ratio of LL/ Total Load for transverse beam at roof level		0.196	0.196	0.196	Sect.7.7.2(A-2)
Beam shear due to LL = above ratio x transverse beam shear = P_{rL}	kN	18.02	56.12	27.13	This will be required later :for column load :in lower storey
Load from above	kN	Nil	Nil	Nil	
Total Gross load = $P_{r3.gr}$	kN	137	309	184	
Reduction in Load due to reduction in LL on upper floors	kN	Nil	Nil	Nil	
Net Design Load = $P_{r3.net}$	kN	137	309	184	

Column Mark		C22	C13	C3	Reference
(b) 3rd-Floor to 2nd Floor (3-2)					
Max.shear from Transverse beam	kN	110.10	378.26	176.88	Sect.7.7.8-IIb
Shear from Longitudinal beams	kN	75.30	35.22	75.30	Sect.7.8.2
Self weight	kN	17.34	17.34	17.34	
Total load at each mid-floor	kN	202.74	430.82	269.52	
3rd Floor Load :	kN	203	431	270	
Ratio of LL/Total Load for transverse beam at each floor level		0.46	0.46	0.46	Sect.7.7.2(B-2)
Beam shear due to LL at each floor level = above ratio x transverse shear from floor beam = P_{fL}	kN	50.65	174.00	81.36	:This is required :later for column :load in lower :column storeys
Load from above = P_r	kN	137	309	184	
Total gross load = $P_{32.gr}$	kN	340	740	454	
Less:Reduction in LL on upper floors : Reduction factor = r_3		0.1	0.1	0.1	
Total shear due to LL on upper floors P_{r3L} = $P_{rL} + 1xP_{fL}$	kN	68.67	230.12	108.49	
Actual load reduction = $r_3 x P_{r3L}$	kN	say 7	23	11	
Net Design Load = $P_{32.net}$:	kN	333	717	443	
(c) 2nd Floor to 1st Floor (2-1)					
2nd floor load P_2 = P_f	kN	203	431	270	
Load from above = $P_r + P_f$	kN	340	740	454	
Total gross load $P_{21.gr}$ = $P_r + 2P_f$	kN	543	1171	724	
Less:Reduction in LL on upper floors : Reduction factor = r_2		0.2	0.2	0.2	
Total shear due to LL on upper floors P_{r2L} = $P_{rL} + 2xP_{fL}$	kN	119.32	404.12	189.85	
Actual load reduction = $r_2 x P_{r2L}$	kN	say 24	81	38	
Net Design Load = $P_{21.net}$	kN	519	1090	686	
(d) 1st Floor to Plinth (1 - Pl)					
1st floor load P_1 = P_f	kN	203	431	270	
Load from above = $P_r + 2P_f$	kN	543	1171	724	
Total gross load $P_{1P.gr}$ = $P_r + 3P_f$	kN	746	1602	994	
Less:reduction in LL on upper floors : Reduction factor r_p		0.3	0.3	0.3	
Total shear due to LL on upper floors P_{rpL} = $P_{rL} + 3P_{fL}$	kN	169.97	578.12	271.21	
Actual load reduction = $r_p x P_{rpL}$	kN	say 51	173	81	
Net Design Load = $P_{1PL.net}$	kN	695	1429	913	
(e) Plinth to Footing ($P_l - F_t$)					
Load from longi.plinth beams	kN	90	45	90	Sect.7.7.2(C)
Self weight of column 2m high = (2/3.35)x17.34 = 10.35 say	kN	10	10	10	
Load from above = $P_r + 3P_f$	kN	746	1602	994	
Total gross load $P_{Pf.gr}$	kN	846	1657	1094	
Less:Reduction in LL on upper floors = same as that at pl. level	kN	51	173	81	
Net Design Load = $P_{PLFT.net}$	kN	795	1484	1013	

7.9.2 Calculation of Column Loads in Differnt Storeys: (Continued)

(B) Approximate Method

This is used when columns and footings are to be designed prior to analysis and design of frame and beams.

I - Assessment of Unit Loads

(A) Slab (D = 110 mm) : Intensities of Loads in kN/m².

		Working Load	Ultimate Load
Roof :	DL	q_{d1} = 25xD + FF = 25x.11+2.5 = 5.25 kN/m²	q_{ud1} = 1.5xqd = 1.5x5.25 = 7.875 kN/m²
	LL	q_{l1} = 1.5 = 1.50 kN/m²	q_{ul1} = 1.5x1.5 = 2.25 kN/m²
	Total	q_1 = 5.25 + 1.5 = 6.75 kN/m²	q_{u1} = 7.875+2.25 =10.125 kN/m²
Floor:	DL	q_{d2} = 25x.11+1.0 = 3.75 kN/m²	q_{ud2} = 1.5x3.75 = 5.625 kN/m²
	LL	q_{l2} = 4.0 = 4.00 kN/m²	q_{ul2} = 1.5x4.00 = 6.00 kN/m²
	Total	q_2 = 3.75 + 4.0 = 7.75 kN/m²	q_{u2} = 5.625+6.00 =11.625 kN/m²

(B) Wall and Beam* : Weight of wall + beam in kN/metre length.

***Note** : In approximate method, weight of beam need not be considered seperately, but the wall load may be taken for full height (i.e. floor to floor height or actual height of wall plus the depth of beam below) ignoring the difference between the unit weight of concrete and unit weight of masonry and also between widths of wall and beam if any.

Roof Level:External Parapet:250mm thick,(1m +.3m roof beam depth)=1.3m high.

$w_{w.ex1}$ = 20x.25x1.3 = 6.50 kN/m

$w_{u.exw1}$ = 1.5x6.5 = 9.75 kN/m

:Internal Beam only: $w_{u.in1}$ =1.5x25x.23x.3=2.6kN/m

Floor Level: External : 250 mm thick, 3.35 m high*

$w_{w.ex2}$ = 20x.25x3.35 = 16.75 kN/m

$w_{uw.ex2}$ = 1.5x16.75 = 25.125 kN/m

Internal : 150 mm thick, 2.2+0.3 = 2.5 m high*

$w_{w.in2}$ = 20x.15x2.5 = 7.50 kN/m

$w_{uw.in2}$ = 1.5x7.50 = 11.25 kN/m

Plinth Level:External : 250 mm thick,(3.35m +.5m plinth height)=3.85m high.

$w_{w.ex3}$ = 20x.25x3.85 = 19.25 kN/m

$w_{uw.ex3}$ = 1.5x19.25 = 28.875 kN/m

Internal : 150 mm thick,(2.2+0.3+0.5m plinth height) =3m high.

$w_{w.in3}$ = 20x.15x3.00 = 9.00 kN/m

$w_{uw.in3}$ = 1.5x9.00 = 13.50 kN/m

(C) Column Weight : This may be ignored in approximate method since wall load is taken from centre to centre of columns. However the same is considered here because in exact method neither the wall load is taken between the inn-er faces of columns nor the column weight excluded.

$P_{u.self}$ = 1.5x25x.23x.6x3.35 = 17.34 kN for middle storeys.
Weight of extra length of column for plinth height of 0.5 m = 17.34x.5/3.35 = 2.6 kN.
Weight of column between first floor and plinth = 17.34 + 2.6 = say 20 kN.
Weight of column below plinth for height 2 m = 17.34x 2 /3.35 = 10 kN.

II - Assessment of Loads on Columns in Different Storeys

Column Mark		C22	C13	C3
1. Column Load Area : A_{col}	m^2	3.05x6/2 = 9.15	3.05(6+8.4)/2 = 21.96	3.05x8.4/2 = 12.81
Wall Length L_w	m	3.05	3.05x6.2/8.4+ 3.05 = 5.30	3.05x2.2/8.4 + 3.05
2. Load on Column in each storey :				
(a) Column between Roof and 3rd Floor (R - 3)				
Slab:Total:$P_{us} = q_{u1} \times A_{col}$	kN	10.125x9.15 = 92.64	10.125x21.96 = 222.35	10.125x12.81 = 129.70
*Live :$P_{rL} = q_{uL1} \times A_{col}$	kN	2.25x9.15 = say 21	2.25x21.96 = say 49	2.25x12.81 = say 29
*This is required later in calculation of reduction in column loads in lower storeys due to reduction of live loads in upper floors.				
Wall: $P_{uw} = w_{uw1} \times L_w$	kN	9.75x3.05 = 29.74	2.60x5.30 = 13.78	2.6x3.05x2.2/8.4 +9.75x3.05=31.81
Self weight $P_{u.self}$	kN	17.34	17.34	17.34
Total roof load		139.72	253.47	178.85
$P_r = P_{us}+P_{uw}+P_{u.self}$	kN say	140	254	179
Load from above	kN	Nil	Nil	Nil
Total gross load= $P_{r3.gr}$	kN	140	254	179
Add 25% allowance at intermediate support to allow for continuity	kN	Nil	64	Nil
Total Design Load	kN	140	318	179
(b) Column between 3rd and 2nd Floor (3 - 2)				
Slab:Total:$P_{us} = q_{u2} \times A_{col}$	kN	11.625x9.15 = 106.37	11.625x21.96 = 255.29	11.625x12.81 = 148.92
*Live :$P_{fL} = q_{uL2} \times A_{col}$	kN	6.00x9.15 = say 55	6.00x21.96 = say 132	6.00x12.81 = say 77
Wall: $P_{uw'} = w_{uw2} \times L_w$	kN	25.125x3.05 = 76.63	11.25 x5.3 = 59.63	11.25x3.05x2.2/8.4 +25.125x3.05=85.62
Self weight $P_{u.self}$	kN	17.34	17.34	17.34
Total foor load = P_3 =	kN	200.34	332.26	251.88
$P_f = P_{us}+P_{uw}+P_{u.self}$	kN say	200	332	252
Load from above = $P_{r3.gr}$	kN	140	254	179
Total Gross Load $P_{32.gr} = P_r + P_f$	kN	340	586	431
Less:Reduction in LL on upper floors : Reduction factor r_3		0.1	0.1	0.1
Total shear due to LL on upper floors $P_{3L} = P_{rL} + P_{fL}$	kN	21+55 = 76	49+132 = 181	29+77 = 106
Actual load reduction=$r_3 \times P_{3L}$	kN	.1x76=say 8	.1x181=say 18	.1x106=say 11
Net design Load = $P_{32.net}$:	kN	332	568	420
Add 25% allowance at intermediate support for continuity	kN	---	142	---
Total Design Load = P_{32}	kN	332	710	420

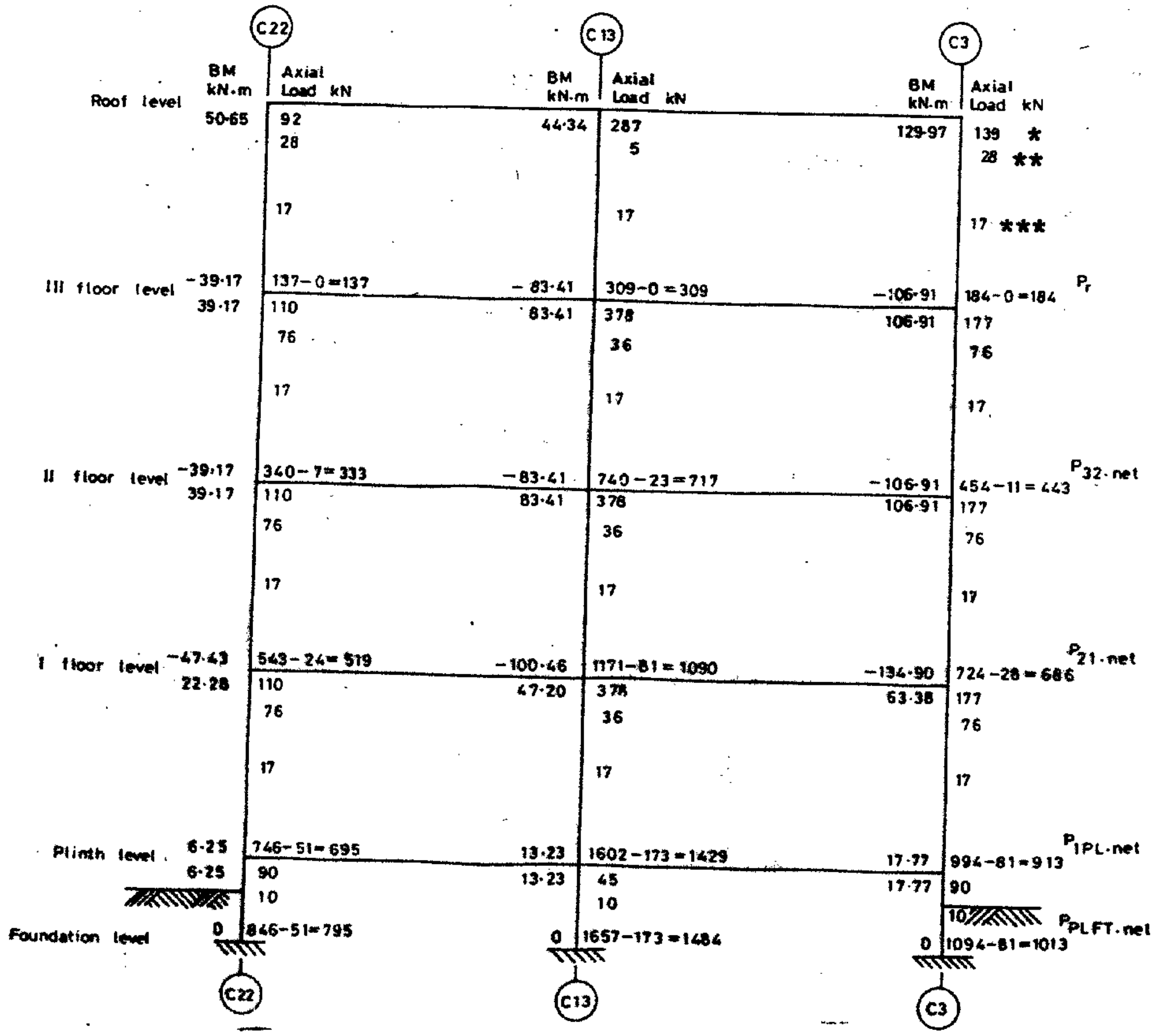
Assessment of Column Loads in Different Storeys : continued

Column Mark		C22	C13	C3
(c) Column between 2nd and 1st Floor (2 - 1)				
Total Floor load $P_2 = P_f$	kN	200	332	252
Load from above = $P_{32.gr}$	kN	340	586	431
Total Gross Load $P_{21.gr} = P_r + 2P_f$	kN	540	918	683
Less:Reduction in LL on upper floors : reduction factor r_2		0.2	0.2	0.2
Total shear due to LL on upper floors $P_{2L} = P_{3L} + P_{fL}$	kN	76+55=131	181+132=313	106+77=183
Actual load reduction=$r_2 x P_{2L}$	kN	.2x131=- 26	.2x313=-62	.2x183=-37
Net Design Load = $P_{21.net}$	kN	514	856	646
Add 25% allowance at intermediate support for continuity	kN	---	214	---
Total Design Load = P_{21}	kN	514	1070	646
(d) Co;umn between 1st Floor and Plinth (1 - P_L)				
Total Floor Load $P_1 = P_f$	kN	200	332	252
Load from above = $P_{21.gr}$	kN	540	918	683
Add weight of extra length of column for plinth height .5m	kN	=2.6 say 3	3	3
Total Gross Load $P_{1P.gr} = P_r + 3P_f$	kN	743	1253	938
Less:Reduction in LL on upper floors : reduction factor r_1		0.3	0.3	0.3
Total shear due to LL on upper floors $P_{1L} = P_{2L} + P_{fL}$	kN	131+55=186	313+132=445	183+77=260
Actual load reduction=$r_1 x P_{1L}$	kN	.3x186=-56	.3x445 =-133	.3x260=-78
Net Design Load = $P_{1p.net}$	kN	743-56=687	1253-133=1120	938-78=860
Add 25% allowance at intermediate support for continuity	kN	---	.25x1120=280	---
Total Design Load = P_{1p}	kN	687	1400	860
Column between Plinth and Footing ($P_{PL} - P_{FT}$)				
Wall : $P_{u3} = w_{uw3} \times L_f$	kN	28.875x3.05 = 88	13.5x3.05 = 41	13.5x3.05x2.2/8.4 +28.875x3.05=99
Self weight $P_{u.self}$	kN	10	10	10
Load from above = $P_{1P.gr}$	kN	743	1253	938
Total Gross Load $P_{P1.ft.gr}$	kN	841	1304	1047
Less:Reduction in LL on upper floors = that at plinth level	kN	-56	-133	-78
Net Design Load = $P_{P1.Ft.net}$	kN	785	1171	969
Add 25% allowance at intermediate support for continuity	kN	---	.25x1171=293	---
Total Design Load = $P_{PL.FT}$	kN	785	1464	969

Comparison of Column Loads (in kN) obtained by Exact and Approximate Methods :

Column Mark		C22		C13		C2	
Method of Calculation		Exact	Appr.	Exact	Appr.	Exact	Appr
Storey	R-3	137	140	309	318	184	179
	3-2	333	332	717	710	443	420
	2-1	519	514	1090	1070	686	646
	1-P1	695	687	1429	1400	913	860
	P1-Ft	795	785	1484	1464	1013	969

Remarks : The loads obtained by exact method are higher because of duplication of triangular load near internal longitudinal beam in Point load on span 5-6. If the triangular load is excluded from point load it would give bending moments and shear in transverse beam less than actual values and this would be on unsafe side in design of transverse beam. Column loads obtained by approximte method are more near to the real loads. In general, it is difficult to assess loads on internal column when the spans, loads and end conditions are unequal or different. It requires good judgement based on experience. The line diagram showing loads is shown in Fig. 7.9.1. Diagram of this type helps in better understanding of the load transfer. The figure also shows the bending moments in the columns as obtained by exact analysis.



NOTES

* 1) First value of load represents Maximum shear from transverse beam.

** 2) Second value of load represent Shear from longitudinal beams.

*** 3) Third value of load represent self weight of column.

4) Value of loads at floor level reduced to account for reduction in LL on upper floors.

FIG. 7·9·1 LOAD FLOW DIAGRAM

7.9.3 Moments in Columns

Moments in columns are obtained by exact method i.e, the stiffness solution of Substitute Frame - I using computer. The results are as under.

	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)	Rf-3	Top	120	50.65	292	44.34	167	129.97
		Bottom	137	-39.17	309	-83.41	184	-106.91
(b)	3-2	Top	316	39.17	700	83.41	427	106.91
		Bottom	333	-39.17	717	-83.41	440	-106.91
(c)	2-1	Top	502	39.17	1073	83.41	669	106.91
		Bottom	519	-47.43	1090	-100.46	686	-134.90
(d)	1-P1	Top	678	22.28	1412	47.20	896	63.38
		Bottom	695	6.25	1432	13.23	916	17.77
(e)	P1-Ft	Top	787	6.25	1476	13.23	1005	17.77
		Bottom	795	0	1484	0	1013	0

Column moments according to approximate method will be those obtained by column sub-frames of Method - III.

7.9.4 Determination of Effective Length and Slenderness of Column :

(A) Exact Method

I - Bending about x-axis : Plane of Transverse Frame

Since all columns and beams are of same cross section for all storeys, only the lengths being different, the Moments of Inertia of common sections and stiffnesses will be calculated first.

		Columns	Beam 4-5	Beam 5-6
Section bxD mm		230x600	230x600 Flanged	230x600 Flanged
Moment of Inertia mm^4 I = $bxd^3/12$		$230x600^3/12$ = $4140x10^6$	$2x230x600^3/12$ = 8280×10^6	$2x230x600^3/12$ = 8280×10^6
Length mm L	Above 1st floor	3350	6000	8400
	Below 1st floor	5350		
Stiffness K =I/L mm^3	Above 1st floor	1236×10^3	1380×10^3	986×10^3
	Below 1st floor	774×10^3		
Jt. K mm^3	Roof Level	1236×10^3	Joint-4 : 1380×10^3	
	3rd & 2nd Fl.level	2472×10^3	Joint-5 : 2366×10^3	
	1st Floor level	2010×10^3	Joint-6 : 986×10^3	

Calculation of Rotation Release Factors β : Bending @ X - Axis

Level		C22	C13	C3
Roof	ΣKc $x10^3$ mm^3	1236	1236	1236
	ΣKb $x10^3$ mm^3	1380	2366	986
	$\Sigma Kc+\Sigma Kb/2$ mm^3	1236+1380/2 = 1926	1236+2366/2 = 2419	1236+986/2 = 1729
	$\beta = \Sigma Kc/(\Sigma Kc+\Sigma Kb/2)$	1236/1926 = 0.64	1236/2419 0.52	1236/1729 0.71

Calculation of Rotation Release Factors β : Bending @ X - Axis : Continued

Level		C22	C13	C3
3rd Floor	$\Sigma Kc \times 10^3$ mm^3	2472	2472	2472
	$\Sigma Kb \times 10^3$ mm^3	1380	2366	986
	$\Sigma Kc+\Sigma Kb/2$ mm^3	2472+1380/2 = 3162	2472+2366/2 = 3655	2472+986/2 = 2965
	$\beta = \Sigma Kc/(\Sigma Kc+\Sigma Kb/2)$	2472/3162 = 0.78	2472/3655 = 0.68	2472/2965 = 0.83
2nd Floor	Same as 3rd Floor	0.78	0.68	0.83
1st Floor	$\Sigma Kc \times 10^3$ mm^3	2010	2010	2010
	$\Sigma Kb \times 10^3$ mm^3	1380	2366	986
	$\Sigma Kc+\Sigma Kb/2$ mm^3	2010+1380/2 = 2700	2010+2366/2 = 3193	2010+986/2 = 2503
	$\beta = \Sigma Kc/(\Sigma Kc+\Sigma Kb/2)$	2010/2700 = 0.74	2010/3193 0.63	2010/2503 0.80
Footing	Hinged End	1.00	1.00	1.00

Calculation of Effective Length and Slenderness : Bending @ X - Axis

Storey	End	Level		C22	C13	C3
Rf-3	Top	Roof	$\beta 1$	0.64	0.52	0.71
	Bottom	3rd Floor	$\beta 2$	0.78	0.68	0.83
			@ $L_{eff.x}/L$	0.80	0.73	0.835
			L_{cc} mm	3350	3350	3350
			D mm	600	600	600
			$L = L_{cc}-D$ mm	2750	2750	2750
			L_{eff} mm	2200	2007	2296
			L_{eff}/D	3.67	3.34	3.83
			Column Type	Short	Short	Short
3-2	Top	3rd Floor	$\beta 1$	0.78	0.68	0.83
	Bottom	2nd Floor	$\beta 2$	0.78	0.68	0.83
			@ $L_{eff.x}/L$	0.84	0.77	0.87
			L_{cc} mm	3350	3350	3350
			D mm	600	600	600
			$L = L_{cc}-D$ mm	2750	2750	2750
			$L_{eff.x}$ mm	2310	2117	2393
			$L_{eff.x}/D$	3.85	3.53	3.99
			Column Type	Short	Short	Short
2-1	Top	2nd Floor	$\beta 1$	0.78	0.68	0.83
	Bottom	1st Floor	$\beta 2$	0.74	0.63	0.80
			@ $L_{eff.x}/L$	0.83	0.76	0.86
			L_{cc} mm	3350	3350	3350
			D mm	600	600	600
			$L = L_{cc}-D$ mm	2750	2750	2750
			$L_{eff.x}$ mm	2283	2090	2365
			$L_{eff.x}/D$	3.80	3.48	3.94
			Column Type	Short	Short	Short

Calculation of Effective Length and Slenderness : Bending @ X - Axis : Continued

Storey	End	Level			C22	C13	C3
1-Ft	Top	1st Floor	β_1		0.74	0.63	0.83
	Bottom	Footing	β_2 (Hinged)		1.00	1.00	1.00
			@ $L_{eff.x}/L$		0.89	0.86	0.92
			L_{cc}		5350	5350	5350
			D	mm	600	600	600
			$L = L_{cc} - D$	mm	4750	4750	4750
			$L_{eff.x}$	mm	4228	4085	4370
@ See Fig.4.5.2			$L_{eff.x}/D$		7.05	6.81	7.28
			Column Type		Short	Short	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	mm	---	600x230	230x300	230x300
	$I = bD^3/12$		---	$600\text{x}230^3/12$	$230\text{x}300^3/12$	$230\text{x}300^3/12$
	$(\text{x}10^6)$	mm^4	---	= 608.4	= 517.5	= 517.5
	Length = L_{cc}	mm	---	3350	3050	3050
	$K = I/L_{cc}$ $(\text{x}10^3)mm^3$		---	181.6	169.7	169.7
	ΣK $(\text{x}10^3)$	mm^3	181.6		339.4	339.4
3 & 2 Floor	Section bxD	mm	600x230	600x230	230x450	230x300
	$I = bD^3/12(\text{x}10^6)mm^4$		608.4	608.4	1746.6	517.5
	Length = L_{cc}	mm	3350	3350	3050	3050
	$K = I/L_{cc}$ $(\text{x}10^3)mm^3$		181.6	181.6	572.6	169.7
	ΣK $(\text{x}10^3)$	mm^3	363.2		1145.2	339.4
1st Fl.	Section bxD	mm	600x230	600x230	230x450	230x300
	$I = bD^3/12(\text{x}10^6)mm^4$		608.4	608.4	1746.6	517.5
	Length = L_{cc}	mm	3350	3850	3050	3050
	$K = I/L_{cc}$ $(\text{x}10^3)mm^3$		181.6	158.0	572.6	169.7
	ΣK $(\text{x}10^3)$	mm^3	339.6		1145.2	339.4
Plinth	Section bxD	mm	600x230	600x230	230x450	230x300
	$I = bD^3/12(\text{x}10^6)mm^4$		608.4	608.4	1746.6	517.5
	Length = L_{cc}	mm	3850	1500	3050	3050
	$K = I/L_{cc}$ $(\text{x}10^3)mm^3$		158.0	405.6	572.6	169.7
	ΣK $(\text{x}10^3)$	mm^3	563.6		1145.2	339.4

Calculation of Rotation Release factors : Bending @ Y - Axis :

(All values of K are actual K x 10^{-3} mm^3)

Level	C22 and C3 ΣKc	ΣKb	(ΣKc+ΣKb/2)	β	:	C13 ΣKc	ΣKb	(ΣKc+ΣKb/2)	β
Roof	181.6	339.4	351.3	0.52	:	181.6	339.4	351.3	0.52
3rd Fl	363.2	1145.2	935.8	0.39	:	363.2	339.4	532.9	0.68
2nd Fl	363.2	1145.2	935.8	0.39	:	363.2	339.4	532.9	0.68
1st Fl	339.6	1145.2	912.2	0.37	:	339.6	339.4	509.3	0.67
Plinth	563.6	1145.2	1136.2	0.50	:	563.6	339.4	733.3	0.77
Footing				1.00	:				1.00

Calculation of Effective Length and Slenderness : Bending @ Y - Axis :

Width of all columns for full height = b = 230 mm.

Storey	End	Level		C22 & C3	C13
Rf-3	Top	Roof	β	0.52	0.52
	Bottom	3rd Floor	β	0.39	0.68
			$L_{eff.y}/L$	0.6675	0.735
			L_{cc} mm	3350	3350
			D of beam mm	300	300
			$L=L_{cc}-D$ mm	3050	3050
			$L_{eff.y}$ mm	2036	2462
			$L_{eff.y}/b$	< 12	< 12
			Column Type	Short	Short
3-2	Top	3rd Floor	β	0.39	0.68
	Bottom	2nd Floor	β	0.39	0.68
			$L_{eff.y}/L$	0.635	0.755
			L_{cc} mm	3350	3350
			D of beam mm	450	300
			$L=L_{cc}-D$ mm	2900	3050
			$L_{eff.y}$ mm	1841	2303
			$L_{eff.y}/b$	< 12	< 12
			Column Type	Short	Short
2-1	Top	2nd Floor	β	0.39	0.68
	Bottom	1st Floor	β	0.37	0.67
			$L_{eff.y}/L$	0.630	0.775
			L_{cc}	3350	3350
			D of beam mm	450	300
			$L=L_{cc}-D$ mm	2900	3050
			$L_{eff.y}$ mm	1827	2364
			$L_{eff.y}/b$	< 12	< 12
			Colum Type	Short	Short
1-P1	Top	1st Floor	β	0.37	0.67
	Bottom	Plinth	β	0.50	0.77
			$L_{eff.y}/L$	0.660	0.81
			L_{cc} mm	3850	3850
			D of beam mm	450	300
			$L=L_{cc}-D$ mm	3400	3550
			$L_{eff.y}$ mm	2240	2875
			$L_{eff.y}/b$	< 12	12.5
			Column Type	Short	Long
P1-Ft	Top	Plinth	β	0.50	0.77
	Bottom	Footing	β	1.00	1.00
			$L_{eff.y}/L$	0.82	0.91
			L_{cc} mm	1500	1500
			D of beam mm	450	300
			$L=L_{cc}-D$ mm	1050	1200
			$L_{eff.y}$ mm	861	1092
			$L_{eff.y}/b$	< 12	< 12
			Column Type	Short	Short

Determination of Effective Length and Slenderness of Column (Continued)

(B) Approximate Method

Bending @ X - Axis

For D = 600 mm of the column, maximum allowable $L_{eff.x}$ = 12x600 = 7200 mm for the column to be short. Assuming $L_{eff.x}$ = L, maximum unsupported length of column is only 5350 - 600 = 4750 mm between first floor and footing < 7200mm. Therefore, all columns can be considered to be short for bending @ x - axis. The calculations by exact method are normally not necessary and should be done only when the slenderness ratio by approximate method is found to be greater than 12 but nearer to 12.

Bending @ Y - Axis

For b=230mm, maximum allowable $L_{eff.y}$ = 12x230 = 2760mm. Taking $L_{eff.y}$ = L, maximum L for column to be short is 2760 mm. Since unsupported length for practically all storeys are greater than 2760 mm, the columns will be slender for bending @ y - axis. The calculations are shown below.

Column Mark		C22 & C3 (Outer)					:	C13 (Inner)				
Storey	Lcc mm	Dbeam mm	L mm	$L_{eff.y}$ mm	$L_{eff.y}$/b	Type	:	Dbeam mm	L mm	$L_{eff.y}$ mm	$L_{eff.y}$/b	Type
Rf-3	3350	300	3050	3050	13.26	Long	:	300	3050	3050	13.26	Long
3 -2	3350	450	2900	2900	12.61	Long	:	300	3050	3050	13.26	Long
2 -1	3350	450	2900	2900	12.61	Long	:	300	3050	3050	13.26	Long
1-P1	3850	450	3400	3400	14.78	Long	:	300	3550	3550	15.44	Long
P1-Ft	1500	450	1050	1050	< 12	Short	:	300	1500	1200	< 12	short

Thus, according approximate method, all columns, except those below plinth, prove to be long. If this observation is followed and calculations for determination of exact value of L_{eff} are avoided, the design will not only be on conservative side but it will also involve calculations for design of long columns.

7.9.5 Grouping of Columns

Since loads and moments in the three columns are totaly 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 along with each of the above columns to form a group.

7.9.6 Design of Column Section

Since exact values of P_u and M_u are known for all storeys for all columns, the column sections wil be designed using exact method using charts and tables. The tables 6.12 to 6.15 of Author's Handbook give axial load and bending resistances of column sections 230 mm wide and various depths for areas of bars from minimum to maximum. The values of P_u-M_u from Tables may be referred for the given depth of section starting from the rightmost column of P_u-M_u on the first line correspon- ding 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 calcula-tions required in use of charts. However, they are useful for a fixed width of 230 mm. Charts are useful for any width of column. It is advisable to have curves plotted for P_u-M_u for standard sections normally used in building design to avoid calculations.

All the columns are subjected to axial loads and uniaxial bending. They will be designed to resist P_u and M_u for bending @ x- axis and the section will be checked for P_u and $M_{uy.min}$ corresponding to minimum eccentricity $e_{y.min}$ for bending @ y - axis.

(a) Column C22

$e_{x.min}$ = L/500 + D/30 or 20 mm whichever is greater. For design, the load P_u at the base of the column is considered in combination with maximum of the two moments at top and bottom of column on safe side.

Design by use of Tables: Bending @ x - axis

Storey	L mm	$e_{x.min}$ mm	P_u kN	M_{ux} kN.m	$M_{ux.min}$ kN.m	M_{uD} kN.m	Section b(mm) D(mm)	Main Steel N - ϕ(mm)	Ties ϕ(mm)-s(mm)
Rf-3	2750	25.5	137	50.65	3.49	50.65	230 x 600	6 - #16*	ϕ6 - 230
3 -2	2750	25.5	333	39.17	8.49	39.17	230 x 600	6 - #16*	ϕ6 - 230
2- 1	2750	25.5	519	47.43	13.23	47.43	230 x 600	6 - #16*	ϕ6 - 230
1-P1	:4750	29.5	695	22.28	20.50	22.28	230 x 600	6 - #16*	ϕ6 - 230
P1-Ft:	-	-	795	6.25	23.45	23.45	230 x 600	6 - #16*	ϕ6 - 230

* Using Table 6.14, p.232 of Hand book, partly reproduced as Table E-2 in this book.

For top storey,10 bars of #12 with lesser area of steel are also adequate. However, that combination is not considered because in lower columns 6 Nos. of #16 are required, and it is not advisable to use greater number of bars (though of smaller diameter) in upper storey and lesser number of bars in lower storeys.

Design by use of Charts : Bending @ x - axis :

d' = 50 mm, D = 600 mm, d'/D =0.083 say 0.1, b = 230 mm.

$f_{ck}bD$ = 15x230x600/1000 = 2070 kN, $f_{ck}bD^2$ = 2070x600/1000 = 1242 kN.m.

$A_{sc.min^1}$ = .8x230x600/100 = 1104 mm² i.e. 6 of #16 = 1206 mm².

Storey	$P_u/(f_{ck}bD)$	$M_u/(f_{ck}bD^2)$	Chart	Req.p/fck	A_{sc}	$A_{sc.min}$	N - ϕ(mm)	ϕ -s(mm)
Rf- 3	0.066	0.041	E-3	0.01	207 mm²	1104	6 - #16	ϕ6 - 230
3 - 2	0.161	0.032	E-3	minimum	mm²	1104	6 - #16	ϕ6 - 230
2 - 1	0.251	0.038	E-3		mm²	1104	6 - #16	ϕ6 - 230
1 -P1	0.337	0.018	E-3	Steel	mm²	1104	6 - #16	ϕ6 - 230
P1-Ft	0.384	0.019	E-3		mm²	1104	6 - #16	ϕ6 - 230

Design by use of Tables : Bending @ y - axis :

Maximum value of minimum eccentricity is for storey below first floor for which centre to centre length Lcc is maximum equal to 3850 mm. For this,

$e_{y.min}$ = 3850/500 + 230/30 but not less than 20 mm. ∴ $e_{y.min^1}$ = 20 mm.

Therefore, Table 6.11 of Handbook, partly reproduced as Table E-1, in this book can be used directly. From this table, minimum steel for a column section 230x530 mm is 10 Nos. of 12mm dia. for P_u = 934 kN. Since maximum load on column is 795 kN, 6 of #16 would be more than enough for a column section 230x600mm.

(b) Column C13

The design of remaining two columns is done by use of tables.

Storey	L mm	$e_{x.min}$ mm	P_u kN	M_{ux} kN.m	$M_{ux.min}$ kN.m	M_{uD} kN.m	Section b(mm)xD(mm)	Main Steel N − ø(mm)	Ties ø(mm)-s(mm)
Rf-3	2750	25.5	309	83.41	7.88	83.41	230 x 600	6 − #16	ø6 − 230
3 -2	2750	25.5	717	83.41	18.28	83.41	230 x 600	6 − #16	ø6 − 230
2 -1	2750	25.5	1090	100.46	27.80	100.46	230 x 600	6 − #20	ø6 − 230
1-P1	:4750	29.5	1432	47.70	42.24	47.70	230 x 600	6 − #22	ø6 − 230
P1-Ft:	−	−	1484	13.23	43.78	43.78	230 x 600	8 − #22	ø6 − 230

(c) Column C3

Storey	L mm	$e_{x.min}$ mm	P_u kN	M_{ux} kN.m	$M_{ux.min}$ kN.m	M_{uD} kN.m	Section b(mm)xD(mm)	Main Steel N − ø(mm)	Ties ø(mm)-s(mm)
Rf-3	2750	25.5	184	129.97	4.69	129.97	230 x 600	6 − #16	ø6 − 230
3 -2	2750	25.5	440	106.91	11.30	106.91	230 x 600	6 − #16	ø6 − 230
2 -1	2750	25.5	686	134.90	17.49	134.90	230 x 600	8 − #16	ø6 − 230
1-P1	:4750	29.5	916	63.38	27.02	63.38	230 x 600	8 − #16*	ø6 − 230
P1-Ft:	−	−	1013	17.77	29.88	29.88	230 x 600	8 − #16*	ø6 − 230

*** Note :** Though for Lower columns steel required is less because of smaller moment, it is, normally, not reduced in practice. Maximum area of steel is required for column in storey 2-1 since greater moment is transferred to upper column at floor 1 because of greater stiffness of upper column compared to stiffness of lower column. This is because length of column in storey 1-P1 is greater than that of storey 2-1. This shows that provision of a beam at plinth level is helpful in column design if cost of plinth beam is less than the cost of masonry wall below plinth.

7.10 DESIGN OF FOOTINGS

This has been done in a tabular form vide Sect.6.5.5. For description of steps, refer to Sect.6.5.4.

Step No.			Results of Calculations		
I - Data					
1.	Column Mark		C22	C13	C3
2.	q	kN/m²	250	250	250
3.	P_u	kN	795	1484	1013
4.(a)	b	mm	230	230	230
(b)	D	mm	600	600	600
5.(a)	fck	N/mm²	15	15	15
(b)	f_y	N/mm²	415	415	415
(c)	$R_{u.max}$	N/mm²	2.07	2.07	2.07
(d)	$p_{y.min}$	%	.205	.205	.205

Step No.	Column Mark		C22	C13	C3
II - Size of Footing for Bearing					
6.	Req.A_f	m^2	2.332	4.353	2.971
7.(a)	Req.L_f	mm	1724	2280	1919
(b)	Req.B_f	mm	1353	1910	1549
(c)	Pro.L_f	mm	1725	2300	1925
(d)	Pro.B_f	mm	1375	1925	1550
(e)	Pro.A_f	m^2	2.372	4.427	2.984
8.	w_u	kN/m^2	335.18	335.18	339.51
9.	X_i	mm	562.5	850.0	662.5
10.	Y_i	mm	572.5	847.5	660.0
III - Depth of Footing for Two-way bending					
11.(a)	M_{ux}	kN.m	72.91	233.08	115.48
(b)	M_{uy}	kN.m	94.75	276.86	142.34
12.(a)	b'_f	mm	330	330	330
(b)	L'_f	mm	700	700	700
13.(a)	Req.dx	mm	326.7	584	411
(b)	Req.dy	mm	255.7	437	314
14.(a)	ø	mm	8	12	10
(b)	d'x	mm	54	56	55
(c)	d'y	mm	62	68	65
(d)	Req.D_f	mm	381	640	466
15.(a)	Prov.D_f	mm	400	650	475
(b)	Prov.dx	mm	346	594	420
(c)	Prov.dy	mm	338	582	410
(d)	Prov.$D_{f.min}$	mm	150	150	150
(e)	Prov.$d_{f.minx}$	mm	96	94	95
(f)	Prov.$d_{f.miny}$	mm	88	82	85
IV - Check for Two-way Shear					
16.(a)	L_2	mm	938	1182	1010
(b)	B_2	mm	568	812	640
(c)	d_2	mm	281.1	430.9	327.4
(d)	A_2	mm	846559	1718441	1080480
17.(a)	T'_{uc}	N/mm^2	.968	.968	.968
(b)	B_s		.383	.383	.383
(c)	k_s		.883	.883	.883
(d)	T_{uc2}	N/mm^2	.855	.855	.855
18.	V_{uc2}	kN	724.05	1469.70	924.12
19.	V_{uD2}	kN	616.42	1162.30	793.54
20.	D_{fs}	mm	400 Revised Cal. 425	650	475 Revised Cal. 500

Design of Footings continued

Step No.	Column Mark		C22		C13	C3	
V - Area of Steel for Two-way Bending including check for Development Length							
21.(a)	Req.A_{stx}	mm^2	704	636	1341	937	858
(b)	Req.A_{sty}	mm^2	864	792	1464	1073	997
22.(a)	Req.$A_{stx.min}$	mm^2	234	251	402	284	301
(b)	Req.$A_{sty.min}$	mm^2	485	521	835	588	624
23.(a)	Max Bar Dia.	mm	8	8	12	10	10
(b)	Req. Ld	mm	456	456	684	570	570
(c)	Ava. Ld	mm	562.5	562.5	847.5	660	660
(d)	Mod.A_{stx}	mm^2	704	636	1341	937	858
(e)	Mod.A_{sty}	mm^2	961	881	1593	1188	1104
24.(a)	a_{st}	mm^2	50.24	50.24	113	78.5	78.5
(b)	N_x		15	13	12	12	11
(c)	S_x	mm	94	110	170	136	150
(d)	Prov.A_{stx}	mm^2	753	653	1356	942	863
25.(a)	N_y		20	18	15	16	15
(b)	S_y	mm	88	98	160	125	133
(c)	Prov.A_{sty}	mm^2	1004	904	1695	1256	1177
VI - Check for One-way Shear for Bending about X-axis							
26.(a)	L_1	mm	1276	1326	1764	1420	1470
(b)	dy1	mm	200.2	198.3	248.5	218.2	214.1
(c)	Ay1	mm^2	320157	320006	526842	386396	382770
27.(a)	Prov.Ny		20	18	15	16	15
(b)	Prov.Asty	mm^2	1004	904	1695	1256	1177
(c)	P_{ty}	%	.314	.283	.322	.325	.308
28.	T_{ucy1}	N/mm^2	.386	.370	.390	.391	.382
29.	V_{ucy1}	kN	123.49	118.2	205.30	151.20	146.4
30.	V_{uDy1}	kN	135.58	121.1	204.70	163.39	147.1
	Remarks		*Unsafe	Unsafe#	Safe	Unsafe	Unsafe#

*Increase the depth as $V_{uDy1} > 1.05V_{ucy1}$. Go to back to Step 20.
#Increase only the steel as $V_{uDy1} < 1.05V_{ucy1}$. The steps for revised calculations for Columns C22 and C3 are given below.

Col	Ny	Asty mm^2	P_{ty} %	T_{ucy1} N/mm^2	V_{ucy1} kN	V_{uDy1} kN	Remarks:Ny	A_{sty} mm^2	P_{ty} %	T_{uy1} N/mm^2	V_{ucy1} kN	V_{uDy1} kN	Remarks
C22	19	954	.298	.378	120.90	121.13	Unsafe:20	1004	.314	.386	123.4	121.1	Safe
C3	16	1256	.328	.393	150.33	147.05	Safe						

Design of Footings continued

Step No.	Column Mark		C22	C13	C3

VII - Check for One-way Shear for Bending about X-axis

Step No.			C22	C13	C3
31.(a)	B1	mm	972	1418	1120
(b)	dx1	mm	198.8	254	219.3
(c)	Ax1	mm²	252584	448390	313171
32.(a)	Prov.Nx		13	12	11
(b)	Prov.A_{stx}	mm²	653	1356	863
(c)	p_{tx}	%	.258	.302	.276
33.	τ_{ucx1}	N/mm²	.356	.380	.366
34.	V_{ucx1}	kN	89.97	170.4	114.55
35.	V_{uDx1}	kN	88.26	165.2	114.45
36.	Remarks		Safe	Safe	Safe

Results :

Col.	P_u kN	b mm	D mm	q kN/m²	Grades of Con	Steel	L_f mm	B_f mm	D_f mm	$D_{f,min}$ mm	Nx ø mm	Sx mm	Ny φ mm	Sy mm
C22	795	230	600	250	M15	Fe415	1725	1375	425	150	13-#8	110	20-#8	88
C13	1484	230	600	250	M15	Fe415	2300	1925	650	150	12-#12	170	15-#12	160
C3	1013	230	600	250	M15	Fe415	1925	1550	500	150	11=#10	150	16-#10	125

Project – III : Design of Multi-storeyed Residential Building

8.1 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, an office 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 mutistoreyed 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 a 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.8.1, 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 helps in saving time and labour. The design aids used in this project have been given in Appendices for the benefit of users.

8.2 DATA

Type : Multi-storeyed Residential Building (G+3)
Building Plan : As shown in Fig.8.1
Floor to floor height = 3000 mm
Height of Plinth = 450 mm above ground level.
Depth of Foundation = 1000 mm below ground level.
Bearing Capacity of Soil = 200 kN/m² (20t/m²)
External Walls : 150 mm (6") thick solid concrete block.
Internal Walls : 125 mm (4.5") thick brick masonry.
Assumed Imposed Loads :
Roof : Roof Finish = 1.5 kN/m² (150 kg/m²), Live Load = 1.5 kN/m² (150 kg/m²)
Imposed Load = 1.5 + 1.5 = 3.0 kN/m² (300 kg/m²)
Floor: Floor Finish = 1.0 kN/m² (100 kg/m²), Live Load = 2.0 kN/m² (200 kg/m²)
Imposed LOad = 1.0 + 2.0 = 3.0 kN/m² (300 kg/m²)
(for imposed loads Refer to Note in Sect.2.2.1 in Table 2.2 of Chapter 2)

Assumed Materials : Concrete M15, Steel Main – Fe415, Secondary – Fe250
Design Basis : Limit State Method based on IS:456-1978. S.I.Units.
The axonometric view of the building is shown on the cover page.

8.2.1 Structural Planning

The work of the designer starts with planning of structural members in the given architectural plan. 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. 8.2. 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 midlanding slab overhanging beyond beam B19 and part of the floor beam B15(i.e.B15a) cantilever.This reduces the midspan 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.8.4.

8.2.2 Numbering & 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 rightwards 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.8.2.

8.3 ULTIMATE LOADS

As per Table A-1 of Appendix.

1) Roof : Assumed D = 120 mm, q_u = 9 kN/m² (900 kg/m²)
2) Floor : Assumed D = 120 mm, q_u = 9 kN/m² (900 kg/m²)
3) Bath-W.C. : Assumed D = 100 mm, q_u = 10.5kN/m²(1050 kg/m²)
4) Loft : Assumed D = 100 mm, q_u = 8 kN/m² (800 kg/m²)

5) Balconies : Cantilever - Assumed D = 150 mm, q_u = 12 kN/m² (1200 kg/m²)
: Simply supported- Assumed D = 100 mm, q_u = 10 kN/m² (1000 kg/m²)
6) Stairs : Assumed D = 130 mm, q_u = 15 kN/m² (1500 kg/m²)
7) R.C.C.Grill: Assuming 80mm thick(50% opening) Ultimate load = 1.5 kN/m/m
For 1.4m height W_{ug}=1.5x1.4=2 kN/m ,For 2.7m W_{ug}=1.5x2.7=4 kN/m

8) Balcony Parapet: 80 mm thick RCC.,1 m. high, W_{uw} = 3.00 kN/m (300 kg/m)
9) Weather Sheds : Assuming 100mm, 600mm wide, W_{uc} = 2.25 kN/m (225 kg/m)

10)Beam Rib : Assuming minimum depth of slab D_f = 100 mm,

	Factored Self weight w_{us} in kN/m				
Depth of beam (D) in mm	300	375	450	600	*Depth of rib
* Flanged beam b_w = 150 mm	1.1	1.5	2.0	2.8	$=D_r=(D-D_f)$
Rectangular beam b = 225 mm	2.6	3.2	3.8	5.0	

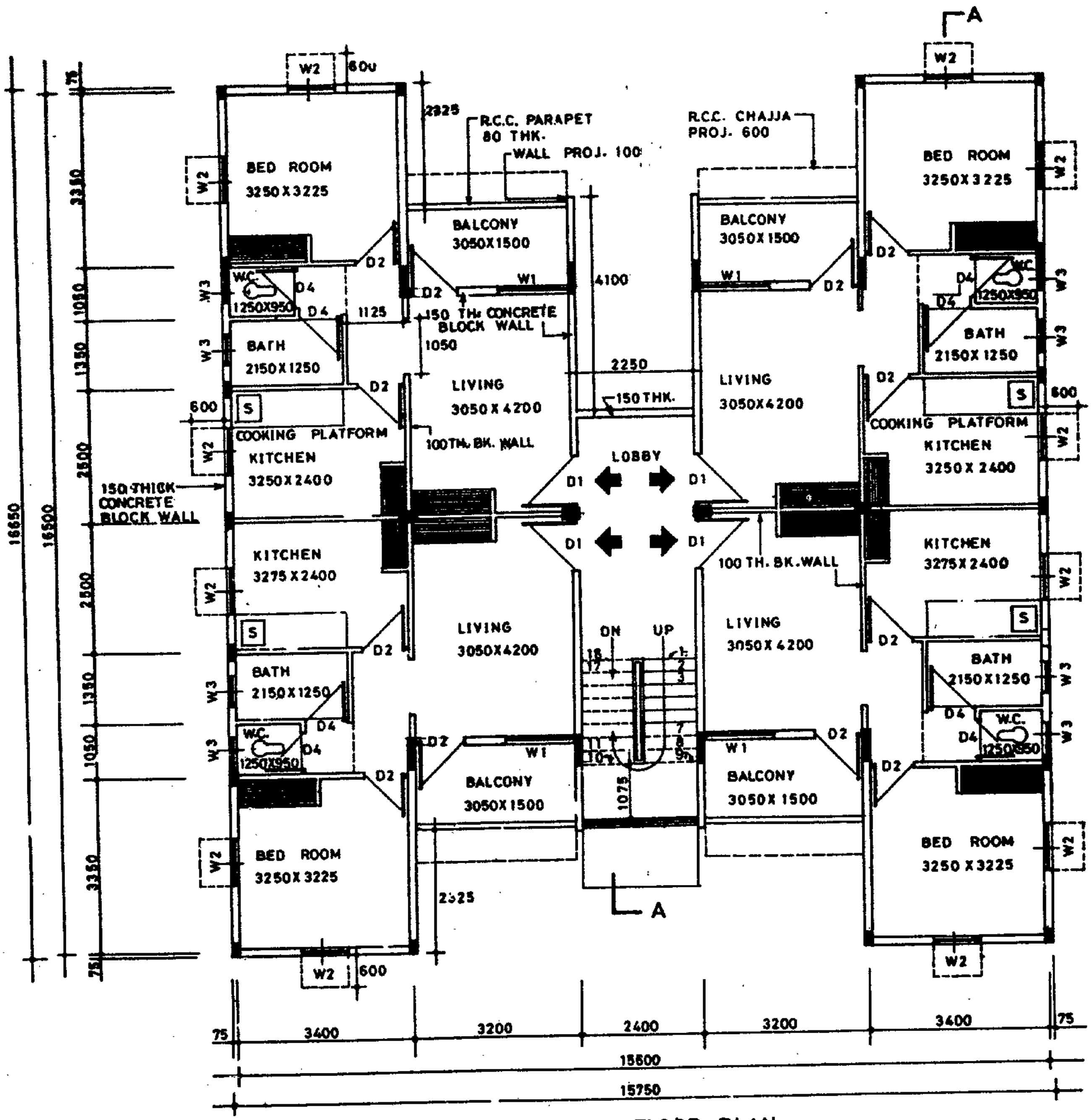
11) Walls : Values of Ultimate Loads.

	External- Concrete solid blocks 150mm (175mm with plaster)	Internal- solid Brick 125mm (150mm with plaster)
1.00 m high	6.75 kN/m	4.50 kN/m
2.00 m high	13.50 Kn/m	9.00 kN/m
2.70 m high	18.00 kN/m	12.00 kN/m
3.00 m high	20.00 kN/m	13.50 kN/m

0.45 m high, 225 mm thick = 3.4 kN/m (between G.L. and Plinth)

SCHEDULE OF DOORS

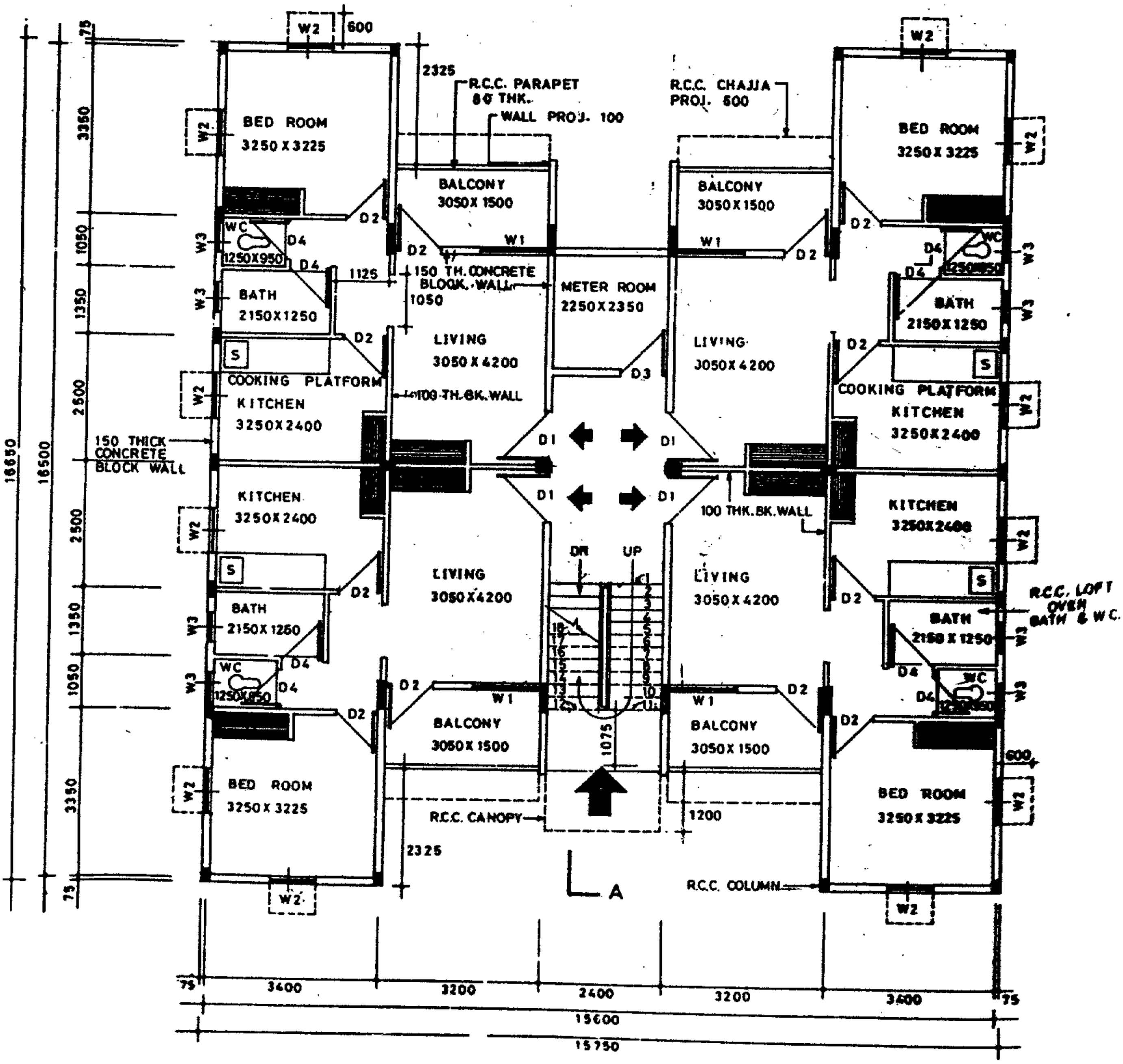
TYPE	SIZE	DESCRIPTION
DOORS		
D1	1000 X 2400	T.W. FRAME COMMERCIAL BLOCK BOARD WITH GLAZED FAN LIGHT
D2	900 X 2100	T.W. FRAME COMMERCIAL BLOCK BOARD
D3	900 X 2000	T.W. FRAME COMMERCIAL BLOCK BOARD
D4	800 X 1900	T.W. PANELLED DOOR



TYPICAL FLOOR PLAN

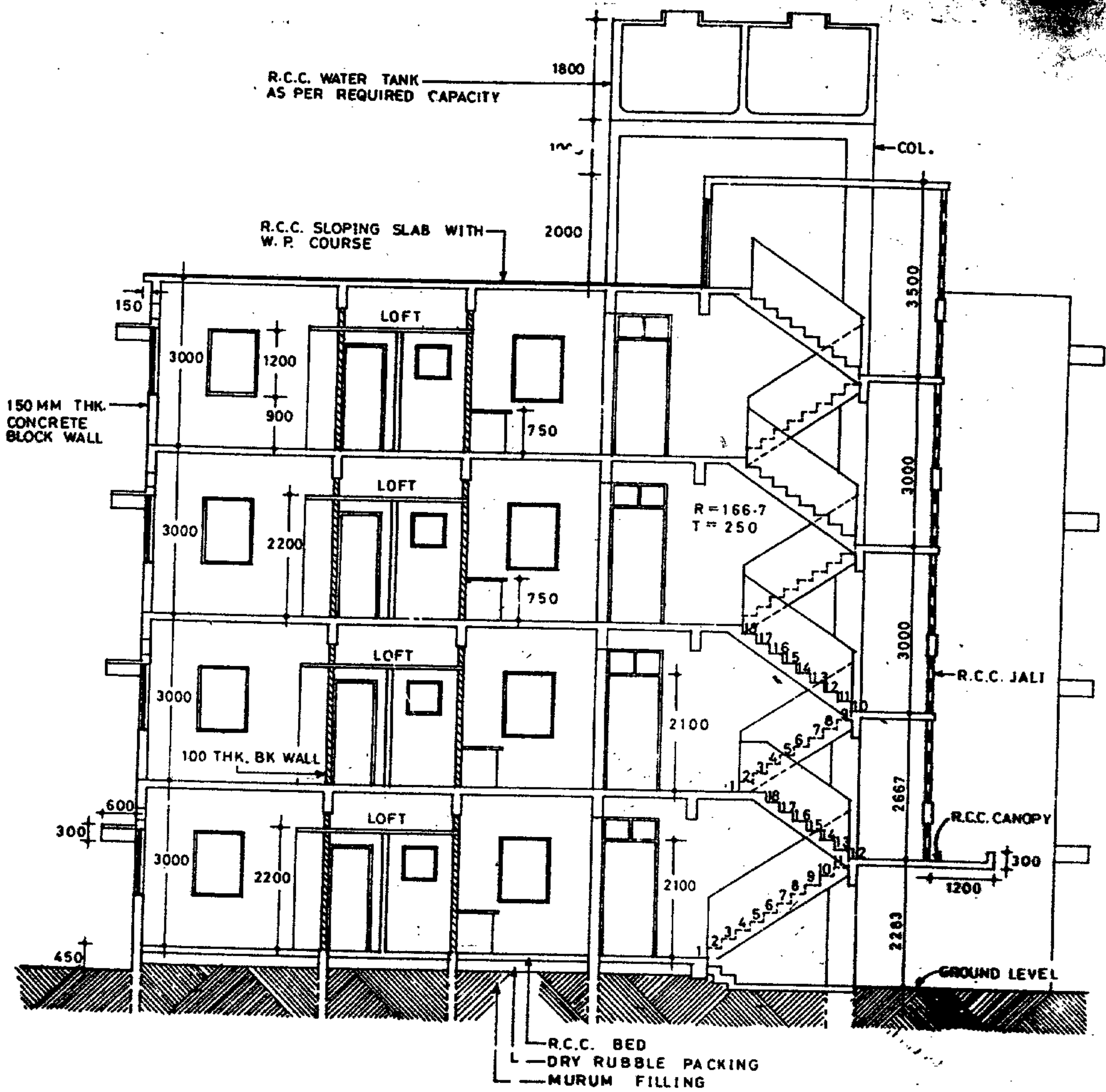
SCHEDULE OF WINDOWS

TYPE	SIZE	DESCRIPTION
WINDOWS		
W1	1350X1200	T.W. GLAZED WINDOW THREE SHUTTERS
W2	900X1200	T.W. GLAZED WINDOW DOUBLE SHUTTERS
W3	650X 650	T.W. FRAME WITH LOUVERS



GROUND FLOOR PLAN

FIG. 8·1



SECTION A-A

	INSTITUTE OF STRUCTURAL ENGINEERING 36 Parvati, Pune - 411 009.
Job. No.	PROJECT — 3 RESIDENTIAL BUILDING

Drg. No.	Date / /	Drawn by	
	Scale as shown	Checked by	Structural Engineer

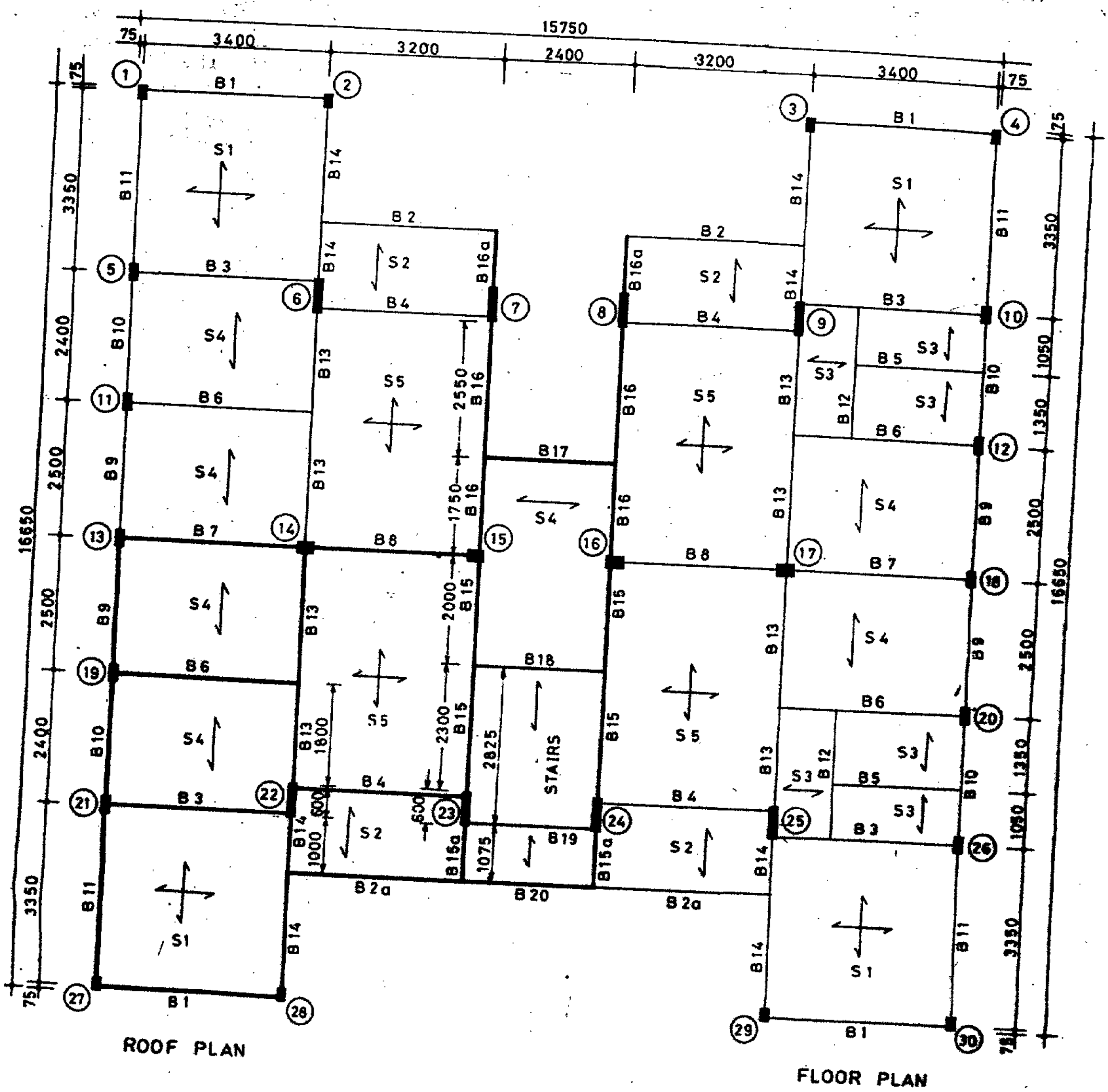


Fig. 8.2 Structural Plan

8.4 DESIGN OF SLABS

8.4.1 Roof Slab

The typical marking plan shown in Fig.8.2 is a sectional plan. 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 slab supported on B3 and B6 is a single slab having span 2.4 m instead of 1.35m. Since end conditions (both ends continuous) and span of this slab (2.4 m) are approximately the same as that of S4 supported on B6 and and B7, this slab is also categorised as S4. Thus, category of slab S3 is absent at the roof level.The cap slab over the stairs is provided above the door level. The span of this slab will be 2.4 m and it will be supported by beams B15a and B15 over a length of 2.3m. The water tank of size 4.3m x2.4m x2m having capacity of 20,000 litres will be provided over the columns C15,C16,C23,C24. The bottom of the tank shall be 1m above the staircase cap slab so that any unforseen water leakage problem can be attended independently without causing inconvenience to the occupants.

Spanning of Slabs :

It has been mentioned in Sect.1.3(c) , 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 eventhough 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 3d) governs. However, as mentioned earlier minimum steel (0.12% of bD for HYSD or 0.15% of bD for mild steel)should be provided across the short edge support to avoid cracking. Therefore, slabs S2 and S4 having spans less than 3m 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 3m 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 beams B1 and B3 requires D = 120mm with #8 at 150mm (See Table C-4,Type B) for span of 3.35m. If the same slab is made to span across B11 and B14, the slab has a span of 3.4m and is discontinuous at both ends, and it requires D = 150 mm with #8 at 150 mm (See Table C-4, Type A). On the contrary, if it is designed as two-way,it requires D = 110 mm with #8 at 250 mm along short span and #8 at 270 mm along long span, thus proving economical. Slab S5, if designed as one-way across beams 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 B16 because slab S4 beyond it eventhough 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=140 mm with #8 at 160 mm for L =3.2 m (See Table C-4,Type A). On the contrary,if S5 is designed as two way,it gets continuity at both supports B4 nd B8 in short direction, requiring D = 100 mm with #8 at 240,thus proves to be conomical. 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 one-way slab category, at roof level, span of central corridor slab between beams B15 is 2.4m,and that of slab spanning across B6 and B7 is 2.5m.Since, he difference in spans is very small (only 4%), all slabs are grouped together nd designated as S4 and are designed for span,L = 2.5m.

In two-way slab category, since S1 and S5 have different aspect ratios y/L_x) and boundary conditions,they are not grouped together but designated and signed separately.

Category - 1 : One Way Slabs

Step No.	Slab Mark.		S4	S2	Reference	Note No.
1.	Span	L m.	2.5	1.6		
2.	End Condition (EC) Number		3 (2)*	2		
3(a)	Assumed Total Working Load q kN/m^2		6	6.5	Table A-1	
(b)	Total ultimate Load qu =1.5q kN/m^2		9	10		
4.	Ultimate moment Mu kN.m.		4.69	2.56		
5.	Required Depth	D mm	100	100	Table C-1	
6(a)	Short Span Steel: Dia.(mm)-spacing(mm)		#8-240	#8-240	Table C-2	2
(b)	Long Span Steel: Dia.(mm)-Spacing(mm)		∅6-180	∅6-180	Table C-3	
7(a)	End Shear in kN :Long edge -					
	Penultimate support, $SF=0.6q_uL$ for EC=2		* 13.50	9.60		
	Continuous support, $SF=0.5q_uL$ for EC=3		11.25	-	Sect.3.4.1	
	Simple support $SF=0.45q_uL$ for EC=2		* 10.12	7.20		
(b)	Short edge- Discontinuous end q_u L/6		3.75	2.67		3
8.	Check for Development length at Discontinuous end:					
(a)	At Continuous End :					
	Required $L_d = 57\phi = 57x8$	mm	456	-		
	Available L_d = L/4 = 2500/4	mm	625	-		
(b)	At Simply supported End : $V_{uD}=V$	kN	-	7.20		
	At midspan, M_{ur} for # 8 @ 240	kN.m.	-	5.69	Eq. 4.1-4b	
	Assuming alternate bars bent up at support, M1 available at bottom = 5.69/2	kN.m.	-	2.84		
	Required $L_{ex}=L_d-1.3M1/V$ =456-(1.3x2.84x1000/7.2)	mm	-	56		
	However, minimum end anchorage=$L_d/3$	mm	-	152		
	Assuming 90^o bend, Anchorage available from inner face $=(b_s-25-5\phi+8\phi)$ = 150-25+3x8	mm	-	149		
∴	Extension required beyond bend	mm	-	3	O.K.	
9.	Check for Shear :					
(a)	At Continuous End :					
	Maximum Design shear = V_{uD}	kN	11.25	9.60		
	A_{st1} = #8 @ 240	mm^2	209	209		
	Effective depth = 100 -(15+8/2)	mm	81	81		
	P_t = 209x100/(1000x81)	%	0.26	0.26		
	T_{uc} =	N/mm^2	.351	.351	Eq.4.2-3	
	Multiplying factor for slab Depth=100 mm		1.3	1.3	Sect.4.2.4	
	V_{uc} =1.3xT_{uc}xbxd = 1.3x.351x81>V_{uD}	kN	36.96	36.96		4
(b)	At Simply Supported End :					
	Design shear	kN	-	7.20		
	A_{st1} 50 % of $A_{st.max}$ = 0.5x209	mm^2	-	104		
	P_t = 104x100/(1000x81)	%	-	0.13		
	T_{uc} =	N/mm^2	-	.235	Eq.4.2-3	
	V_{uc} = 1.3x.235x81x1000/1000 > V_{uD}	kN	-	19.05		4

Explanatory Note 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 S4, EC = 3 for roof slab, EC = 2 for floor slab.
2.Spacing of 240mm is governed by maximum spacing of 3d (=3 x 80 = 240 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	Expl. Note No.
1(a)	Short Span L_x metres	3.35	3.20		
(b)	Long Span L_y "	3.40	4.20		
(c)	AspectRatio $L_y/L_x = \beta$	1.015	1.344		
2.	Boundary Case No.	7	6	Table B-2	
3(a)	Allowable L/D ratio	32	32	Cl.23.1 Note-2	
(b)	Depth D for serviceability mm	110	100		
4.	Assumed Ultimate load q_u kN/m²	9	9		
	$q_u L_x^2$ KN.m/m	101	92.16		
5.	B.M.coefficients α_x (maximum)	0.058	0.060	Table B-2	1
	α_y (maximum)	0.043	0.045	Table B-2	1
	$m_x = \alpha_x q_u L_x^2$ kN.m/m	5.86	5.53		
	$m_y = \alpha_y q_u L_x^2$ kN.m/m	4.34	4.15		
6.	Main Steel: Short Span: Dia.(mm)-s(mm)	#8-250	#8-240	Table C-2	2
	Long Span: Dia.(mm)-s(mm)	#8-250	#8-220	Table C-2	2,3
7(a)	Equivalent ultimate UDL to be transferred from slab to beam for Bending Momen				
	Long Edge: Loads in kN.				
	End support(EC=2) $w_{ueqb} = 0.45\, q_u L_x \alpha_1$	9.18	-		
	Penultimate support, (EC=2) $w_{ueqb} = 0.60\, q_u L_x \alpha_1$	12.24	-		
	Continuous/Dis-continuous support (EC = 1 or 3), $w_{ueqb} = 0.50\, q_u L_x \alpha_1$	-	11.77		
	where, $\alpha_1 = [1-1/(3\beta^2)]$				
	Short Edge:				
	Continuous/Dis-continuous support (EC = 1 or 3), $w_{ueqb} = 0.50\, q_u L_x \alpha_2$	10.05	9.60		
	where, $\alpha_2 = 2/3$				
7(b)	Equivalent ultimate UDL to be transferred from slab to beam for Shear Force :				
	Long Edge:				
	End support(EC=2), $w_{ueqs} = 0.45\, q_u L_x \alpha_3$	6.88	-		
	Penultimate support, (EC = 2) $w_{ueqs} = 0.60\, q_u L_x \alpha_3$	9.18	-		
	Continuous/Dis-Continuous support (EC = 1 or 3), $w_{ueqs} = 0.50\, q_u L_x \alpha_3$	-	9.07		
	where, $\alpha_3 = [1-1/(2\beta)]$				
	Short Edge :				
	Continuous/Dis-continuous support, (EC = 1 or 3), $w_{ueqs} = 0.50\, q_u L_x \alpha_4$	7.54	7.20		
	where, $\alpha_4 = 1/2$				
8.	Torsion steel :		See Explanatory Note No.4.		

Explanatory Notes :

1) Out of the two coefficients, one at midspan and the other at support, the greater one is considered, and the same steel is provided at both locations for convenience of bending and detailing.

2) Spacing is governed by limiting spacing of 3d.
For S1, for long span, di = 110-15-8-8/2 = 83mm .:.3d = 3x83 = 249mm say 250mm.
For S5, for long span, di = 100-15-8-8/2 = 73mm .:.3d = 3x73 = 219mm say 220mm.

3) In practice, maximum spacing of main bars is restricted to 200mm, in which case both long span and short span steels for all slabs having Fe415 will be #8 at 200 mm.

Explanatory Notes (continued) :

4) 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° by taking them at top . 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	110	# 8 - 250	#8 - 250	Two-way
S2	100	# 8 - 240	∅6 - 180	One-way
S4	100	# 8 - 240	∅6 - 180	One-way
S5	100	# 8 - 240	#8 - 220	Two-way

End shears from Roof slab have been shown in Fig.8.4

8.4.2 Floor Slabs

Since assumed total loads on roof and floor slabs are equal,the floor slabs will have same design details as those of roof slab. No seperate design of floor slabs is, therefore, necessary except in case of slab across beams B3,B5,B6,B12 and B13 since beams B5 and B12 are provided at floor level.

If the total roof load is very less compared to floor load, seperate design may be done for roof slabs. However, for small difference of loads,it is helpful to design the roof slabs also for floor loads if latter is greater. 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 1.35m 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. 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 dicontinuous 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. This will be evident from Table C-4.

Slab S3 will be just minimum 100 mm thick with minimum steel #8 at 240 mm for short span and 06 at 180mm as distribution steel as obtained from Table C-3. End shear = 10.5x1.35/2 = 7.09 kN/m. End shear for loft =8x1.35/2=5.4 kN/m.Total together = 12.50 kN/m for bath room and

Total end shear for W.C. = 10.5x1.05/2 + 8x1.05/2 = 9.70 kN/m.

Brief design calculations for slab S1 are given in tabular form below.

Slab	L_x m	L_y m	$\beta = L_y/L_x$	Boun. cond ~~No.~~	Allow L/D ratio	Req. D mm	Assumed q_u kN/m²	$q_u L_x^2$ kN.m/m	α_x	α_y --	m_x kN.m/m	m_y kN.m/m
S1	3.35	3.40	1.015	9	28	120	9	101	.0572	.056	5.78	5.66

Short span steel : #8 at 300 (d = 100 mm) from Table C-2.
Long span steel : #8 at 270 (d = 90 mm) from Table C-2.

In practice, reinforcement in both direction is provided at spacing of 200 mm.

Equivalent UDL for B.M.-Long edge: $w_{ueqb} = 0.5 \times 9 \times 3.35\ [1-1/(3 \times 1.015^2)] = 10.20$ kN/m

-Short Edge: $w_{ueqb} = 9 \times 3.35/3 = 10.05$ kN/m

Equivalent UDL for Shear-Long Edge: $w_{ueqs} = 0.5 \times 9 \times 3.35\ [1-1/(2 \times 1.015)] = 7.65$ kN/m

-Short Edge: $w_{ueqs} = 9 \times 3.35/4 = 7.54$ 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 - 300	#8 - 270	Two-way
S2	100	#8 - 240	ϕ6 - 180	One-way
S3	100	#8 - 240	ϕ6 - 180	One-way
S4	100	#8 - 240	ϕ6 - 180	One-way
S5	100	#8 - 240	#8 - 220	Two-way

End Shears from Floor slabs have been shown in Fig.8.7

8.4.3 Design of Stairs

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 2.825 m and overhang of 1.075 m.

Planning : Assuming Tread T = 250 mm and 9 risers in each of two flights,
Rise R = 3000/18 = 167mm

Hence, $\sec\theta = \sqrt{R^2 x T^2}/T = 1.20$.

Imposed load : q_i = (weight of steps + FF + LL) = 25x.167/2+1+3 = 6.09 kN/m²

Required D for serviceability = 130mm (Refer Appendix C-1).

Total ultimate load, * q_u = 1.5(25x.13x1.20+6.09) = 15.00 kN/m²

Maximum support moment $M_{u,max}$ = 15.00 x 1.075²/2 = 8.67 kN.m/m

For maximum span moment, there shall be only 0.9DL on overhang, and 1.5(DL+LL) on span portion as shown in the Fig.8.3.
0.9DL=0.9x(25x.13x1.2+6.09-3.0)=6.29kN/m

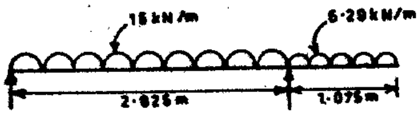


Fig. 8.3 Loads on stairs for maximum span moment

Support moment due to this = 6.29x1.075²/2 = 3.63 kN.m/m.
Maximum span moment * = 15.00x2.825²/8- 3.63/2 = 13.15 kN.m/m

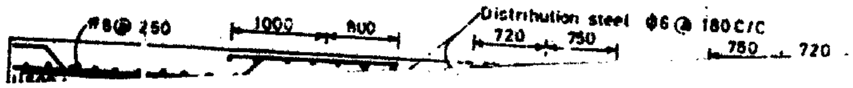
Required Main steel is 8 at 130 mm from Table C-2, and
Distribution steel is ϕ6 at 140 mm from Table C-3.

End Reactions

At discontinuous edge (B18):R_{B18}= 15x2.825/2-3.63/2.825 = 19.90 kN/m

At overhanging edge (B19): Taking moments about B18 with all spans fully loaded ,R_{B19}= 15(2.825 + 1.075)²*/(2x2.825) = 40.38 kN/m

***Note** : At ground floor the first flight of stairs will have 11 risers while the second flight will have only 7 risers and subsequently each will have 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 2.825 m. Infact, 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 one flight and the beam will get partially loaded. However the beam is designed to carry full load for computational simplicity and ease in laying.


#8@ 250
1000
800
Distribution steel ϕ6 @ 180 C/C
720
750
750
720

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 beams.

b) Values in parenthesis represent equivalent UDL in kN/m for bending moment transferred to beams.

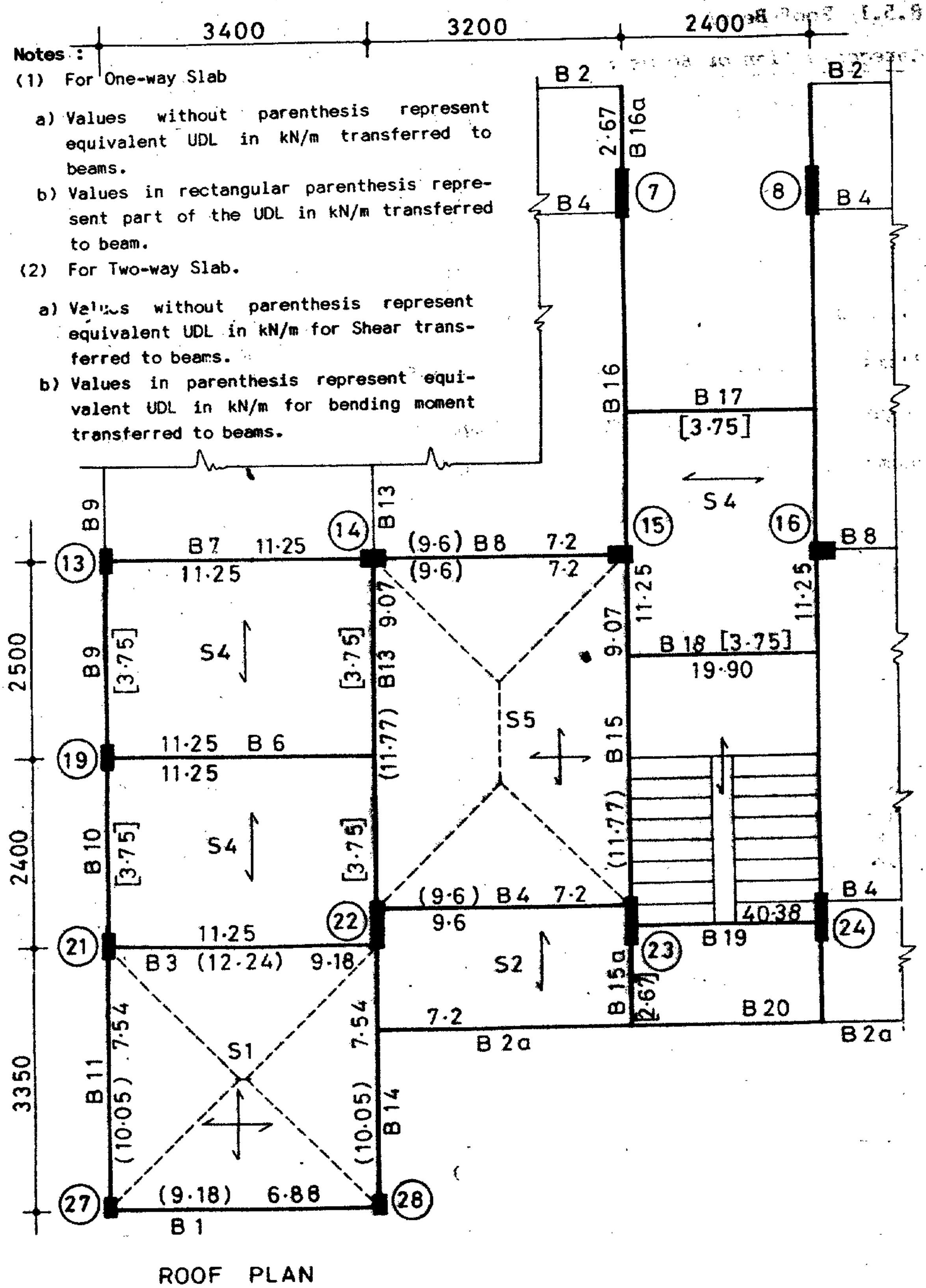


Fig. 8.4 Plan Showing Roof Slab End Shears Transferred to Beams.

8.5 DESIGN OF BEAMS

8.5.1 Roof Beams

Categorisation of Beams :

Category: Ia-Simply supported - UDL		Ib-S.S.UDL & Pt.Load.		II-One end: S.S.&other: continuous: UDL		III-Both: end conti.: UDL		V - Miscellaneous		
Beam	Span	Beam	Span	Beam	Span	Beam	Span	Beam	Span	Cross beam
B1	3.4	B6	3.4	- Nil -		B7	3.4	B9	2.5	B13	4.3	(B6)
B2	3.2	B17	2.4			B8	3.2	B10	2.4	B14	3.35	(B2)
B3	3.4	B18	2.4			B11	3.5	B20	2.4	B15-15a	1.6/4.3	(B2a,B20,B18)
B4	3.2	B19	2.4			B2a	3.2		_	B16-16a	4.3/11.6	(B17,B2)

Beams B5 and B12 are not provided at roof level.

(a) Design of Beams of Category. Ia : Simply Supported Beams -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 line in Fig.8.4 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). * At mid-landing

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	150	150	150	150	150	150	150	150
Depth D mm	300	300	300	300	300	300	300	300
Flange thickness D_f mm	100	100	110	100	100	130	100	100
4. Equivalent ultimate U load for Bending Moment :								
(a) Slab Right	S4	S4	S1	S5	S2	Stair	S4	-
Load w_{u1} kN/m	11.25	11.25	9.18	9.60	7.20	40.38	3.75	-
(b) Slab Left	S1	S4	-	S2	-	-	Stair	S4
Load w_{u2} kN/m	12.24	11.25	-	9.60	-	-	19.90	3.75
(c) Wall w_{uw} kN/m	-	-	3.00	-	5.25	-	13.50	3.00
(d) Imposed w_{ui} kN/m	23.49	22.50	12.18	19.20	12.45	40.38	37.15	6.75
(e) Self w_{us} kN/m	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10
(f) Total w_u kN/m	24.59	23.60	13.28	20.30	13.55	41.48	38.25	7.85
5. $M_{u.max} = w_u L^2/8$ kN.m	35.53	34.10	19.19	25.98	17.34	29.80	27.54	5.65
6. Section at Midspan	T	T	L	T	L	L	L	R
$b_f = L_o/6 + b_w + 6D_f$	1317	1317	-	1283	-	-	-	-
$b_f = L_o/12 + b_w + 3D_f$	-	-	763	-	716	740	650	150
b_f/b_w	8.78	8.78	5.09	8.55	4.78	4.93	4.33	1.00
7. Main Steel:								
Top St. N - # mm	2-10	2-10	2-10	2- 8	2- 10	2-10	2-10	2-10
Bottom Bt. N - # mm	-	-	1- 8	1-10	1- 8	-	-	-
Bottom St. N - # mm	2-16	2-16	2-10	2-12	2-10	3-12	3-12	2-10
A_{st} provided sq.mm	402	402	207	305	207	339	339	157
Effective Depth mm	267	267	270	269	270	269	269	270
M_{ur} provided kN.m	37.53	37.53	19.64	28.87	19.61	31.39	31.16	13.66

8.5.1 Design of Roof Beams : (a) Category – I Continued :

Beam Mark	B3	B6	B1	B4	B2	B19	B18	B17
8. Equivalent ultimate UDL for Shear: (Refer Fig.8.4)								
(a) Slab Right.	S4	S4	S1	S5	S2	Stair	S4	–
Load w_{u1} kN/m	11.25	11.25	6.88	7.20	7.20	40.38	@3.75	–
(b) Slab Left.	S1	S4	–	S2	–	–	Stair	S4
Load w_{u2} kN/m	9.18	11.25	–	9.60	–	–	19.90	@3.75
(c) Wall w_{uw} kN/m	–	–	3.00	–	5.25	–	13.50	3.00
(d) Imposed w_{ui} kN/m	20.43	–	9.88	16.80	12.45	40.38	37.15	6.75
(e) Self w_{us} kN/m	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10
(f) Total w_{ue} kN/m	21.53	23.60	10.98	17.90	13.55	41.48	38.25	7.85
9. Design for Shear								
(a) $V_{u.max}=\frac{1}{2}w_{ue}L$ kN	36.60	40.12	18.67	28.64	21.68	49.78	45.90	9.42
(b) A_{st1} N1 – # mm	2-16	2-16	2-10	2-12	2-10	3-12	3-12	2-10
(c) $V_{ur.min}$ kN	37.85	37.85	31.10	33.58	31.10	36.70	36.70	31.10
(d) V_{uD} kN	–	32.05	–	–	–	35.51	32.74	–
(e) V_{uc} kN	–	–	–	–	–	–	–	–
(f) $V_{us}=V_{uD}-V_{uc}$ kN	–	–	–	–	–	–	–	–
(g) Des.Stirr.#-s mm	–	–	–	–	–	–	–	–
(h) L_{s1}(mm)/N1	–	–	–	–	–	–	–	–
(j) Min.Stirr.ø-s mm	6-200	6-200	6-200	6-200	6-200	6-200	6-200	6-200
10. Load transferred to	C21&C22	C19	C27&C28	C22&C23	–	C23&C24	–	–
column at each end	36.60	40.12	18.67	28.64	21.68	49.78	@41.40	@4.92

Explanatory Notes : (Reference number corresponds to the serial number of the design given above.)

4.(a & b) The slab loads are taken from slab end shears given in Fig. 8.4.

7.(a) Top straight bars are only anchor bars and are normally 2 Nos. #10 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 225mm for 150mm wide beam. Hence, 8mm bars should not be provided as anchor bars for outer beams.

7.(b) 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 midspan steel should be available at top at support. If top anchor bars are able to provide this percentage, then there is no need of bent up bars.

7.(c) 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 D-3 such that M_{ur} provided is greater than $M_{u.max}$.

9.(b) Astl represent the area of bottom straight bars continued upto 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 (Vusv.min) This is obtained directly from Table D-1.

9 (d) V_{uD} = $V_{u.max}$ – $w_{ue}(b_s/2 + d)$ where, b_s = breadth of support = 150 mm.

9.(e) V_{uc} = shear resistance of concrete for given A_{st1} obtained from Table D-1.

9 (g) Design of stirrups is done using Table D-8.

9 (h) L_{s1} = length of shear zone for designed stirrups.= $(V_{u.max} - V_{ur.min})/w_{ue}$

9.(j) The spacing of minimum stirrups can be obtained from Table D-7. But the maximum spacing shall be limited to .75d or 450mm whichever is less i.e. 200mm

@ These loads are getting duplicated, and hence their contribution has not been taken.That is why column loads for B17 & B18 are less by 3.75x2.4/2=4.5 kN.

8.5.1 Design of Roof Beams : (a) Category – I Continued

Beam Mark	B3	B6	B1	B4	B2	B19	B18	B17
11. Bond:								
(a) Max. Bar dia. mm	16	16	10	12	10	12	12	10
(b) Requd. $L_d = 57\phi$ mm	912	912	570	684	570	684	684	570
(c) $L_{ex} = L_d/3 - b_s/2$ mm	229	229	115	153	115	153	153	115
(d) A_{st1}/A_{st}	1	1	0.76	0.74	0.76	1	1	1
(e) Available L_d mm	1558	1441	1152	1021	1056	969	885	2001
12. Deflection:								
(a) $p_t = 100A_{st}/(b_f d)$%	0.12	0.12	0.10	0.09	0.10	0.16	0.19	0.39
(b) Allowable L/d	>20	>20	>20	>20	>20	>20	>20	>20
(c) Actual L/d ratio	<20	<20	<20	<20	<20	<20	<20	<20

Explanatory Notes (continued) (Reference No. to Left is Step No. in design above)

11.(a) Maximum bar diameter is that of bottom straight bar in tension at discontinuous end.

11.(b) For M15-Fe415 required $L_d = 57\,\phi$.

11.(c) bs is assumed to be equal to 150 mm on the safer side because the column dimension can be known only later on.

In ratio A_{st1}/A_{st}, A_{st} is the area of midspan tension steel and A_{st1} area of bottom steel at support. This ratio is required for obtaining L_d.

where, $L_d = 1.3(A_{st1}/A_{st})\,M_{ur}/V_{u.max} + L_{ex}$ at simple support offering compressive reaction.

$L_d = (A_{st1}/A_{st})\,M_{ur}/V_{u.max} + L_{ex}$, where compression reaction is not available (as at point of contraflexure i, $V_i = V_{u.max}$),

and M_{ur} is the moment of resistance provided at the midspan.

12.(a) In the expression for p_t, A_{st} is the area of steel at midspan.

12.(b) Allowable L/d ratio = Basic L/d $x\alpha_1 x\alpha_3$

Basic L/d raio = 20 for simply supported beam and 26 for continuous beam

(i) For Rectangular beam: $\alpha_3 = 1$, and for $p_t < 0.87$ %, $\alpha_1 > 1$.

Hence, allowable L/d ratio > 20 for $p_t < 0.87$ %.

(ii) For Flanged beam, $\alpha_3 > 0.8$ for $b_w/b_f < 0.3$ i.e. $b_f/b_w > 3.33$

and $\alpha_1 > 1.25$ for $p_t < 0.43$ %.

Hence, $\alpha_1\alpha_3 > 1$ for $p_t < 0.43$ and $b_f/b_w > 3.33$.

Therefore, allowable L/d > 20 for $p_t < 0.43$ and $b_f/b_w > 3.33$

(b) Category – I(b) Simply Supported Beam UD Load & Point Load Nil

(c) Category – II & III :

		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	metres	3.4	3.2	3.35	3.2	2.5	2.4	2.4
3. Section : Width b	mm	150	150	150	150	150	150	150
Depth D	mm	300	300	300	300	300	300	300
Slab thickness D_f	mm	100	100	110	100	100	100	100

8.5.1 Design of Roof Beams (c) Category II & III Continued

			Category -II One End S.S. and other continous - UDL				Category-III Both Ends continuous-UDL		
1.	Beam Mark		B7	B8	B11	B2a	B9	B10	B20
4.	Equivalent UDL for Bending Moment								
	(a) Slab Right		S4	S5	S1	S2	S4	S4	-
	Load w_{u1}	kN/m	11.25	9.60	10.05	7.20	3.75	3.75	-
	(b) Slab Left		S4	S5	-	-	-	-	-
	Load w_{u2}	kN/m	11.25	9.60	-	-	-	-	-
	(c) Wall w_{uw}	kN/m	-	-	3.00	5.25	3.00	3.00	3.00
	(d) Imposed w_{ui}	kN/m	22.50	19.20	13.05	12.45	6.75	6.75	3.00
	(e) Self w_{us}	kN/m	1.10	1.10	1.10	1.10	1.10	1.10	1.10
	(f) Total w_u	kN/m	23.60	20.30	14.15	13.55	7.85	7.85	4.10
5	$M_{u.max}$	kN.m	27.28	20.78	15.88	13.87	4.09	3.77	1.98
6	Section at Midspan		T	T	L	L	R	R	R
	L_o/L		0.7	0.7	0.7	0.7	-	-	-
	L_o	mm	2380	2240	2345	2240	-	-	-
	$b_f=L_o/6+6D_f+b_w$ T	mm	1146	1123					
	$b_f=L_o/12+3D_f+b_w$ L	mm			675	636	150	150	150
	b_f/b_w		7.64	7.48	4.50	4.24	1.00	1.00	1.00
7	Main Steel :								
	(a) At Midspan:								
	Top St. N-#	mm	2-12	2-12	2-10	2-10	2-10	2-10	2-10
	Bottom Bt. N-#	mm	1-12	1-12	1-8	-	-	-	-
	Bottom St. N-#	mm	2-12	2-12	2-10	2-10	2-10	2-10	2-10
	Provided A_{st}	sq.mm	339	339	207	157	157	157	157
	Effective depth	mm	269	269	270	270	270	270	270
	M_{ur} provided	kN.m	31.94	31.92	19.57	14.92	13.67	13.67	13.67
	(b) At Continuous End								
	Top St. N-#	mm	1-12+2-12	2-12+1-12	2-10+1-8	2-10	2-10	2-10	2-10
	Bottom St. N-#	mm	2-12	2-12	2-10	2-10	2-10	2-10	2-10
	A_{st} provided	sq.mm	339	339	207	157	157	157	157
	Effective depth	mm	269	269	270	270	270	270	270
	M_{ur} provided	kN.m	29.32	29.32	17.33	13.66	13.66	13.66	13.66
8	Equivalent UDL for Shear :								
	(a) Slab Right		S4	S5	S1	S2	S4	S4	-
	Load	kN/m	11.25	7.2	7.54	7.20	@3.75	@3.75	-
	(b) Slab Left		S4	S5	-	-	-	-	-
	Load	kN/m	11.25	7.2	-	-	-	-	-
	(c) Wall	kN/m	-	-	3.00	5.25	3.00	3.00	3.00
	(d) Imposed	kN/m	22.50	14.40	10.54	12.45	6.75	6.75	3.00
	(e) Self	kN/m	1.10	1.10	1.10	1.10	1.10	1.10	1.10
	(f) Total w_{ue}	kN/m	23.60	15.50	11.64	13.55	7.85	7.85	4.10
9	Design for Shear								
	(a) At Continuous End								
	$V_{u.max}$	kN	48.14	29.76	23.40	26.02	9.81	9.42	4.92
	A_{st1} N1-#	mm	1-12+2-12	2-12+1-12	2-10+1-8	2-10	2-10	2-10	2-10
	$V_{ur.min}$	kN	36.70	36.70	33.05	31.10	31.10	31.10	31.10
	V_{uD}	kN	40.02	-	-	-	-	-	-
	V_{uc}	kN	22.66	-	-	-	-	-	-
	V_{us}	kN	17.36	-	-	-	-	-	-
	Des.Stirrups ϕ-smm		6-180	-	-	-	-	-	-
	L_{s1} / N_{s1}	mm	484/4	-	-	-	-	-	-
	Min.stirrups ϕ-smm		6-200	6-200	6-200	6-200	6-200	6-200	6-200

8.5.1 Design of Roof Beams : (c) Category II & III Continued

			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
9.	Design for Shear : Continued								
	(b) At Discontinuous End								
	$V_{u.max}$	kN	36.11	22.32	17.55	19.51	-	-	-
	A_{st1} N1-#	mm	2-12	2-12	2-10	2-10			
	$V_{ur.min}$	kN	33.58	33.58	31.10	31.10	-	-	-
	V_{uD}	kN	28.00	-	-	-	-	-	-
	V_{uc}	kN	-	-	-	-	-	-	-
	V_{us}	kN	-	-	-	-	-	-	-
	Des.Stirrups #-smm		-	-	-	-	-	-	-
	L_{s1}/N_{s1}	mm	-	-	-	-	-	-	-
	Min.Stirrups ϕ-smm		6-200	6-200	6-200	6-200	6-200	6-200	6-200
10	Load transferred to column:		C14	C14	C21	-	C19&C13	C19&C21	
	(a) Continuous end	kN	48.14	29.76	23.40	26.02	@5.13	@4.92	4.12
			C13	C15	C27	-			
	(b) Discontinuous end	kN	36.11	22.32	17.55	19.51	-	-	-
11	Bond :								
	(a) Continuous end								
	Max.bar diam.	mm	12	12	10	10	10	10	10
	Required L_d=57ϕ	mm	684	684	570	570	570	570	570
	Provided L_d=L/4	mm	850	800	837	800	625	600	600
	(b) Discontinuous end								
	Max.bar diam.	mm	12	12	10	10	10	10	10
	Required L_d=57ϕ	mm	684	684	570	570	570	570	570
	L_{ex}=L_d/3 - b_s/2	mm	153	153	115	115	270	270	270
	A_{st1}/A_{st}		0.67	0.67	0.76	1	-	-	-
	Available L_d	mm	904	1390	1217	1163	3817	3964	5956
12	Deflection :								
	p_t = 100 $A_{st}/(b_f.d)$	mm	0.11	0.11	0.11	0.09	0.25	0.25	0.25
	Allowable L/d ratio		> 26	> 26	> 26	> 26	> 26	> 26	> 26
	Actual L/d ratio		< 26	< 26	< 26	< 26	< 26	< 26	< 26

@ These loads are getting duplicated and hence their contribution in column loads has not been taken. 9.81-3.75x2.5/2 = 5.13 for B9, and
9.42-3.75x1.2/2 = 4.92 for B10.

Explanatory Notes For Design Step No.

4.(c) Beam B20 : This beam at roof level will carry grill for height of 2 m imposing load of 3 kN/m.

9.(a) 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.

(d) Category - V : Miscellaneous Beams

		B14	B13	B15a	B15	B16	B16a
1 Beam Mark		B14	B13	B15a	B15	B16	B16a
2 Span L	metres	3.35	4.3	1.6	4.3	4.3	1.6
3 Section : Width b	mm	150	150	150	150	150	150
Depth D	mm	375	375	*600	*600	*600	*600
Depth of Slab D_f	mm	110	100	100	100	100	100
4 Equivalent UDL for B.M.							
(a) Slab Right		-	S5	S4	S4	S4	-
Load w_{u1}	kN/m	-	11.77	11.25	11.25	11.25(1.75m)-	
(b) Slab Left		S1	S4	S2	S5	S5	S2
Load w_{u2}	kN/m	10.05	@3.75	2.67	11.77	11.77	2.67
(c) Wall w_{uw}	kN/m	3.00	-	13.5	13.5(2.3m)	3(2.55m)	3.0
(d) Imposed w_{ui}	kN/m	13.05	15.52	27.42	36.52/23.02	14.77	5.67
(e) Self w_{us}	kN/m	1.5	1.5	2.8	2.8	2.8	2.8
(f) Total Ult. w_u	kN/m	14.55	17.02	30.22	39.32/25.82	17.57	8.47
(g) Beam Reaction from		B2	B6	B2a+B20	B18	B17	B2
Point Load	kN	21.68	40.12	30.94	45.90	9.42	21.68
Distance from nearest column		1.0	1.80	-	2.0	1.75	1.0

		Midsp	Intsup	Midsp	Right	Left	Midsp	Intsup	Midsp	Right
5 $M_{u.max}$ at Value in	kN.m	13.24	37.64	40.25	43.38	46.05	68.50	65.78	9.50	25.92
		B14		B13			B15		B16	
6 Section type at midspan		Fl.	Rect.	Fl.	Rect.	Rect.	Fl.	Rect.	Fl.	Rect.
L_o = .7L	mm	2345		3010			3010		3010	
b_f=.7L/12+b_w+3D_f	mm	645		700			700		700	
b_f/b_w		4.30		4.67			4.67		4.67	
7 Main Steel Top St. N - #	mm	2-10	2-12 +2-10	2-10	3-12 +2-10	1-12 +2-10	2-10	2-12 +2-10	2-10	2-10
Bottom St. N - #	mm	-	-	2-10	-	-	1-12	-	-	-
Bottom St. N - #	mm	2-10	2-10	+2-12	2-12	2-12	+2-12	2-12	2-10	2-12
Provided Ast	mm²	157	383	383	497	305	339	383	157	226
Asc	mm²	-	157	-	226	226	-	226	-	-
Effective Depth	mm	345	342	331	331	569	569	555	570	569
M_{ur} provided	kN.m	19.18	40.40	43.62	44.60	50.62	68 10	67.04	31.92	43.04

			B14	B13	B15a	B15	B16	B16a
8 Equivalent UDL for Shear.								
(a) Slab Right			-	S5	S4	S4	S4	-
Load	w_{u1}	kN/m	-	9.07	11.25	11.25	11.25(1.75m)-	
(b) Slab Left			S1	S4	S2	S5	S5	S2
Load	w_{u2}	kN/m	7.54	@3.75	@ 2.67	9.07	9.07	@2.67
(c) wall		kN/m	3.00	-	13.5	13.5(2.3m)	3(2.55m)	3.0
(d) Imposed		kN/m	10.54	12.82	27.42	33.82/20.32	/12.07	5.67
(e) Self		kN/m	1.50	1.50	2.8	2.8	2.8	2.8
(f) Total		kN/m	12.04	14.32	30.22	36.62/23.12	/14.87	8.47
(g) Beam reaction from			B2	B6	B2a+B20	B18	B17	B2
Point Load		kN	21.68	40.12	30.94	45.90	9.42	21.68
Distance from nearest col.			1.0	1.8	1.0	2.0	1.75	1.0

8.5.1 Design of Roof Beams : (d) Category – V Continued

Beam Mark		B14		B13		B15a	B15		B16		B16a
9. Shear at		Left	Right	Left	Right	Canti.	Left	Right	Left	Right	Canti
$V_{u.max}$	kN	15.40	46.61	52.78	48.92	61.16	89.23	87.20	58.37	29.48	30.15
A_{st1} N – #	mm	2-10	2-10 +2-12	2-10 +2-12	3-12 +2-10	2-10 +1-12	2-10 +1-12	2-12 +2-10	2-12 +2-10	2-10	2-12
A_{st1}/A_{st}		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
$V_{ur.min}$	kN		44.27	44.27	45.11	62.74	62.74	66.41	66.41	56.12	60.41
V_{uD}	kN		39.74	45.58	–	–	74.78	74.92	45.77	–	–
V_{uc}	kN		–	27.02	–	–	33.07	37.42	–	–	–
V_{us}	kN		–	18.56	–	–	32.65	34.94	–	–	–
Des.Stirrups ø –s	mm		–	6-225	–	–	6-180	6-180	–	–	–
L_{s1}(mm) / N1			–	–	–	–	640/5	900/6	–	–	–
Min.Stirrups ø – s	mm		6-200	6-200	6-200	6-200	6-200	6-200		6-200	6-200
10. Load transferred to column.		C28	C22	C22	C14	C23	C23	C15	C15	C7	C7
		15.40	46.61	44.72	40.86	58.49	89.23	87.20	58.37	29.48	27.48

11. Bond :		B14		B13		B15a	B15		B16		B16a
(a) Top Steel		Left	Right	Left	Right	Right	Left	Rht	Left	Rht	Left
Max. bar diam.	mm	10	12	12	12	12	12	12	12	10	10
Required L_d=57ø	mm	570	684	684	684	684	684	684	684	570	570
Available L_d=L/4	mm	1075	1075	837	837	1000	1075	1075	1075	1075	1000
			Provide	912	912						
(b) Bottom Steel :											
Check at		Sup.	P.I.	P.I.	P.I.	–	P.I.	P.I.	P.I.	P.I.	–
Dist. of PI from Sup.		–	.84	.73	.90	–	.54	.80	1.44	.97	–
Max. bar diam.	mm	10	10	12	12	–	12	12	10	10	–
Required L_d=57ø	mm	570	570	684	684	–	684	684	570	570	–
Available A_{st1} N-#		2-10	2-10	2-12	2-12	–	2-12	2-12	2-10	2-10	–
Available M_1=M_{ur}		19.16	19.16	27.24	27.24	–	43.04		13.92		–
V	kN	15.40	36.50	40.35	33.60	–	73.63	72.3	26.95	18.24	–
L_{ex}	mm	115	345	344	344	–	342	342	345	345	–
Available L_d	mm	1757	885	1021	1152	–	927	937	861	1180	–

12 Deflection:		B14	B13	B15a	B15	B16
p_t = 100 Ast/(b_fd)	%	0.07	0.15		0.25	0.07
Allowable L/d		> 26	> 26		> 26	> 26
Actual L/d		< 26	< 26		< 26	< 26

Explanatory Notes to Design Step No.

(3) Since main doors in walls below B15-B16 have ventilators, depth of beams of 600mm has been provided for these beams since the clearance between the top of door and floor is 600 mm only. The beam serves the purpose of lintel as well as floor beam.

(5),(8),(9) <u>Calculations of Bending Moments and Shears for Beam B13 – B14</u>
Since live load is small compared to dead load, various loading cases need not be considered. The details of loading are shown in Fig.8.5. (a)

* Fixed End Moments :
(* See Note on page 237)

M_{FCB} = 40.12x2.5x1.8²/4.3² +17.02x4.3²/12=43.80 kN.m

M_{FBC} = 40.12x2.5²x1.8/4.3² +17.02x4.3²/12=50.63 kN.m

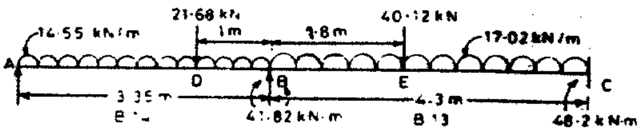


Fig 8.5(a) Loading for Calculation of bending moment

M_{FBA} = 21.68x1.0x2.35²/3.35² +14.55x3.35²/8 + (21.68x1.0²x2.35/3.35²)/2 = 33.35 kN.m

* Distribution Factors:- $d_{BC} = (4/4.3)/(4/4.3+3/3.35) = 0.51$, $d_{BA} = 1.0 - 0.51 = 0.49$

Moment Distribution:

	A	B		C
Distribution factors		.49	.51	
Fixed end moments	0	33.35	-50.63	43.80
Distributed moments & Carry over moment		8.47	8.81	4.40
Final support moments:	0	41.82	-41.82	48.20

* Note: The beam is symmetrical @ central column C14. Considering this even span symmetry, support C is assumed as fixed. Support at A is simply supported. The stiffness of BC 4/4.3 while that of AB is 3/4th of (4/3.35).

Span Moments :

Span BC : $V_{CB} = 40.12 \times 1.8/4.3 + 17.02 \times 4.3/2 + (48.20 - 41.82)/4.3 = 54.87$ kN

$x_{max} = 54.87/17.02 > 2.5$ m $\therefore x_{max} = 2.5$ m.

$M_{span.max} := 54.87 \times 2.5 - 17.02 \times 2.5^2/2 - 48.20 = 35.61$ kN.m

$V_{BC} = 40.12 + 17.02 \times 4.3 - 54.87 = 58.44$ kN

Span AB : $V_{BA} = 21.68 \times 2.35/3.35 + 14.55 \times 3.35/2 + 41.82/3.35 = 52.06$ kN

$V_{AB} = 21.68 + 14.55 \times 3.35 - 52.06 = 18.36$ kN

$x_{max} = 18.36/14.55 = 1.262$ m

$M_{span.max} = 18.36 \times 1.262 - 14.55 \times 1.262^2/2 = 11.58$ kN.m

	A	Mspan	B	Mspan	C
Elastic Moments before Redistribution	0	11.58	41.82	35.61	48.20
Redistribution of moments 10 % at C					- 4.82
10 % at B			- 4.18		
Design Moments after Redistribution		13.24	37.64	40.25	43.38

These values have been entered in Step.5.

Revised Calculations for Span Moments :

Revised $V_{AB} = 18.36 + 4.18/3.35 = 19.61$ kN. $x_{max1} = 19.61/14.55 = 1.35$ m from A.

Revised $M_{max} = 19.61 \times 1.35/2 = 13.24$ kN.m in AB.

Distance of PI (in DB) from A : $19.61 x_o - 14.55 x_o^2/2 - 21.68(x_o - 2.35) = 0$

$x_o = 2.51$ m from A or 0.84 m from B.

Revised $V_{BC} = 58.44 + 0.64/4.3 = 58.58$ kN ; $V_{CB} = 40.12 + 17.02 \times 4.3 - 58.58 = 54.73$ kN

$x_{max2} = 54.73/17.02 > 2.5$. $\therefore x_{max2} = 2.5$ m from C.

Revised $M_{max} = 54.73 \times 2.5 - 17.02 \times 2.5^2/2 - 43.38 = 40.25$ kN.m in BC.

Distance of Point of Inflexion (PI)(Left) from B :

$58.58 x_1 - 17.02 x_1^2/2 - 37.64 = 0$ which gives $x_1 = 0.73$ m from B.

Distance of Point of Inflexion (PI)(Right) from C :

$54.73 x_2 - 17.02 x_2^2/2 - 43.38 = 0$ which gives $x_2 = 0.90$ m from C.

Shear in Beams : [For this, equivalent loading of Two-way slabs for shear as given in Step 8(f) on page 236 will be taken. This is shown in Fig.8.5(b)].

$V_{AB} = 12.04 \times 3.35/2 + 21.68 \times 1.0/3.35 - 37.64/3.35 = 15.40$ kN

$V_{BA} = 12.04 \times 3.35/2 + 21.68 \times 2.35/3.35 + 37.64/3.35 = 46.61$ kN

$V_{BC} = 14.32 \times 4.3/2 + 40.12 \times 2.5/4.3 - (43.38 - 37.64)/4.3 = 52.78$ kN

$V_{CB} = 14.32 \times 4.3/2 + 40.12 \times 1.8/4.3 + (43.38 - 37.64)/4.3 = 48.92$ kN

These values have been entered in Step.9.

Shears at Point of Inflexions :

In AB : $V_o = 46.61 - 12.04 \times 0.84 = 36.50$ kN.

In BC : $V_1 = 52.78 - 17.02 \times 0.73 = 40.35$ kN.

$V_2 = 48.92 - 17.02 \times 0.90 = 33.60$ kN.

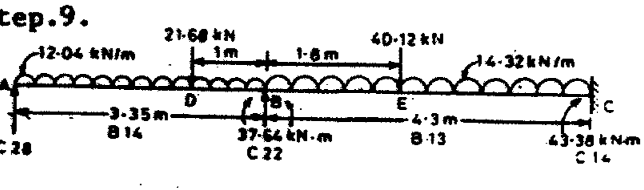


Fig 8.5(b) Loading for calculation of Shear

@Loads transferred to columns from

AB: On column C28 = V_{AB} = 15.40kN ; On column C22 = V_{BA} = 46.61 kN

BC: C22 = $V_{BC} - 3.75 \times 4.3/2$ = 44.72 kN ; C14 = $V_{CB} - 3.75 \times 4.3/2$ = 40.86 kN

These value have been entered in Step.10.

@The slab load of 3.75kN/m is duplicated, and hence deducted for finding column loads.

Calculations of Bending Moments & Shears for Beams B15a-B15-B16-B16a

For a beam AB partially loaded by UDL the fixed end moments are given by:-

$M_{FAB} = w\,k^2(3k^2-8k+6)L^2/12$
$M_{FBA} = w\,k^3(4-3k)L^2/12$
see Fig.8.6(a)

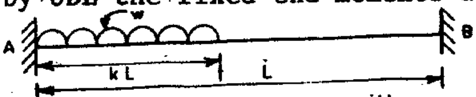
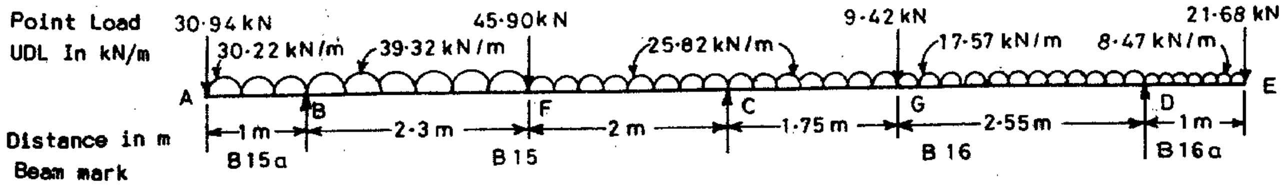


Fig. 8.6 (a) Fixed beam with partial UDL of intensity w

The loading on beam for calculation of bending moments is shown in Fig.8.6(b).



Span in m

Fig 8.6(b) Loading for calculation of bending moment.

For BF $w' = (39.32 - 25.82) = 13.50$ kN/m , $k = 2.30/4.3 = 0.535$
For CG $w' = (25.82 - 17.57) = 8.25$ kN/m , $k = 1.75/4.3 = 0.407$

Fixed End Moments :

$M_{FBA} = 30.94 \times 1.0 + 30.22 \times 1 \times 1/2 = 46.05$ kN.m

$M_{FBC} = 45.9 \times 2.3 \times 2^2/4.3^2 + 13.5 \times .535^2(3 \times .535^2 - 8 \times .535 + 6)4.3^2/12 + 25.82 \times 4.3^2/12 = 77.97$ kN.m

$M_{FCB} = 45.9 \times 2.3^2 \times 2/4.3^2 + 13.5 \times .535^3 \times (4 - 3 \times .535) \times 4.3^2/12 + 25.82 \times 4.3^2/12 = 72.39$ kN.m

$M_{FCD} = 9.42 \times 1.75 \times 2.55^2/4.3^2 + 8.25 \times .407^2(3 \times .407^2 - 8 \times .407 + 6) \times 4.3^2/12 + 17.57 \times 4.3^2/12 = 39.69$ kN.m

$M_{FDC} = 9.42 \times 1.75^2 \times 2.55/4.3^2 + 8.25 \times .407^3 \times (4 - 3 \times .407) \times 4.3^2/12 + 17.57 \times 4.3^2/12 = 32.98$ kN.m

$M_{FDE} = 21.68 \times 1.00 + 8.47 \times 1 \times 1/2 = 25.92$ kN.m

Distribution Factors : $d_{CB} = d_{CD} = 0.5$ as spans and end conditions are same.

Moment Distribution		B	C	D
Distribution factors	:	0 : 1	.5 : .5	1 : 0
Initial Fixed End Moments	:	46.05:-77.97	72.39:-39.69	32.98:-25.92
Distributed moments	:	: 31.92	:	-7.06:
Carry over moments	:	:	15.96:- 3.53	:
	:	46.05:-46.05	88.35:-43.22	25.92:-25.92
Distributed moments	:	:	-22.56:-22.56	
Final support moments	:	46.05:-46.05	65.78:-65.78	25.92:-25.92

Span Moments :

$V_{BA} = 30.94 + 30.22 = 61.16$ kN

$V_{BC} = 45.9 \times 2.00/4.3 + 25.82 \times 4.3/2 + (39.32-25.82) \times 2.3 \times (1.15+2)/4.3 - (65.78-46.05/4.3 = 95.02$ kN

$V_{CB} = 45.9 + 39.32 \times 2.3 + 25.82 \times 2 - 95.02 = 92.96$ kN

$V_{CD} = 9.42 \times 2.55/4.3 + 17.57 \times 4.3/2 + 8.25 \times 1.75 \times (1.75/2+2.55)/4.3 + (65.78-25.92)/4.3 = 64.13$ kN

$V_{DC} = 9.42 + 25.82 \times 1.75 + 17.57 \times 2.55 - 64.13 = 35.28$ kN

$V_{DE} = 21.68 + 8.47 = 30.15$ kN

Span BC : x_{max} = 95.02/39.32 > 2.3 .'. x_{max} = 2.3 m

$M_{span.max}$ = 95.02x2.3 − 39.32 x 2.3²/2 − 46.05 = 68.50 kN.m

Span CD : x_{max} = 35.28/17.57 = 2.00 m

$M_{span.max}$ = 35.28x2.00 − 17.57x2.00²/2 − 25.92 = 9.50 kN.m

Beam Shear and Loads on Columns : For this, equivalent load for shear from two-way slabs as given in Step 8(f) of design (on page 235) will be taken. This is shown in Fig.8.6(c) below.

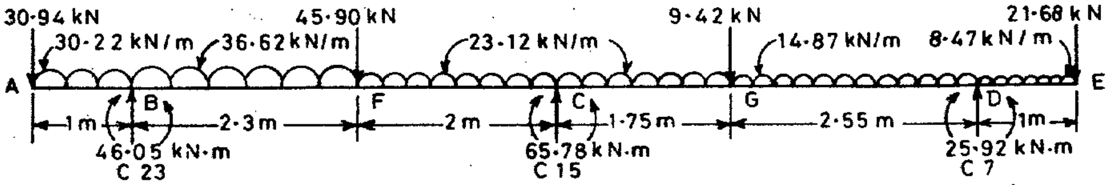


Fig. 8·6(c) Loading for calculation of shear

Shear in Beams:

	AB	BC	CB	CD	DC	DE
Con.load	30.94	21.35	24.55	5.59	3.83	21.68
UDL	30.22	61.70	22.53	11.24	26.68	8.47
		10.77	35.53	32.27	8.24	
Moment	–	-4.59	4.59	9.27	-9.27	–
Total	61.16	89.23	87.20	58.37	29.48	30.15
Less	-2.67					-2.67
Load on col.	58.49	89.23	87.20	58.37	29.48	27.48

Notes : (1) For One-way Slabs

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 beams.

b) Values in parenthesis represent equivalent UDL in kN/m for bending moment transferred to beams.

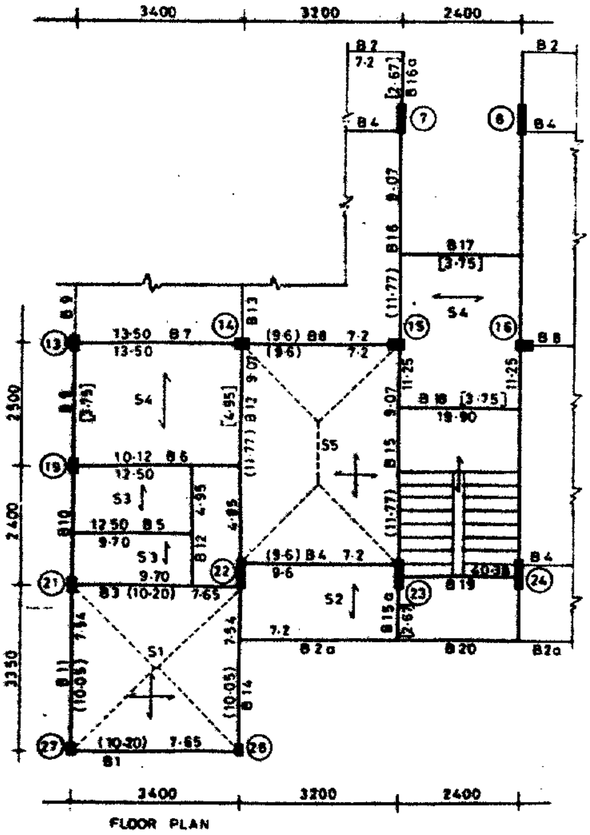


Fig. 8.7 Plan Showing Floor Slab End Shears Trasferred to Beams.

8.5.2 Floor Beams

(A) Categorisation and Grouping of Beams

Category - Ia Simply supported(SS) at both ends-UDL				Category Ib S.S. at both ends UDL & Point Load			Category II SS at one end & continuous at other-UDL		Category-III Continuous at both ends		Category V Miscellaneous		
Beam	Span	Beam	Span	Beam	Span	Cross	Beam	Span	Beam	Span	Beam	Span	Cross
B1	3.4	B11	3.35	B3	3.4	(B12)	B7	3.4	B20	2.4	B13	4.3	(B6)
B2	3.2	B17	2.4	B6	3.4	(B12)	B8	3.2			B14	3.35	(B2)
B4	3.2	B18	2.4	B10*	2.4	(B5)	B9	2.5			B15a	1.6	(B2a,20)
B5	2.3	B19	2.4	B12#	2.4	(B5)	B2a	3.2			B15	4.3	(B18)
											B16	4.3	(B17)
											B16a	1.6	(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 450mm. However, the beam is designed assuming D=450mm. The minor loading differences are ignored.

(B) Design of Beams

Category - I(a) : Simply Supported Beam : UDL only

Beams have not been taken serially but in descending order of span and loading so that heavier beams are designed first.

1. Beam Mark			B4	B1	B11	B2	B19	B18	B17	B5
2. Span L		metres	3.2	3.4	3.35	3.2	2.4	2.4	2.4	2.3
3. Section : Width b		mm	150	150	150	150	150	150	150	150
Depth D		mm	375	375	375	375	375	375	300	450
Depth of Slab	D_f	mm	100	120	120	100	130	100	100	100
4. Equivalent UDL for Bending Moment :										
(a) Slab Right			S5	S1	S1	-	Stair	S4	-	S3
Load	w_{u1}	kN/m	9.60	10.2	10.05	-	40.38	@3.75	-	12.50
(b) Slab Left			S2	-	-	S2	-	Stair	S4	S3
Load	w_{u2}	kN/m	9.60	-	-	7.20	-	19.90	@3.75	9.70
(c) Wall	w_{uw}	kN/m	18.00	18.00	18.00	5.25	-	-	3.00	9.00
(d) Imposed	w_{uf}	kN/m	37.20	28.20	28.05	12.45	40.38	23.65	6.75	31.20
(e) Self	w_{us}	kN/m	1.50	1.50	1.50	1.50	1.50	1.50	1.00	1.50
(f) Total	w_u	kN/m	38.70	29.70	29.55	13.95	41.88	25.15	7.75	32.70
5. $M_{u.max}$		kN.m	49.54	42.92	41.45	17.86	30.15	18.11	5.58	21.62
6. Section at Midspan			T	L	L	L	L	L	R	R
$b_f = .7xL/6 + 6D_f + b_w$		mm	1283							
$b_f = .7xL/12 + 3D_f + b_w$		mm		793	789	716	740	650	150	150
b_f/b_w			8.55	5.28	5.26	4.77	4.93	4.33	1.00	1.00
7. Main Steel										
Top St.	N - #	mm	2- 8	2-10	2-10	2-10	2-10	2- 8	2-10	2- 8
Bottom Bt.	N - #	mm	1-12	2-10	2-10	-	1- 8	1- 8	-	-
Bottom St.	N - #	mm	3-12	2-12	2-12	2-10	2-12	2- 8	2-10	2-10
M_{ur} Provided		kN.m	52.38	43.87	43.86	19.22	33.30	18.48	13.66	22.17
Ast Provided		sq.mm	452	383	383	157	276	151	157	157
Effective Depth		mm	331	331	331	345	344	346	270	420

Design of Floor Beams : Category I(a) : Continued

Beam Mark		B4	B1	B11	B2	B19	B18	B17	B5
8. Equivalent UDL for Shear :									
(a) Slab Right		S5	S1	S1	-	Stair	S4	-	S3
Load	kN/m	7.20	7.65	7.54	-	40.38	@3.75	-	12.50
(b) Slab Left	kN/m	S2	-	-	S2	-	Stair	S4	S3
Load	kN/m	9.60	-	-	7.20	-	19.90	@3.75	9.70
(C) Wall	kN/m	18.00	18.00	18.00	5.25	-	-	3.00	9.00
(d) Imposed	kN/m	34.80	25.65	25.54	12.45	40.38	23.65	6.75	31.20
(e) Self	kN/m	1.50	1.50	1.50	1.50	1.50	1.50	1.10	1.50
(f) Total	kN/m	36.30	27.15	27.04	13.95	41.88	25.15	7.85	32.70
9. Shear :									
$V_{u.max}$	kN	58.08	46.16	45.29	22.32	50.26	30.18	9.42	37.60
Astl N1 - #	mm	3-12	2-12	2-12	2-10	2-12	2-8	2-10	2-10
Astl /Ast		0.75	0.59	0.59	1.00	0.82	0.76	1.00	1.00
$V_{ur.min}$	kN	44.48	40.70	40.70	37.70	40.70	34.43	31.10	44.03
V_{uD}	kN	42.19	35.14	34.31	-	32.71	-	-	-
V_{uc}	kN	-	-	-	-	-	-	-	-
V_{us}	kN	-	-	-	-	-	-	-	-
Des.Stirrups N - #	mm	-	-	-	-	-	-	-	-
Lsl(mm)/N_1		-	-	-	-	-	-	-	-
Min.Stirrups N - ø	mm	6-225	6-225	6-225	6-225	6-225	6-225	6-225	6-225
10. Load transferred to		C22&23	C27&28	C21&27	-	C23&24	-	-	-
Columns at each end	kN	58.08	46.16	45.29	22.32	50.26	@25.68	@4.80	37.60
11. Bond: Max. bar diam.	mm	12	12	12	10	12	10	10	10
Required L_d = 57 ø	mm	684	684	684	570	684	570	570	570
L_{ex}	mm	153	153	153	115	153	115	115	115
Available L_d	mm	1054	883	897	1291	854	937	2026	882
12. Deflection :									
$p_t = 100A_{st}/(b_f.d)$	%	0.11	0.15	0.15	0.06	0.11	0.09	0.39	0.25
Allowable L/d ratio		> 20	> 20	> 20	> 20	> 20	> 20	> 20	> 20
Actual L/d ratio		< 20	< 20	< 20	< 20	< 20	< 20	< 20	< 20

@Column load = $V_{u.max}$ - 3.75x2.5/2

Category - *I(b) Simply Supported : II One End Sim.Supp. : III Both*
UD Load & Pt.Load :Other Continuous - UDL : Conti.

1. Beam Mark			B6	B3	B10	B12	B7	B8	B9	B2a	B20
2. Span L		metres	3.4	3.4	2.4	2.4	3.4	3.2	2.5	3.2	2.4
3. Section : Width b		mm	150	150	150	150	150	150	150	150	150
Depth D		mm	450	450	375	450	375	375	375	375	375
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	10.12	9.70	-	4.95	13.50	9.60	3.75	7.20	-
(b) Slab Left :			S3	S1	-	-	S4	S5	-	-	-
Load	w_{u2}	kN/m	12.50	10.20	-	-	13.50	9.60	-	-	-
(c) Wall	w_{uw}	kN/m	12.00	12.00	18.00	9.00	12.00	12.00	18.00	5.25	4.00
(d) Imposed	w_{ui}	kN/m	34.62	31.90	18.00	13.95	39.00	31.20	21.75	12.45	4.00
(e) Self	w_{us}	kN/m	2.00	2.00	1.50	2.00	1.50	1.50	1.50	1.50	1.50
(f) Total	w_u	kN/m	36.62	33.90	19.50	15.95	40.50	32.70	23.25	13.95	5.50
(g) Supported Beam			B12	B12	B5	B5	-	-	-	-	-
Beam Shear		kN	35.59	40.29	37.60	37.60	-	-	-	-	-
Distance from nearest col			1.10	1.10	1.05	1.05	-	-	-	-	-

Design of Floor Beam : Categories I(b), II & III : Continued

Beam Mark	Unit	B6	B3	B10	B12	B7	B8	B9	B2a	B20
5. $M_{u.max}$	kN.m	74.30	73.67	37.99	33.51	46.82	33.48	14.53	13.55	2.64
6. Section at Midspan		L	L	R	L	T	T	R	L	R
$L_o = 0.7\ L$	mm	3400	3400	2400	2400	2380	2240	1750	2240	–
L_o/L		1.00	1.00	1.00	1.00	0.70	0.70	0.70	0.70	–
$b_f = L_o/6 + b_w + 6D_f$ T	mm					1146	1123			
$b_f = L_o/12 + b_w + 3D_f$ L	mm	733	793		650				636	
b for Rect.	mm			150				150		150
b_f/b_w		4.89	5.29	1.00	4.33	7.64	7.48	1.00	4.24	1.00
7. Main Steel: (a) At Midspan										
Top St. N-#	mm	2-12	2-12	2-10	2-10	2-12	2-12	2-10	2-10	2-10
Bottom Bt. N-#	mm	1-12	1-12	2-10	1-12	2-12	2-10	–	–	–
Bottom St. N-#	mm	4-12	3-12	2-12	2-10	+2-12	+2-12	2-10	2-10	2-10
M_{ur} Provided	kN.m	78.41	78.73	40.40	39.83	53.19	44.42	17.91	19.20	17.91
Provided A_{st}	sq.mm	565	452	383	270	452	383	157	157	157
Effective Depth	mm	406	406	331	419	331	344	345	345	345
(b) At Continuous End						2-12	2-12			
Top St. N-#	mm	–	–	–	–	+2-12	+2-10	2-10	2-10	2-10
Bottom St. N-#	mm	–	–	–	–	2-12	2-12	2-10	2-10	2-10
M_{ur} Provided	kN.m	–	–	–	–	47.72	41.31	17.92	17.92	17.92
A_{st} Provided	sq.mm	–	–	–	–	452	383	157	157	157
Effective Depth	mm	–	–	–	–	331	331	345	345	345
8. Equivalent UDL for Shear										
(a) Slab Right		S4	S3		S3	S4	S5	S4	S2	–
Load	kN/m	10.12	9.70		4.95	13.50	7.2	@3.75	7.20	–
(b) Slab Left		S3	S1	–	–	S4	S5			
Load	kN/m	12.50	7.65	–	–	13.50	7.2	–	–	–
(c) Wall	kN/m	12.00	12.00	18.00	9.00	12.00	12.00	18.00	5.25	4.00
(d) Imposed	kN/m	34.62	29.35	18.00	13.95	39.00	26.40	21.75	12.45	4.00
(e) Self	kN/m	2.00	2.00	1.50	2.00	1.50	1.50	1.50	1.50	1.50
(f) Total	kN/m	36.62	31.35	19.50	15.95	40.50	27.90	23.25	13.95	5.50
(g) Supported beam		B12	B12	B5	B5					
Beam Shear		35.59	40.29	37.60	37.60	–	–	–	–	–
Dist. from nearest col.		1.1	1.1	1.05	1.05	–	–	–	–	–
9. Shear at continuous end : Right										
$V_{u.max}$	kN	73.76	66.30	44.55	40.29	82.62	53.57	34.88	26.78	6.60
A_{st1} N_1 – #	mm	4-12	3-12	2-12	2-10	4-12	(2-10+2-12)	2-10	2-10	2-10
$V_{ur.min}$	kN	53.86	50.57	40.70	44.03	45.89	44.27	37.70	37.70	37.70
V_{uD}	kN	55.74	50.88	36.38	–	66.18	41.88	–	–	–
V_{uc}	kN	32.69	–	–	–	28.64	–	–	–	–
V_{us}	kN	23.05	–	–	–	37.54	–	–	–	–
Des. Stirrups ϕ-smm		6-210	–	–	–	6-110	–	–	–	–
L_{s1} (mm)/N_1		543/4	–	–	–	906/10	–	–	–	–
Min. Stirrups ϕ-smm		6-225	6-225	6-225	6-225	6-225	6-225	6-225	6-225	6-225
(b) At Discontinuous End : Left										
$V_{u.max}$	kN	86.33	80.56	39.85	35.59	61.96	40.18	26.16	20.09	6.60
A_{st1} N_1 – #	mm	4-12	3-12	2-12	2-10	2-12	2-12	2-10	2-10	2-10
$V_{ur.min}$	kN	53.86	50.57	40.70	44.03	40.70	40.70	37.70	37.70	37.70
V_{uD}	kN	68.31	65.14	–	–	40.52	–	–	–	–
V_{uc}	kN	32.69	29.40	–	–	–	–	–	–	–
V_{us}	kN	35.62	35.74	–	–	–	–	–	–	–
Des. Stirrups ϕ-smm		6-140	6-140	–	–	–	–	–	–	–
L_{s1} (mm)/N_1		887/8	957/8	–	–	–	–	–	–	–
Min. Stirrups ϕ-smm		6-225	6-225	6-225	6-225	6-225	6-225	6-225	6-225	6-225

Design of Floor Beam : Categories I(b), II & III : Continued

Beam Mark		B6	B3	B10	B12	B7	B8	B9	B2a	B20
10.Load transferred to Column :										
at Continuous end		C19	C21	C21	B3	C14	C14	C13	–	–
Load in	kN	73.76	66.30	44.55	40.29	82.62	53.57	@30.19	–	–
at Discontinuous end		B13	C22	C19	B6	C13	C15	C19		
Load in	kN	86.33	80.56	39.85	35.59	61.96	40.18	@21.47	–	–
11.Bond:(a)Continuous End										
Max.bar diam.	mm	–	–	–	–	12	12	10	10	10
Required L_d=57ϕ	mm	–	–	–	–	684	684	570	570	570
Provided L_d	mm	–	–	–	–	850	800	625	800	600
(b) Discontinuous End										
Max.bar diam.	mm	12	12	12	10	12	12	10	10	10
Required L_d=57ϕ	mm	684	684	684	570	684	684	570	570	570
L_{ex}	mm	153	153	153	115	153	153	115	115	115
A_{st1}/A_{st}		0.8	0.75	0.74	0.58	0.50	0.67	1.0	1.0	1.0
M_1	kN.m	62.73	59.02	29.90	23.10	26.10	29.76	17.92	19.20	17.92
V	kN	86.33	80.56	39.85	35.59	61.96	40.18	26.16	20.09	6.60
Available L_d	mm	1097	1105	1128	997	709	1115	1005	1357	3645
12.Deflection :										
$p_t = 100A_{st}/(b_f.d)$	%	0.19	0.14	0.77	0.10	0.12	0.09	0.30	0.07	0.30
Allowable L/d ratio		> 20	> 20	> 20	> 20	> 26	> 26	> 26	> 26	> 26
Actual L/d ratio		< 20	< 20	< 20	< 20	< 26	< 26	< 26	< 26	< 26

Note :(i)Left or Right end of the beam is decided by looking at the plan from below or from right.(ii)Bentup bars are not used as shear reinforcement.
@(iii)Column load = $V_{u.max}$ −3.75x2.5/2

(Category – V : Miscellaneous Beams :

1. Beam Mark		B14	B13	B15a	B15	B16	B16a
2. Span L	m	3.35	4.3	1.0	4.3	4.3	1.0
3. Section : Width b	mm	150	150	150	150	150	150
Depth D	mm	450	450	600	600	600	600
Depth of Slab D_f	mm	120	100	100	100	100	100
4. Loads :							
(a) Slab Right			S5	–	S4	S4	–
Load w_{u1}	kN/m	–	11.77	–	11.25(2m)	11.25(1.75m)	–
(b) Slab Left		S1	S3	S2	S5	S5	S2
Load w_{u2}	kN/m	10.05	@4.95(1.8m)	2.67	11.77	11.77	2.67
(c) Wall w_{us}	kN/m	18.00	12.00	18.00	18.00/18.00/18.00		18.00
(d) Imposed w_{ui}	kN/m	28.05	28.72	20.67	29.77/41.02/29.77		20.67
(e) Self w_{us}	kN/m	2.00	2.00	2.80	2.80	2.80	2.80
(f) Total w_u	kN/m	30.05	30.72	23.47	32.57/43.82/32.57		23.47
(g) Pt.Load from beam		B2	B6	B2A +B20	B18	B17	B2
Value	kN	22.32	86.33	33.38	30.18	9.42	22.32
Distance from nearest column in metres		1.00	1.80	1.00	2.00	1.75	1.0
5. $M_{u.max}$ at		Midsp / Intsup	Midsp / Right	Supp	Middle / Supp	Middle	Supp.
Value	kN.m	26.89 / 61.79	86.73 / 86.82	45.11	65.98 / 58.67	45.98	34.05
6. Section at Midspan		L	L		L	L	
b_f=.7L/12+b_w+3D_f	mm	700	645		700	700	
b_f/b_w		4.67	4.30		4.67	4.67	

Category – V : Miscellaneous Beams – Continued

Beam Mark		B14	B13			B15		B16		B16a
7. Main Steel		Midsp	Intsup	Midsp	Right	Supp	Midsp	Supp	Midsp	Supp.
Top St. N – #	mm	2-12	4-12	2-12	2-16 +3-12	2-12 +1-8	2-12	3-12	2-12	2-12
Bottom Bt. N – #	mm	–	–	2-12	–	–	1-12	–	1-8	–
Bottom St. N – #	mm	2-12	3-12	4-12	3-12	2-12	2-12	2-12	2-12	2-12
M_{ur} Provided	kN.m	30.79	61.35	98.55	96.43	51.68	68.04	62.00	55.69	43.04
Provided Ast	sq.mm	226	452	678	741	276	339	339	276	226
Effective Depth	mm	419	405	405	401	569	569	569	569	569

8. Equivalent UDL for shear:		B14	B13	B15a	B15	B16	B16a
(a) Slab Right		–	S5	–	S4	S4	–
load	kN/m	–	9.07	–	11.25(2m)	11.25(1.75m)	–
(b) Slab Left		S1	S3	S2	S5	S5	S2
Load	kN/m	7.54	4.95	@2.67	9.07	9.07	@2.67
(c) Wall	kN/m	18.00	12.00	18.0	18.0/18.0/18.0		18.00
(d) Imposed	kN/m	25.54	26.02	20.67	27.07/38.32/27.07		20.67
(e) Self	kN/m	2.00	2.00	2.8	2.8	2.8	2.8
(f) Total	kN/m	27.54	28.02	23.47	29.87/41.12/29.87		23.47
(g) Beam reaction from		B2	B6	B2a+B20	B18	B17	B2
Point load	kN	22.32	86.33	33.38	30.18	9.42	22.32
Dist from nearest Col.	m	1.0	1.8	1.0	2.3	1.75	1.0

Beam		B14		B13		B15A	B15		B16		B16A
9. Design Shear:		Left	Right	Left	Right	Right	Left	Right	Left	Right	Left
$V_{u.max}$	kN	34.35	80.23	104.61	102.20	56.85	80.34	100.78	91.20	66.34	45.79
A_{st1} N1-#	mm	2-12	.. 4-12	...	2-16+3-12	2-12+1-8	...	3-12	...	3-12	2-12
$V_{ur.min}$	kN	–	53.86	53.86	59.18	63.05	63.05	65.90	65.90	65.90	60.41
V_{uD}	kN	–	67.01	91.16	88.75	–	61.10	74.30	64.72	47.10	–
V_{uc}	kN	–	32.69	32.69	38.25	–	–	36.20	–	–	–
V_{us}	kN	–	34.32	58.47	50.50	–	–	38.10	–	–	–
Des.Stirup	mm	– –	6-140	8-250	8-270	–	–	6-180	–	–	–
L_{s1}(mm)/N_1	mm	–	958/8	1800/9	1535/7	–	–	848/6	–	–	–
Min.St ϕ-s	mm	6-225	6-225	6-225	6-225	6-225	6-225	6-225	6-225	6-225	6-225
10. Load transferred to Columns :		C28	C22		C14	C23		C15		C7	
load in kN		34.35	80.23	101.0	93.42	54.18	80.34	100.78	91.20	66.34	43.12 *

11. Bond :		B14		B13	B15a	B15	B16	B16a
(a) Support Steel								
Dia.	mm		12	16	12	12	12	
Req. L_d	mm		684	912	684	684	684	
Pro. L_d	mm		1075	912	1075	1075	1075	

(b) Bottom Steel		Left	Right	Left	Right	Left	Right	Left	Right
Dia.	mm	12	12	12	167	12	12	12	12
Req. L_d	mm	684	684	684	912	684	684	684	684
L_{ex}	mm	153	405	405	405	569	569	569	569
A_{st1}	N-#	2-12	2-12	4-12	4-12	3-12	3-12	2-12	2-12
M1	kN.m	30.79	30.79	52.49	52.49	61.99	61.99	43.04	43.04
V	kN	34.35	54.07	87.52	76.42	80.37	100.75	91.17	66.37
Av. L_d	mm	1318	976	1004	1092	1596	1183	1041	1218

12. Deflection :	B14	B13	B15	B16
$p_t = 100A_{st}/(b_f.d)$%	0.08	0.26	0.08	0.07
Allowable L/d	> 26	> 26	> 26	> 26
Actual L/d	< 26	< 26	< 26	< 26

Calculations for Bending Moment and Shear in Beam B13 – B14 :

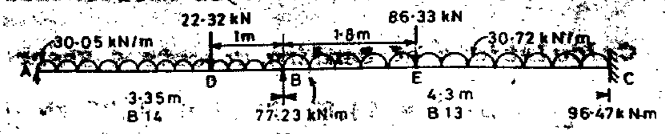


Fig 8.8(a) Loading for calculation of bending moment.

Fixed End Moments :

$M_{FBA} = 30.05 \times 3.35^2/8 + 22.32 \times 1.0 \times 2.35^2/3.35^2 + (22.32 \times 1.0^2 \times 2.35/3.35^2)/2 = 55.48$ kN.m

$M_{FCB} = 30.72 \times 4.3^2/12 + 86.33 \times 2.5 \times 1.8^2/4.3^2 = 85.15$ kN.m

$M_{FBC} = 30.72 \times 4.3^2/12 + 86.33 \times 2.5^2 \times 1.8/4.3^2 = 99.86$ kN.m

Distribution Factors: $d_{BC} = (4/4.3)/(4/4.3 + 3/3.35) = 0.51$, $d_{BA} = 1 - 0.51 = 0.49$

Moment Distribution :

	A	B		C
Distribution Factors		0.49	0.51	
Initial Fixed End Moments	0	55.48	-99.86	85.15
Distributed and Carry over moments		21.75	22.63	11.32
Final Support Moments kN.m	0	77.23	-77.23	96.47

Span Moments :

Span BC : $V_{CB} = 86.33 \times 1.8/4.3 + 30.72 \times 4.3/2 + (96.47 - 77.23)/4.3 = 106.66$ kN

$V_{BC} = 86.33 + 30.72 \times 4.3 - 106.66 = 111.77$ kN

$x_{max} = 106.66/30.72 < 2.5 \; \therefore \; x_{max} = 2.5$ m

$M_{span.max} = 106.66 \times 2.5 - 30.72 \times 2.5^2/2 - 96.47 = 74.18$ kN.m

Span AB : $V_{AB} = 22.32/3.35 + 30.05 \times 3.35/2 - 77.23/3.35 = 33.94$ kN

$V_{BA} = 22.32 + 30.05 \times 3.35 - 33.94 = 89.05$ kN

$x_{max} = 33.94/30.05 = 1.13 \text{ m} < 2.35$ m

$M_{span.max} = 33.94 \times 1.13 - 30.05 \times 1.13^2/2 = 19.17$ kN.m

Redistribution of Moments :	A	Mspan	B	Mspan	C
Design Moments before Redistribution	0	19.17	77.23	74.18	96.47
Redistribution of Moments 10 % at C				4.83	-9.65
20 % at B		+7.72	-15.44	+7.72	
Design Moments after Redistribution	0	26.89	61.79	86.73	86.82

These values have been entered in Step.5.

Shear in Beams :

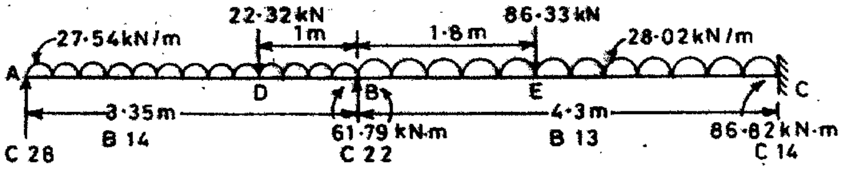


Fig. 8·8(b) Loading for calculation of shear

$V_{AB} = 27.54 \times 3.35/2 + 22.32 \times 1/3.35 - 61.79/3.35 = 34.35$ kN

$V_{BA} = 27.54 \times 3.35/2 + 22.32 \times 2.35/3.35 + 61.79/3.35 = 80.23$ kN

$V_{BC} = 28.02 \times 4.3/2 + 86.33 \times 2.5/4.3 - (86.82 - 61.79)/4.3 = 104.61$ kN

$V_{CB} = 28.02 \times 4.3/2 + 86.33 \times 1.8/4.3 + (86.82 - 61.79)/4.3 = 102.20$ kN

Point of inflection for BA from B is 0.9m, for BC 0.65m & for CB from C is 0.98m

The above values have been entered in Step. 9.

Load on Columns :

Shear shall be reduced to account for the load duplicated for Slab S4. The value of load duplicated from slab S4 = 4.95 kN/m for a length of 2.5m from Column C14

$R_{BC} = V_{BC} - 4.95 \times 2.5 \times 1.25/4.3 = 104.61 - 3.60 = 101.00$ kN

$R_{CB} = V_{CB} - (4.95 \times 2.5 - 3.60) = 102.20 - 8.78 = 93.42$ kN. These values entered in Step 10.

Calculations of Bending Moments & Shear for Beams B15a - B15 - B16 - B16a

$(w_{u2} - w_{u1}) = 43.82-32.57 = 11.25$ kN
For BC , $k = 2.0/4.3 = 0.465$ For CG, $k = 1.75/4.3 = 0.407$

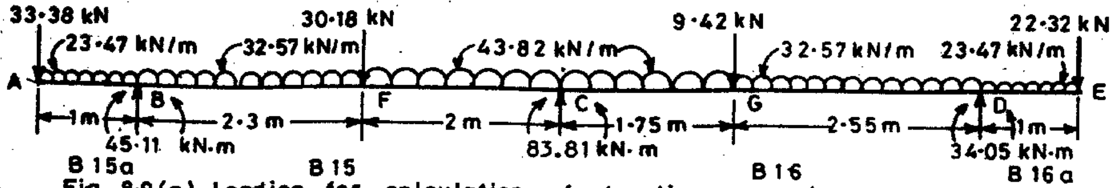


Fig. 8·9 (a) Loading for calculation of bending moment

$M_{FBA} = 33.38x1 + 23.47x1^2/2 = 45.11$ kN.m
$M_{FBC} = 32.57x4.3^2/12 + 11.25x0.465^2(4-3x0.465)\ 4.3^2/12$
$+30.18x2.30x2.00^2\underline{2}/4.3^2 = 69.74$ kN.m
$M_{FCB} = 32.57x4.3^2/12 + 11.25x0.465^2(3x0.465^2 - 8x0.465+6)\ 4.3^2/12$
$+30.18x2.30^2x2.00/4.3^2 = 78.43$ kN.m
$M_{FCD} = 32.57x4.3^2\ /12 + 11.25x0.407^2\ (3x0.407^2 - 8x0.407 + 6)x4.3^2\ /12$
$+9.42x1.75x2.55^2\ /4.3^2 = 65.21$ kN.m
$M_{FDC} = 32.57x4.3^2/12 + 11.25x0.407^2(4 - 3x0.407)x4.3^2/12$
$+ 9.42x1.75^2x2.55/4.3^2 = 57.36$ kN.m
$M_{FDE} = 22.32 + 23.47x1.00^2/2 = 34.05$ kN.m

Moment Distribution :

	A		B		C		D		E
Distribution factors	:	0	:	1 .5	:	.5 1	:	0	:
Initial fixed end moments	:	45.11	:	-69.74 78.43	:	-65.21 57.36	:	-34.05	:
Balancing Cantilever moments	:		:	24.63 12.32	:	-11.66 -23.31	:		:
	:	45.11	:	-45.11 90.75	:	-76.87 34.05	:	-34.05	:
Distributed moments	:		:	-6.94	:	- 6.94	:		:
Final support moments	:	45.11	:	-45.11 83.81	:	-83.81 34.05	:	-34.05	:

Span moments :

$V_{BA} = 33.38 + 23.47 = 56.85$ kN
$V_{BC} = 32.57x4.3/2+30.18X2/4.3+11.25X2.0X1.0/4.3-(83.81-45.11/4.3 = 80.30$ kN
$V_{CB} = 32.57x4.3 + 30.18 + 11.25X2.0 - 80.30 = 112.43$ kN
$V_{CD} = 32.57x4.3/2+9.42x2.55/4.3+11.25x1.75x(4.3-1.75/2)/4.3$
$+(83.81-34.05)/4.3 = 102.87$ kN
$V_{DC} = 32.57x4.3 + 9.42+ 11.25x1.75 - 102.87 = 66.29$ kN
$V_{DE} = 22.32 + 23.47x1 = 45.79$ kN
Span BC: $x_{max} = 80.30/32.57 > 2.3$.'. $x_{max} = 2.30$ m.
$M_{span.max} = 80.30x2.30 - 32.57x2.30^2/2 - 45.11 = 33.41$ kN.m
Span CD: $x_{max} = 66.29/32.57 < 2.55$.'. $x_{max} = 2.03$ m.
$M_{span.max} = 66.29x2.03 - 32.57x2.03^2/2 - 34.05 = 53.41$ kN.m

Redistribution of Moments :

	B		C		D	
Moments before redistribution	45.11	53.41	83.81	33.41	34.05	kN.m
Redistribution at C by 30 %		+ 12.57	-25.14	+12.57		
Design moments after redistribution	45.11	65.98	58.67	45.98	34.05	kN.m

These values have been entered in Step.5.

Shear in Beams and Load on Columns:

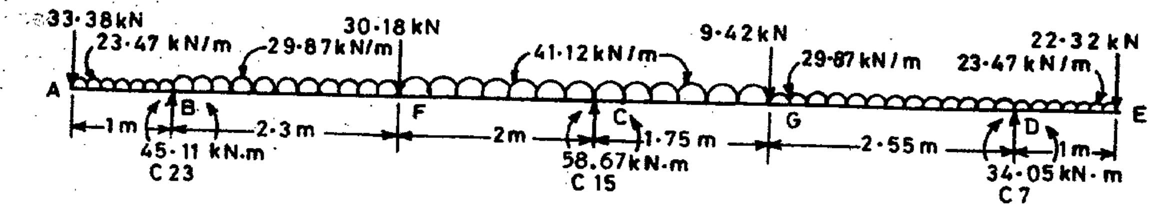


Fig. 8.9(b) Loading for Calculation of shear

	BA	BC	CB	CD	DC	DE
Con.load	33.38	14.04	16.14	5.58	3.84	22.32
UDL	23.47	64.22	64.22	64.22	64.22	23.47
		5.23	17.27	15.68	4.00	
Moment		-3.15	3.15	5.72	-5.72	
Shear in Beam	56.85	80.34	100.78	91.20	66.34	45.79
	-2.67	-	-	-	-	-2.67
Load on Col.	54.18	80.34	100.78	91.20	66.34	43.12
Column No.	C23		C15		C7	

These values of shear in beam and load on column have been entered in Step.9 and 10. respectively.

The details of beams have been shown in Sheet-1 & Sheet-2.

8.5.3 Concluding Remarks

The design of residential building ,which cannot be divided 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 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 obtained for bending moment to find the values of shearing forces,but due to this the load is over-estimated to the extent of 30%. Therefore, it is upto 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 many times governed by the seviceability 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 need not be 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.

8.5.4 Design of Plinth Beams

Category - I : B1,B3,B4,B6
- II : B7,B8,B11
- III : B9,B10,B19,B21
- IV : B13,B14,B15,B16,B15a,B16a

Beams B19 and B21 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

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	B19,21
2. Span L m	3.4	3.4	3.4	3.2	3.4	3.2	3.35	2.5	2.4	2.4
3. Section b mm	225	225	225	225	225	225	225	225	225	225
D mm	375	375	375	375	375	375	375	375	375	300
assumed d mm	340	340	340	340	340	340	340	340	340	265
4. Load wall	23.4	15.5	15.5	23.4	15.5	15.5	23.4	23.4	23.4	-
self	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	2.6
Total	26.6	18.7	18.7	26.6	18.7	18.7	26.6	26.6	26.6	2.6
5. $M_{u.max}$ kN.m	38.44	27.02	27.02	34.05	21.62	19.15	29.05	13.85	13.85	1.25
6. $M_{ur.max}$ kN.m	53.84	53.84	53.84	53.84	53.84	53.84	53.84	53.84	53.84	32.71
7. Main steel:										
Top N-# mm	2-10	2-10	2-10	2-10	2-10	2-10	2-10	2-10	2-10	2-8
Bot bt N-# mm	1-12	1-12	1-12	1-10	1-8	1-8	1-12	-	-	-
Bot st N-# mm	2-12	2-10	2-10	2-12	2-10	2-10	2-10	2-10	2-10	2-10
Prov A_{st} mm²	339	235	235	304	207	207	269	157	157	100
Prov d	344	345	345	344	345	345	344	345	345	270
Prov.M_{ur} kN.m	37.00	30.29	30.29	33.65	23.88	23.88	30.20	18.46	18.46	14.45
8. Shear : (a)S.S end										
$V_{u.max}$ kN	45.22	31.79	31.79	42.56	28.61	26.93	40.10			
A_{st1} N-# mm	2-12	2-10	2-10	2-12	2-10	2-10	2-10			
V_{uc} kN	28.99	24.90	24.90	28.99	24.90	24.90	24.90			
$V_{usv.min}$ kN	27.09	27.17	27.17	27.09	27.17	27.17	27.09			
$V_{ur.min}$ kN	56.08	52.07	52.07	56.08	52.07	52.07	51.99			
Stirrups ϕ-s	ϕ 6 at 150									
(b)Continuous End										
$V_{u.max}$ kN					38.15	35.90	53.47	33.25	31.92	3.12
A_{st1} N-# mm					2-10	2-10	2-10	2-10	2-10	2-10
					+1-8	+1-8	+1-12			
V_{uc} kN					28.00	28.00	31.16	24.90	24.90	24.90
$V_{usv.min}$ kN					27.17	27.17	27.09	27.17	27.17	21.26
$V_{ur.min}$ kN					55.17	55.17	58.25	51.99	51.99	46.16
Stirrups ϕ-s					ϕ 6 at 150					
9. Bond :										
(a)At S.S.End or Point of Inflexion (P.I) : Bottom Bars										
Bar Diam. mm	12	10	10	12	10	10	10	10	10	10
Req. L_d mm	684	570	570	684	570	570	570	570	570	570
L_{ex} mm	113	75	75	113	75	75	75	75	75	75
A_{st1} N-# mm	2-12	2-10	2-10	2-10	2-10	2-10	2-10	2-10	2-10	2-10
$M_1 = M_{ur}$ kN.m	24.67	17.93	17.93	25.02	18.11	18.11	17.63	18.46	18.46	14.45
V kN	45.05	31.79	31.79	42.40	28.61	26.93	39.95	*19.88	19.08	1.87
Avail. L_d mm	825	808	808	880	898	898	664	1003	1042	7802
(b)Continuous End - Top Bars							* at P.I. V=0.6 $V_{u.max}$			
Bar Diam. mm					10	10	10	10	10	10
Req. L_d mm					570	570	570	570	570	570
Avai.L_d mm					850	800	837	625	600	600

Beam mark	B1	B3	B6	B4	B7	B8	B11	B9	B10	B19,20
10. Deflection										
Midspan p_t %	0.44	0.30	0.30	0.39	0.27	0.27	0.35	0.20	0.20	0.16
Allow. L/d	> 20	> 20	> 20	> 20	> 26	> 26	> 26	> 26	> 26	> 26
Actual L/d	< 20	< 20	< 20	< 20	< 26	< 26	< 26	< 26	< 26	< 26

Explanatory Notes to Step Nos. in above Design :

(4) In case of beams B1, B4, B11, B9,and B10, the load due to wall comprises of ground floor wallof solid concrete blocks of width 150mm(175mm with plaster) and plinth wallof brick of thickness 225 mm (250 mm including plaster) for a height of 450mm of plinth giving load of 20 + 3.4 = 23.4 kN/m .See Sect.8.3

Design of Plinth Beams : Category – V

1. Beam Mark :	B14	B13	B15a	B15 & B16
2. Span	3.35	4.3	1.00	4.3
3. Section: Width b mm	225	225	225	225
Depth D mm	375	375	375	375
Assumed d mm	344	344	344	344
4. Loads :				
Wall kN/m	15.5	15.5	23.40	23.40
Self kN/m	3.2	3.2	3.2	3.2
Total kN/m	18.7	18.7	26.60	26.60
Point load from :	–	B6	–	–
Value kN	–	31.79	–	–
Distance from nearest column.		1.8	–	–

5. $M_{u.max}$ at	Midsp	Intsp	Midsp	Right	Left	Midsp	Intsp	Midsp	Right
Value kN.m.	11.00	36.97	37.67	38.67	13.30	36.28	38.39	36.28	13.30
6. $M_{ur.limit}$.(=1.8bd²)	47.93	47.93	47.93	47.93	37.81	37.81	37.81	37.81	37.81
7. Main Steel :									
Top st. N-#	2-10	2-10	2-10	2-10	2-10	2-10	2-10	2-10	2-10
	–	*2-12	–	*2-12	–	–	2-12	–	–
BOt. bent N-#	*1-12	–	*1-12	–	–	@2-12	–	2-12	–
Bot st. N-#	2-10	2-10	2-10	2-10	2-10	2-10	2-10	2-10	2-10
A_{st} Provided mm²	270	383	270	383	157	383	383	383	157
Effd. Provided mm	345	344	344	344	345	344	344	344	345
M_{ur} Provided kN.m.	18.46	41.06	41.06	41.06		41.06	41.06	41.06	

8. Shear	B14		B13		B15a		B15		B16	
	Left	Right	Left	Right	Right	Left	Left	Right	Left	Right
(a) $V_{u.max}$ kN	20.29	42.36	58.29	53.91	26.6	51.36	63.02	63.02	51.36	26.6
(b) A_{st1} N-#	2-10	..2-10 +2-12 ...			2-10		2-10+2-12		2-10	
(c) V_{uc} kN	24.90	35.74		35.74	24.90		35.74		24.90	
(d) $V_{usv.min}$ kN	27.09	27.09		27.09	27.09		27.09		27.09	
(e) $V_{ur.min}$ kN	51.99	62.83		62.83	51.99		62.83		62.83	
(f) ø-S	6-150	6-150		6-150	6-150		6-150		6-150	

Calculation of Bending Moment and Shear for Beam B13-B14

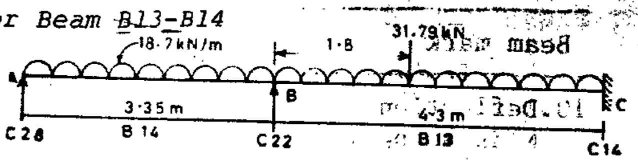


Fig. 8.10 Plinth Beam

Fixed End Moments:

M_{FBA} = $18.7 \times 3.32^2/8 = 26.23$ kN.m.
M_{FBC} = $18.7 \times 4.3^2/12 + 31.79 \times 1.8 \times 2.5^2/4.3 = 48.16$ kN.m.
M_{FCB} = $18.7 \times 4.3^2/12 + 31.79 \times 1.8^2 \times 2.5/4.3 = 42.74$ kN.m.

Distribution factors: $d_{BA} = (0.75/3.35)/(0.75/3.35 + 1/4.3) = 0.49$
$d_{BC} = 1 - 0.49 = 0.51$

Moment Distribution:

	A	B		C
Distribution factors		0.49	0.51	
Fixed End Moments	0	26.23	-48.16	42.74
Distributed & Carry-over moments		10.74	11.19	5.60
Final Moments.	0	36.97	-36.97	48.34

Span Moments:

Span AB: V_{AB} = $18.7 \times 3.35/2 - 36.97/3.35 = 20.29$ kN
V_{BA} = $18.7 \times 3.35 - 20.29 = 42.36$ kN
x_{max} = $20.29/18.7 = 1.085$ m.
$M_{span.max}$ = $20.29 \times 1.085^2/2 = 11.00$ kN.m.
V_{CB} = $18.7 \times 4.3/2 + 31.79 \times 1.8/4.3 + (48.34 - 36.97)/4.3 = 56.16$ kN
V_{BC} = $18.7 \times 4.3 + 31.79 - 56.16 = 56.04$ kN.
x_{max} = $56.04/18.7 > 2.5$ Hence $x_{max} = 2.5$ m.
$M_{span.max}$ = $56.16 \times 2.5 - 18.7 \times 2.5^2/2 - 48.34 = 33.62$ kN.m.

Redistribution of Moments:

	A	Mspan	B	Mspan	C
Design moments before redistribution	0	11.00	36.97	33.62	48.34
Redisreibution of moment 20% at C	-	-	-	4.83	-9.67
Design Moments after redistribution	0	11.00	36.97	@ 37.67	38.67

These values have been entered in Step. 5.

Revised calculations for Shear and span Moments

V_{BC} = $56.04 + 9.67/4.3 = 58.29$ kN
V_{CB} = $56.16 - 9.67/4.3 = 53.91$ kN
$x_{max} = 2.5$
$M_{span.max}$: = $53.91 \times 2.5 - 18.7 \times 2.5^2/2 - 38.67$ = @ 37.67 kN.m.
Values of Shear have been entered in Step.8.

Calculation of Bending Moments and Shears in Beam B15a-B15-B16-B16a :

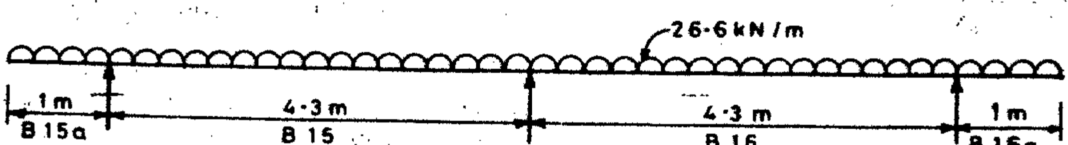


Fig. 8.11 Load on Plinth beam

Fixed End Moments:

M_{FBA} = $26.6 \times 1^2/2$ = 13.3 kN.m M_{FBC} = $26.6 \times 4.3^2/12 = 40.99$ kN.m.

Since the two span beam has support symmetry the moment distribution will be carried out assuming C fixed.

Beam

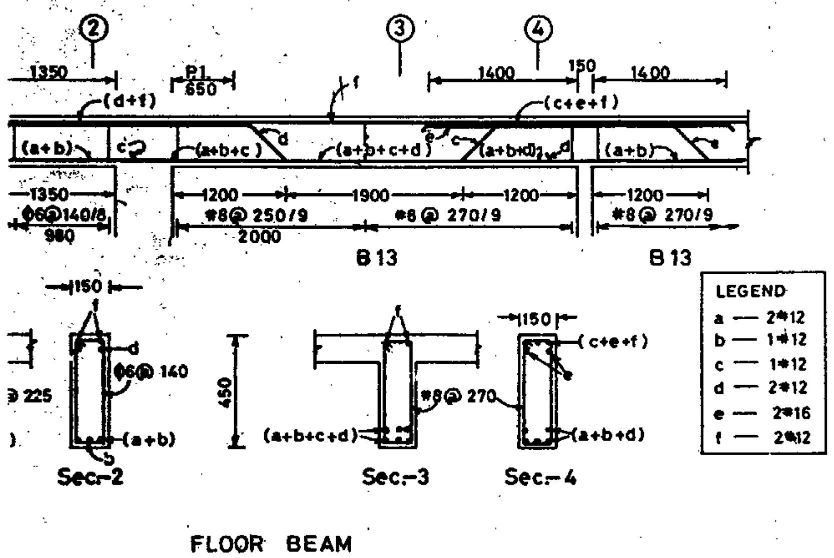
C

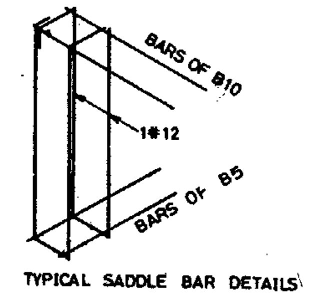
42.74:
5.60:
48.34:

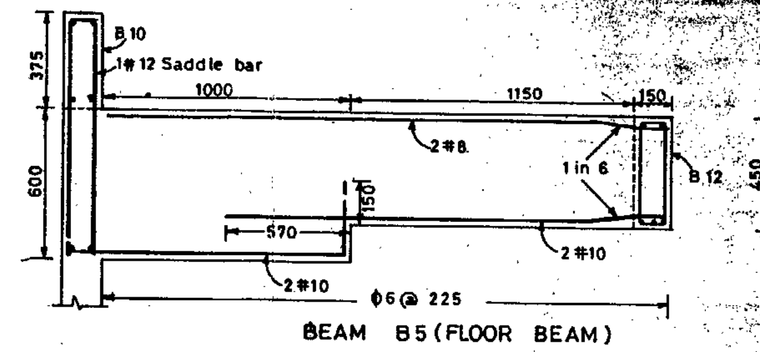
C
48.34
-9.67
38.67

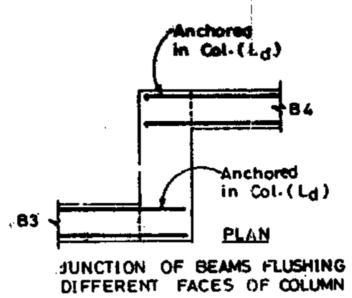
.m.

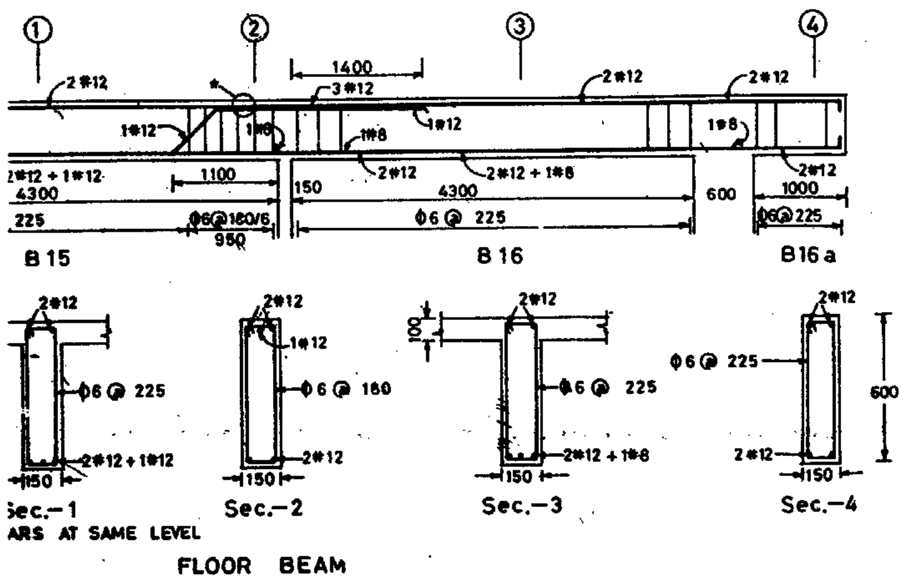
will be

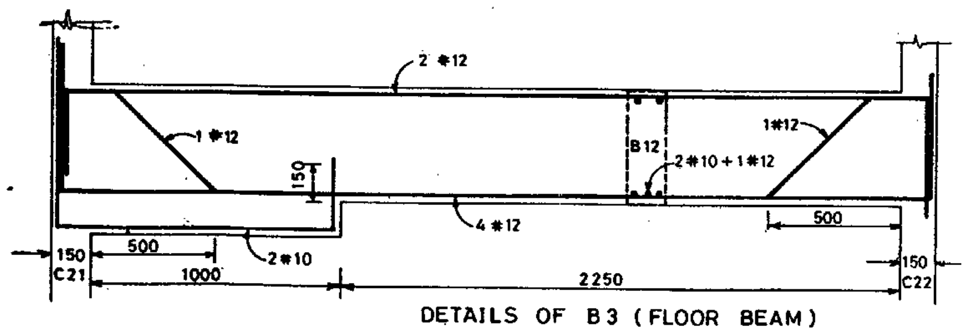












Schedule of Beams

(a) Roof Beams

Beam Mark	Span m	Section b x D mm	Bottom bars St. N #	Bottom bars Bt. N #	Top N #	Extra Bars Left N #	Extra Bars Right N #	Stirrups Minimum O S	Stirrups Designed Left O s /N	Stirrups Designed Right O s /N
B1	3.4	150x300	2-10	1-8	2-10			6-225		
B2	3.2	150x300	2-10	1-8	2-10			6-225		
B2a	3.2	150x300	2-10	-	2-10			6-225		
B3&B6	3.4	150x300	2-16	-	2-10			6-225		
B4	3.2	150x300	2-12	1-10	2-8			6-225		
B7	3.4	150x300	2-12	1-12	2-12			6-225		6-180/4
B8	3.2	150x300	2-12	1-12	2-10			6-225		
B9	2.5	150x300	2-10	-	2-10			6-225		
B10	2.4	150x300	2-10	-	2-10			6-225		
B11	3.3	150x300	2-10	1-8	2-10			6-225		
B13	4.3	150x375	2-12+2-10	-	2-10	2-12	3-12	6-225		
B14	3.3	150x375	2-10	-	2-10		2-12	6-225		
B15a	1.6	150x600	2-12	-	2-10		1-12	6-225		
B15	4.3	150x600	2-12+1-12	-	2-10	1-12	2-12	6-225	6-180/5	6-180/6
B16	4.3	150x600	2-12	-	2-10	2-12	-	6-225		
B16a	1.6	150x600	2-12	-	2-10			6-225		
B17	2.4	150x300	2-10	-	2-10			6-225		
B18	2.4	150x300	3-12	-	2-10	-	-	6-225		
B19	2.4	150x300	3-12	-	2-10			6-225		
B20	2.4	150x300	2-10	-	2-10			6-225		

(b) Floor Beams

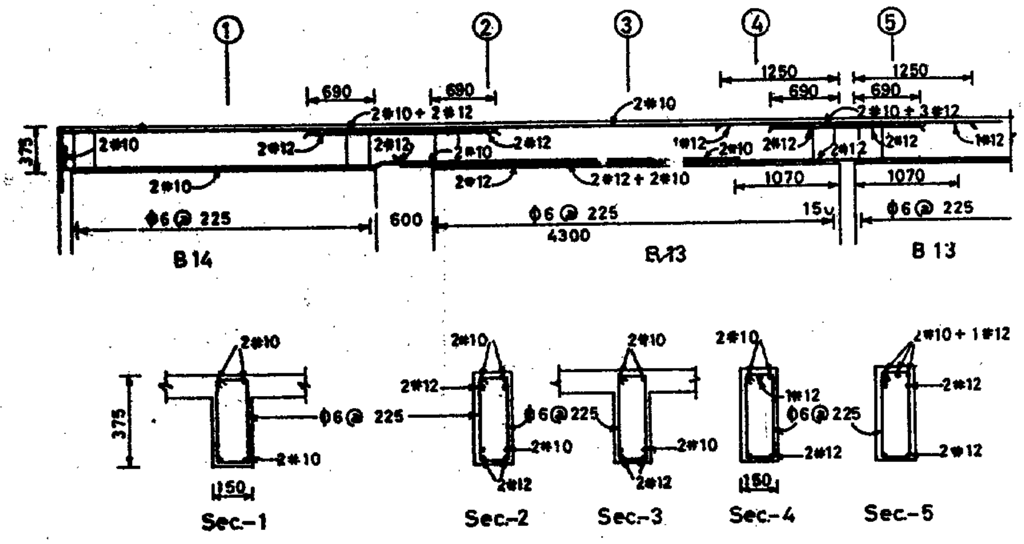
Beam Mark	Span m	Section b x D mm	Bottom bars St. N #	Bottom bars Bt. N #	Top N #	Extra Bars Left N #	Extra Bars Right N #	Stirrups Minimum O S	Stirrups Designed Left O s /N	Stirrups Designed Right O s /N
B1	3.4	150x375	2-12	2-10	2-10			6-225		
B2	3.2	150x375	2-10	-	2-10			6-225		
B2a	3.2	150x375	2-10	-	2-10			6-225		
B3	3.4	150x450/600	3-12	2-12	2-12			6-225		6-140/3
B4	3.2	150x375	3-12	1-12	2-8			6-225		
B5	2.3	150x450/600	2-10	-	2-8			6-225		
B6	3.4	150x450	4-12	1-12	2-12			6-225	6-210/4	6-140/8
B7	3.4	150x375	2-12	2-12	2-12			6-225		6-110/10
B8	3.2	150x375	2-12	2-10	2-12			6-225		
B9	2.5	150x375	2-10	-	2-10			6-225		
B10	2.4	150x375	2-12	2-10	2-10			6-225		
B11	3.35	150x375	2-12	2-10	2-10			6-225		
B12	2.4	150x450	2-10	1-12	2-10			6-225		
B13	4.3	150x450	4-12	2-12	2-12		2-16	6-225	8-250/9	8-270/9
B14	3.35	150x450	2-12	-	2-12	-	3-12	6-225		6-140/8
B15a	1.6	150x600	2-12	-	2-12		1-8	6-225		
B15	4.3	150x600	2-12	1-12	2-12	1-8		6-225		6-180/6
B16	4.3	150x600	2-12+1-8	-	2-12	1-12	-	6-225		
B16a	1.6	150x600	2-12	-	2-12			6-225		
B17	2.4	150x300	2-10	-	2-10			6-225		
B18	2.4	150x375	2-8	1-8	2-8		-	6-225		
B19	2.4	150x375	2-12	1-8	2-10			6-225		
B20	2.4	150x375	2-10	-	2-10			6-225		

(c) Plinth Beams

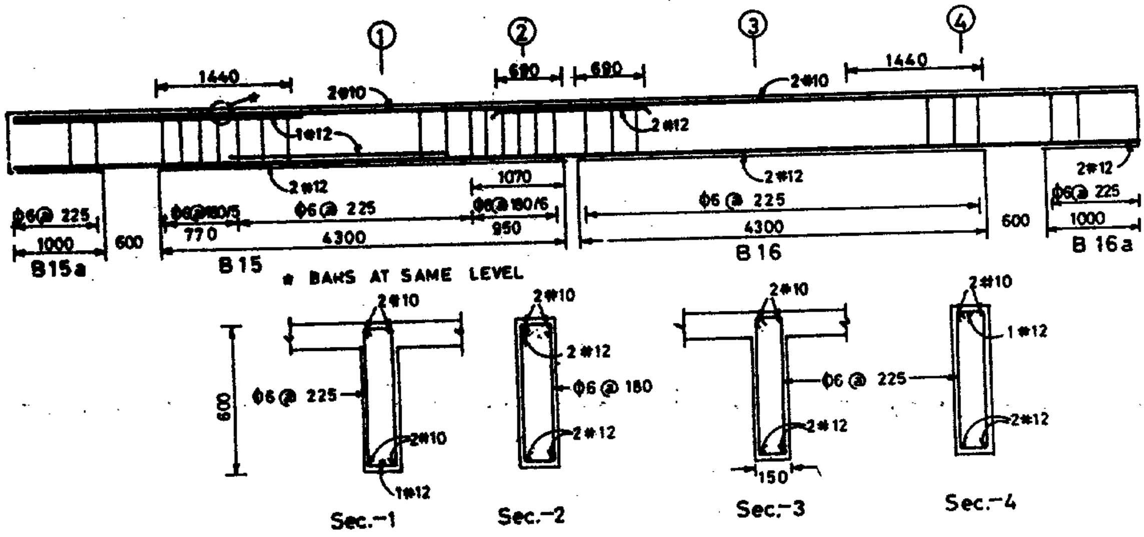
Beam Mark	Span m	Section b x D mm	Bottom bars St. N #	Bottom bars Bt. N #	Top N #	Extra Bars Left N #	Extra Bars Right N #	Stirrups Minimum O S	Stirrups Designed Left O s /N	Stirrups Designed Right O s /N
B1	3.4	225x375	2-12	1-12	2-10			6-150		
B3	3.4	225x375	2-10	1-12	2-10			6-150		
B4	3.2	225x375	2-12	1-10	2-10			6-150		
B6	3.4	225x375	2-10	1-12	2-10			6-150		
B7	3.4	225x375	2-10	1-8	2-10			6-150		
B8	3.2	225x375	2-10	1-8	2-10			6-150		
B9	2.5	225x375	2-10	-	2-10			6-150		
B10	2.4	225x375	2-10	-	2-10			6-150		
B11	3.35	225x375	2-10	1-12	2-10			6-150		
B13	4.3	225x375	2-10	1-12	2-10			6-150		
B14	3.35	225x375	2-10	1-12	2-10			6-150		
B15a	1.6	225x375	2-10	-	2-10			6-150		
B15	4.3	225x375	2-10	2-12	2-10			6-150		
B16a	1.6	225x375	2-10	-	2-10			6-150		
B16	4.3	225x375	2-10	2-12	2-10			6-150		
B19	2.4	225x300	2-10	-	2-8			6-150		
B21	2.4	225x300	2-10	-	2-8			6-150		

Notes :
- Concrete Mix M15
- Clear cover for slab Rein. 15mm, Beam 25mm
- Clear cover for column 40mm, Footing 50mm
- # Represents Tor Steel Fe 415 & represents Mild Steel Fe 250
- First stirrup in beam shall be 50mm away from the face of support

	INSTITUTE OF STRUCTURAL ENGINEERING 36 Parvati, Pune - 411 009.		
Job.No.	PROJECT — 3 RESIDENTIAL BUILDING		
Drg.No 2	Date / /	Drawn by	
	Scale as shown	Checked by	Structural Engineer



ROOF BEAM



ROOF BEAM

Moment Distribution :

	A	B	C
Fixed end Moments	0	13.30: -40.99	40.99
Distributed abd C.O.moments		: 27.69	13.85
Final support moments	0	13.30: -13.30	54.84

Redistribution of Moments:

	B	Mspan	C
Design moments before redistribution	13.30		54.84
Redistribution of moments 20% at C			-16.45
Design moments after redistribution.	13.30	@ 36.28	38.39

These values have been entered in Step. 5.

Calculations for Shear and Span moments:

V_{BA} = 26.6x1 =26.6 kN

V_{BC} = 26.6x4.3/2-*(38.39-13.30)/4.3 = 51.36 kN

V_{CB} = 26.6x4.3-51.36 = 63.02 kN

x_{max} = 51.36/26.6 = 1.931 m. from B

$M_{span.max}$ = 51.36x1.931/2 - 13.3 = @ 36.28 kN.m.

8.6 DESIGN OF COLUMNS

8.6.1 Categorisation of Columns

Category - I : Axially Loaded Columns : C14

Category - II : Columns subjected to axial load and uniaxial bending : C7, C13, C15, C19, C21, C22 and C23.

Category - III : Columns under axial load and biaxial bending : C27, C28.

8.6.2 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 algebric 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.8.11. The beam directions are taken as per adjacent figure. The load on each column is calculated as detailed below.

```
    1
    |
4---C---2
    |
    3
```

COLUMNS IN TOP STOREY

Loads transferred by Roof Beams

Category	Column Mark	Beam End Forces in kN from directions 1	2	3	4	Total kN	Tank Load kN	Total Rounded Pr kN
I	C14	40.86	29.76	40.86	48.14	159.62	-	160
II	C 7	27.48	-	29.48	28.64	85.60	-	86
	C13	5.13	36.11	5.13	-	46.37	-	47
	C15	58.37	-	87.20	22.32	167.89	105	268
	C19	5.13	40.12	4.92	-	50.17	-	52
	C21	4.92	36.60	23.40	-	64.92	-	65
	C22	44.72	28.64	46.61	36.60	156.57	-	157
	C23	89.23	49.78	58.49	28.64	226.14	105	327
III	C27	17.55	18.67	-	-	36.22	-	37
	C28	15.40	-	-	18.67	34.07	-	37

a) Values without parenthesis represent loads transferred by roof beams.
b) Values in parenthesis represent loads transferred by Floor beams.
c) Values in rectangular brackets represent loads transferred by Plinth beams.

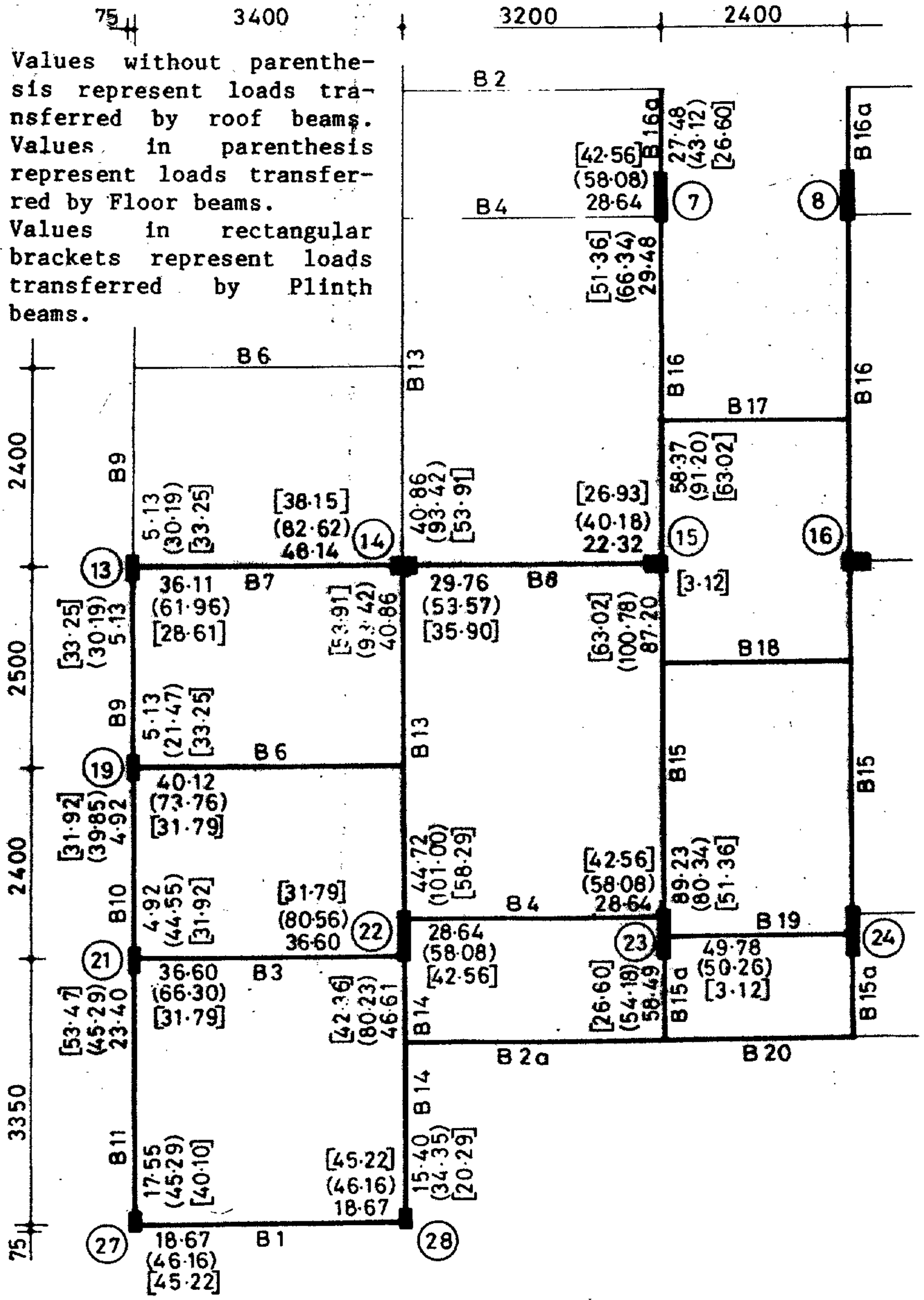


Fig. 8.11 Plan showing Beam end forces transferred to column

The floor load transferred from each beam is obtained from Sr.No.10 of design of floor beam and is shown in Fig.8.11. The load on each column is calculated as detailed below.

COLUMNS IN INTERMEDIATE STOREY
Loads transferred by Floor Beams

Category	Column Mark	Beam End Shears in kN from directions 1	2	3	4	Total kN	Total Rounded P_f kN
I	C14	93.42	53.57	93.42	82.62	323.03	324
II	C 7	43.12	-	66.34	58.08	167.54	168
	C13	30.19	61.96	30.19	-	122.34	123
	C15	91.20	-	100.78	40.18	232.16	233
	C19	21.47	73.76	39.85	-	135.08	135
	C21	44.55	66.30	45.29	-	156.14	157
	C22	101.00	58.08	80.23	80.56	319.87	320
	C23	80.34	50.26	54.18	58.08	242.86	243
III	C27	45.29	46.16	-	-	91.45	92
	C28	34.35	-	-	46.16	80.51	81

On the same lines the loads transferred from plinth beams is obtained from beam end shears as per Sr. No. 8 of design of plinth beams and is shown in Fig. 8.11(c). The loads on each column is calculated as detailed below.

COLUMNS IN FIRST STOREY
Loads transferred by Plinth Beams

Category	Column Mark	Beam End shears in kN from directions 1	2	3	4	Total kN	Total Rounded P_f kN
I	C14	53.91	35.90	53.91	38.15	181.87	182
II	C 7	26.60	-	51.36	42.56	120.52	121
	C13	33.25	28.61	33.25	-	95.11	95
	C15	63.02	3.12	63.02	26.93	156.09	156
	C19	33.25	31.79	31.92	-	96.96	97
	C21	31.92	31.79	53.47	-	117.18	118
	C22	58.29	42.56	42.36	31.79	175.00	175
	C23	51.36	3.12	26.60	42.56	123.64	124
III	C27	40.10	45.22	-	-	85.32	86
	C28	20.29	-	-	45.22	65.51	66

8.6.3 Determination of Effective Length and Slenderness :

The unsupported lengths of the columns is first computed from the floor to floor height.

Floor to floor height = 3000 mm
Depth of shallowest beam = 300 mm
Unsupported Length L = 2700 mm.

The unsupported lengths for top storey and middle storey columns will be the same, but for bottom storey it 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 B1 and B2 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 & 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 150mm or 225mm the column type is decided.

The allowance for slenderness has been approximately arrived at from reduction coefficient used in working stress method.

$$\text{Reduction coefficient } C_r = 1.25 - L_{eff}/(48b)$$
$$= 1.25 - (L_{eff}/b)/48$$
$$\text{Allowance for slenderness in \%} = (1/C_r - 1) \times 100$$

The allowances for different columns have been worked out in the following table.

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	300	300
Unsupported Length	L	mm	2700	2700	3150
L_{eff}			2700	2160	2520
Width of column	b	mm	150/225	150/225	150/225
L_{eff}/b			18/12	14.4/9.6	16.8/11.2
Column type			Long/Short	Long/Short	Long/Short
Allowance for slenderness		%	15 / Nil	5 / Nil	11 / Nil

To account for the effect of slenderness these allowances have been taken in obtaining the equivalent loads on long columns.

8.6.4 Calculation of Column Loads in Each Storey :

Loads obtained in each storey in Sec. 8.6.2 are complied and loads at each storey level are worked out.

(A) Loads in each Storey in kN.

Storey	Category									
	I	II							III	
	C14	C7	C13	C15	C19	C21	C22	C23	C27	C28
Top Storey P_r kN	160	86	47	268	52	65	157	327	37	37
Int. Storey P_f kN	324	168	123	233	135	157	320	243	92	81
1st Storey P_p kN	182	121	95	156	97	118	175	124	86	66

b) Loads at each storey LEVEL in kN.

Group		C14 I	C7 IIa	C13 IIb	C15 IIc	C19 IIb	C21 IIb	C22 IId	C23 IIe	C27 III	C28 III
Top Storey	P_r kN:	160	86	47	268	52	65	157	327	37	37
3rd Storey	P_r+P_f kN:	484	254	170	501	187	222	477	570	129	118
2nd Storey	P_r+2P_f kN:	808	422	293	734	322	379	797	813	221	199
1st Storey	P_r+3P_f kN:	1132	590	416	967	457	536	1117	1056	313	280
Plinth	$P_r+3P_f+P_p$ kN:	1314	711	511	1123	554	654	1292	1180	399	346

8.6.5 Calculation of Equivalent Design Axial Load and Design of Column Section :

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 uni-axial bending the allowance is taken equal to 15% and for bi-axial bending 33% approximately. The allowance for slenderness is taken depending on the ratio of Leff/b as detailed in Sec. 8.6.3 The tentative column section is selected referring to Table. E-1 in Appendix.

Computation of Equivalent Axial Column Loads.

	C14	C15	C13,C19&C21	C22	C23	C 7	C27&C28
(a) Column in storey R-3							
P_r kN	160	268	65	157	327	86	37
% Allowance for fixity	-	15%	15%	15%	15%	15%	33%
Allowance for fixity kN		40	10	23	49	13	12
Total	160	308	75	180	376	99	49
Assumed b mm	150	225	150	150	150	150	150
% Slenderness allowance	15%	-	15%	15%	15%	15%	15%
Value in kN	24	46	11	27	56	15	7
Equivalent Axial Design Load kN	184	354	86	207	432	114	56
Section bXD in mm	225x150	225x225	150x225	150x600	150x600	150x600	150x225
N - #	4-12	4-12	4-12	8-12	4-16	8-12	4-12
ϕ - S	6-150	6-190	6-150	6-150	6-150	6-150	6-150
P_{ur} provided kN	227	383	227	550	531	550	227
(b) Column in storey 3-2							
P_f	324	233	157	320	243	168	92
(P_r+P_f) kN	484	501	222	477	570	254	129
% Allowance for fixity: 0/15/33/on P_f kN	-	35	24	48	36	25	30
Total kN	484	536	246	525	606	279	159
Assumed b mm	225	225	150	150	150	150	150
Allowance for Slenderness @ 5% kN	-	-	12	26	30	14	8x2
Equivalent Axial Design Load kN	484	536	258	551	636	293	175
Section bxD in mm	225x300	225x300	150x300	150x600	150x600	150x600	150x225
N - #	6-12	6-12	4-12	8-12	6-16+2-12	8-12	4-12
ϕ - s	6-190	6-190	6-150	6-150	6-150	6-150	6-150
Provided P_{ur} kN	530	530	275	550	651	550	227

		C14	C15	C13;C19&C21	C22	C23	C 7	C27&C28
(c) Columns in storey 2-1:								
Load on Column (P_r+2P_f)	kN	808	734	379	797	813	422	221
Allowance for fixity on P_f same as in(b)	kN	-	35	24	48	36	25	30
@ Total Load	kN	808	769	403	845	849	447	251
Assumed b	mm	225	225	150	225	225	150	150
Allowance for Slenderness	kN	-	-	20	-	-	22	13x2
Equivalent Axial Design Load	kN	808	769	423	845	849	469	277
Section bxD	mm	225x450	225x450	150x450	225x600	225x600	150x600	150x225
N - #		(4-16+2-12)	8-12	4-16	4-16+4-12	(6-16+2-12)	8-12	6-12
ø - S		6-190	6-190	6-150	6-190	6-190	6-150	6-150
Provided P_{ur}	kN	798	767	436	1034	1079	550	270
(d) Column in storey 1-PL								
Load on columns ($P_r + 3P_f$)	kN	1132	967	536	1117	1056	590	313
Allowance for fixity on P_f	kN	-	35	24	48	36	25	30
Total Load	kN	1132	1002	560	1165	1092	615	343
Assumed b	mm	225	225	150	225	225	150	150
Allowance for slenderness @11%	kN	-	-	62	-	-	67	38x2
Equivalent Axial Load		1132	1002	622	1165	1092	682	419
Section bxD	mm	225x600	225x600	225x450	225x600	225x600	150x600	225x225
N - #		8-16	4-16 +4-12	4-16 +2-12	8-16 +2-12	8-16	8-16	6-12
ø - S		6-225	6-190	6-190	6-190	6-225	6-190	6-190
Provided P_{ur}	kN	1122	1034	798	1178	1122	684	439
(e) Column in PL - FT								
P_p	kN	182	156	118	175	124	121	86
Load on columns $P_r+3P_f+P_p$	kN	1314	1123	654	1292	1180	711	399
Allowance for fixity 15%	kN	-	23	18	26	19	18	13x2
$ Total Load	kN	1314	1146	672	1318	1199	729	425
Section bxD	mm	225x675	225x675	225x450	225x675	225x675	225x600	225x225
N - #		10-16	4-16 +4-12	4-16 +2-12	10-16	8-16	4-16 +4-12	8-12
ø - S		6-225	6-190	6-190	6-225	6-225	6-190	6-190
Provided P_{ur}	kN	1312	1127	798	1312	1213	1024	494

@ 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 about 700mm only the self weight of the column is neglected.

8.6.6 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 $w_uL^2/24$ at their discontinuous end of the beam and $w_uL^2/12$ 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 columns and beams in proportion to their distribution factors.

(c) These moments are compaired with minimum moments (P_u x e_{min}) and the maximum value is taken. Finally the column is checked for axial load and the moments thus obtained.

The stepwise calculations performed are as under:

(A) Calculation of storeywise stiffnesses of columns.
(B) Computations of floorwise stiffnesses of beams.
(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.

(A) Storeywise Stiffness of Column :

Category Column Mark			III C27	II(a) C 7	II(c) C15	I C14
Storey:						
a) R-3	:Section bxD	mm	150x225	150x600	225x225	225x150
	L_{cc}	mm	3000	3000	3000	3000
	I_x x 10^6	mm^4	142.4	2700	213.6	63.3
	K_{R3x} x 10^3	mm^3	47.5	900.0	71.2	21.1
	I_y x 10^6	mm^4	63.3	168.75	213.6	142.4
	K_{R3y} x 10^3	mm^3	21.1	56.25	71.2	47.5
b) 3-2	: Section bxD	mm	150x225	150x600	225x300	225x300
	L_{cc}	mm	3000	3000	3000	3000
	I_x x 10^6	mm^4	142.4	2700	506.25	506.25
	K_{32x} x 10^3	mm^3	47.5	900	168.75	168.75
	I_y x 10^6	mm^4	63.3	168.75	284.76	284.76
	K_{32y} x 10^3	mm^3	21.1	56.25	94.92	94.92
c) 2-1	: Section bxD	mm	150x225	150x600	225x450	225x450
	L_{cc}	mm	3000	3000	3000	3000
	I_x x 10^6	mm^4	142.4	2700	1708.6	1708.6
	K_{21x} x 10^3	mm^3	47.5	900	569.5	569.5
	I_y x 10^6	mm^4	63.3	168.75	427.1	427.1
	K_{21y} x 10^3	mm^3	21.1	56.25	142.4	142.4
d) 1-PL:	Section bxD	mm	225x225	150x600	225x600	225x600
	L_{cc}	mm	3450	3450	3450	3450
	I_x x 10^6	mm^4	213.57	2700	4050	4050
	K_{1Px} x 10^3	mm^3	61.90	782.6	1173.9	1173.9
	I_y x 10^6	mm^4	213.57	168.75	569.5	569.5
	K_{1Py} x 10^3	mm^3	61.90	48.90	165.1	165.1
e) PL-FT	Section bxD	mm	225x225	225x600	225x675	225x675
	L_{cc}	mm	1000	1000	1000	1000
	I_x x 10^6	mm^4	213.57	4050	5766.5	5766.5
	K_{PFx} x 10^3	mm^3	213.57	4050	5766.5	5766.5
	I_y x 10^6	mm^4	213.57	569	640.7	640.7
	K_{PFy} x 10^3	mm^3	213.57	569	640.7	640.7

(B) Floorwise Stiffnesses of Beams : (i) Bending @ x-axis

		Left	Right	Left	Right	Left	Right	Left	Right
(a) Roof									
Beam Mark		B11	-	B16	B16a	-	B8	B7	B8
L	mm	3.35	-	4.3	1.0	-	3.2	3.4	3.2
Section bxD	mm	150x300	-	150x600	150x600	-	150x300	150x300	150x300
Section Type		FL		FL	FL		FL	FL	FL
$I_b \times 10^6$	mm^4	675	-	5400	2700		675	675	675
$K_b \times 10^3$	mm^3	201.5	-	1255.8	0	-	210.9	198.5	210.9
w_u	kN/m	14.15	-	-	-	-	20.3	23.6	20.3
M_{F1}, M_{F2}	kN.m	6.62	-	32.98	25.92	-	8.66	22.73	17.32
M_e	kN.m	6.62		7.06		8.66		5.41	
$\Sigma K_b \times 10^3$	mm^3	201.5		1255.8		210.9		409.4	
(b) 3rd Floor Sec.									
b x D	mm	150x375	-	150x600	150x600	-	150x375	150x375	150x375
(c) 2nd Floor Sec. Type		FL	-	FL	R	-	FL	FL	FL
(d) 1st Floor									
$I_b \times 10^6$	mm^4	1318.4	-	5400	2700	-	1318.4	1318.4	1318.4
$K_b \times 10^3$	mm^3	393.5		1255.8	0	-	412.0	387.8	412.0
w_u	kN/m	29.55	-	32.70	-	40.50	32.70		
M_{F1}, M_{F2}	kN.m	13.82	-	57.36	34.05	13.95	-	39.02	27.90
M_e	kN.m	13.82		23.31		13.95		11.12	
$\Sigma K_b \times 10^3$	mm^3	393.5		1255.8		412.0		799.8	
(e) Plinth Section		B11		B16	B16a		B8	B7	B8
b x D	mm	225x375	-	225x375	225x375	-	225x375	225x375	225x375
Sec. Type		R		R	R		R	R	R
$I_b \times 10^6$	mm^4	988.8	-	988.8	988.8	-	988.8	988.8	988.8
$K_b \times 10^3$	mm^3	295.2	-	229.9	0	-	309.0	290.8	309.0
w_u	kN/m	26.6	-	26.6	26.6	-	18.7	18.7	18.7
M_{F1}, M_{F2}	kN.m	12.44	-	40.99	13.3	-	7.98	18.01	15.96
M_e	kN.m	12.44		27.69		7.98		2.05	
$\Sigma K_b \times 10^3$	mm^3	295.2		229.9		309.0		599.8	

(ii) Bending @ Y - Axis

		Left	Right	Left	Right	Left	Right	Left	Right
(a) Roof									
Beam Mark		-	B1	B4	-	B15	B16	B13	B13
L	mm	-	3.4	3.2	-	4.3	4.3	4.3	4.3
Section bxD	mm		150x300	150x300	-	150x600	150x600	150x375	150x375
Section Type		-	FL	FL	-	FL	FL	FL	FL
$I_b \times 10^6$	mm^4	-	675	675	-	5400	5400	1318.4	1318.4
$K_b \times 10^3$	mm^3	-	198.5	210.9	-	1255.8	1255.8	306.6	306.6
w_u	kN/m	-	13.28	20.30	-	-	-	-	-
M_{F1}, M_{F2}	kN.m	-	6.40	8.66	-	72.39	36.69	43.8	43.8
M_e	kN.m	6.40		8.66		32.70		0.0	
$\Sigma K_b \times 10^3$	mm^3	198.5		210.9		2511.6		613.9	
(b) 3rd Floor Section									
b x D	mm		150x375	150x375	-	150x600	150x600	150x450	150x450

		Left	Right	Left	Right	Left	Right	Left	Right
(c) 2nd Floor			B1	B4		B15	B16	B13	B13
Section Type			FL	FL	–	FL		FL	FL
(d) 1st Floor									
$I_b \times 10^6$	mm^4	–	1318.4	1318.4	–	5400	5400	2278.1	2278.1
$K_b \times 10^3$	mm^3	–	387.76	412.00	–	1255.8	1255.8	529.8	529.8
w_u	kN/m	–	29.70	38.70	–	–	–	–	–
M_{F1} M_{F2}	kN.m	–	14.31	16.51	–	73.81	65.21	85.15	85.15
M_e	kN.m	14.31		16.51		8.60		0.0	
$\Sigma K_b \times 10^3$	mm^3	387.76		412		2511.6		1059.6	
(e) Plinth Section	–								
b x D	mm	–	225x375	225x375	–	225x375	225x375	225x375	225x375
Sec. Type		–	R	R	–	R	R	R	R
$I_b \times 10^6$	mm^4		988.8	988.8	–	988.8	988.8	988.8	988.8
$K_b \times 10^3$	mm^3	–	290.8	309.0	–	229.95	229.95	229.95	229.95
w_u	kN/m	–	26.6	26.6	–	26.6	26.6	18.7	18.7
M_{F1}, M_{F2}	kN.m	–	12.81	11.35	–	40.99	40.99	42.74	42.74
M_e	kN.m	12.81		11.35		0.0		0.0	
$\Sigma K_b \times 10^3$	mm^3	290.8		309.0		459.9		459.9	

(C) Calculation of Moments in Column at Each Floor Level :

(i) Bending @ x-axis

			C27	C7	C15	C14
(a) Roof :	$\Sigma K_b \times 10^3$	mm^3	201.5	1255.8	210.9	409.4
	$K_{R3x} \times 10^3$	mm^3	47.5	900	71.2	21.1
	$\Sigma K \times 10^3$	mm^3	249	2155.8	282.1	430.5
	$d_{col.R3}$		0.191	0.417	0.252	0.049
	M_e	kN.m	6.62	7.06	8.66	5.41
	$M_{col.R3}$	kN.m	1.26	2.94	2.18	0.27
(b) 3rd Floor :						
	$\Sigma K_b \times 10^3$	mm^3	393.5	1255.8	412.0	799.8
	$K_{R3x} \times 10^3$	mm^3	47.5	900.0	71.2	21.1
	$K_{32x} \times 10^3$	mm^3	47.5	900.0	168.75	168.75
	$\Sigma K \times 10^3$	mm^3	488.5	3055.8	651.95	989.65
	$d_{col.3R}$		0.097	0.295	0.109	0.021
	$d_{col.3-2}$		0.097	0.295	0.260	0.171
	M_e	kN.m	13.82	23.31	13.95	11.12
	$M_{col.3R}$	kN.m	1.34	6.88	1.52	0.23
	$M_{col.32}$	kN.m	1.34	6.88	3.63	1.90
(c) 2nd Floor :						
	$\Sigma K_b \times 10^3$	mm^3	393.5	1255.8	412.0	799.8
	$K_{32x} \times 10^3$	mm^3	47.5	900.0	168.75	168.75
	$K_{21.x} \times 10^3$	mm^3	47.5	900.0	569.5	569.5
	$\Sigma K \times 10^3$	mm^3	488.5	3055.8	1150.25	1538.05
	$d_{col.23}$		0.097	0.295	0.146	0.110
	$d_{col.21}$		0.097	0.295	0.495	0.370
	M_e	kN.m	13.82	23.44	13.95	11.12
	$M_{col.23}$	kN.m	1.34	6.88	2.04	1.22
	$M_{col.21}$	kN.m	1.34	6.88	6.91	4.11

(d) 1st Floor :

		C27	C7	C15	C14
ΣK_b	x 10^3 mm^3	393.5	1255.8	412.0	799.8
K_{21x}	x $10^3 mm^3$	47.5	900.0	569.5	569.5
$K_{1PL.x}$	x$10^3 mm^3$	61.9	782.6	1173.9	1173.9
ΣK	x$10^3 mm^3$	502.9	2938.4	2155.4	2543.2
$d_{col.12}$		0.094	0.306	0.264	0.224
$d_{col.1PL}$		0.123	0.266	0.544	0.462
M_e	kN.m	13.82	23.44	13.95	11.12
$M_{col.12}$	kN.m	1.70	7.17	3.68	2.49
$M_{col.1PL}$	kN.m	1.30	6.23	7.59	5.14

e) PL–FT

		C27	C7	C15	C14
ΣK_b	x$10^3 mm^3$	295.2	229.9	309.0	599.8
$K_{1PL.x}$	x$10^3 mm^3$	61.9	782.6	1173.9	1173.9
$K_{PF.x}$	x$10^3 mm^3$	213.5	4050.0	5766.5	6766.5
ΣK	x$10^3 mm^3$	570.6	5062.5	7249.4	7540.2
$d_{col.PL1}$		0.108	0.150	0.162	0.156
$d_{col.PFT}$		0.374	0.800	0.795	0.765
M_e	kN.m	12.44	27.69	7.98	2.05
$M_{col.PL1}$	kN.m	1.34	4.15	1.29	0.32
$M_{col.PFT}$	kN.m	4.65	22.15	6.35	1.57

(ii) **Bending @ y-axis**

a) Roof :

		C27	C7	C15	C14
ΣK_b	x$10^3 mm^3$	198.5	210.9	2511.6	613.9
$K_{3R.y}$	x$10^3 mm^3$	21.1	56.25	71.2	47.5
ΣK	x$10^3 mm^3$	219.6	267.15	2582.8	661.4
$d_{col.R3}$		0.096	0.211	0.028	0.072
M_e	kN.m	6.40	8.66	32.70	0.0
$M_{col.R3}$	kN.m	0.61	1.83	0.92	0.0

(b) 3rd Floor:

		C27	C7	C15	C14
ΣK_b	x10^3 mm^3	387.76	412.0	2511.6	1059.6
$K_{3R.y}$	x10^3 mm^3	21.10	56.25	71.2	47.2
$K_{32.y}$	x10^3 mm^3	21.10	56.25	94.92	94.92
ΣK	x10^3 mm^3	429.96	524.5	2677.72	1201.72
$d_{col.3R}$		0.049	0.107	0.027	0.039
$d_{col.32}$		0.049	0.107	0.035	0.079
M_e	kN.m	14.31	16.51	8.60	0.000
$M_{col.3R}$	kN.m	0.70	1.77	0.23	0.000
$M_{col.32}$	kN.m	0.70	1.77	0.30	0.000

(c) 2nd Floor:

		C27	C7	C15	C14
ΣK_b	x10^3 mm^3	387.76	412.0	2511.6	1059.60
$K_{32.y}$	x10^3 mm^3	21.10	56.25	94.92	94.92
$K_{21.y}$	x10^3 mm^3	21.10	56.25	142.40	142.40
ΣK	x10^3 mm^3	429.96	524.5	2748.92	1296.92
$d_{col.23}$		0.049	0.107	0.034	0.073
$d_{col.21}$		0.049	0.107	0.052	0.110
M_e	kN.m	14.31	16.51	8.60	0.000
$M_{col.23}$	kN.m	0.70	1.77	0.29	0.000
$M_{col.21}$	kN.m	0.70	1.77	0.45	0.000

		C27	C 7	C15	C14
(d) 1stFloor:					
ΣK_b	$x10^3$ mm^3	387.76	412.0	2511.6	1059.60
$K_{21.y}$	$x10^3$ mm^3	21.10	56.25	142.4	142.4
$K_{1PL.y}$	$x10^3$ mm^3	61.90	48.9	165.1	165.1
ΣK	$x10^3$ mm^3	470.76	517.15	2819.1	1367.1
$d_{col.12}$		0.045	0.109	0.051	0.104
$d_{col.1PL}$		0.131	0.095	0.059	0.121
M_e	kN.m	14.31	16.51	8.60	0.0
$M_{col.12}$	kN.m	0.640	1.80	0.44	0.0
$M_{col.1PL}$	kN.m	1.87	1.57	0.51	0.0
(e) PL-FT:					
ΣK_b	$x10^3$ mm^3	290.8	309.0	459.9	459.90
$K_{1PL.y}$	$x10^3$ mm^3	61.90	48.9	165.1	165.10
$K_{PF.y}$	$x10^3$ mm^3	213.50	569.00	640.7	640.70
ΣK	$x10^3$ mm^3	566.20	926.90	1265.7	1265.70
$d_{Col.PL1}$		0.109	0.050	0.130	0.130
$d_{col.PLF}$		0.377	0.61	0.506	0.506
M_e	kN.m	12.81	11.35	0.0	0.0
$M_{col.PL1}$	kN.m	1.40	0.56	0.0	0.0
$M_{col.PLF}$	kN.m	4.83	6.92	0.0	0.0

(1) Summary of Moments in each floor

(i) Bending @ Y - Axis

	Floor	Between		C27	C7	C15	C14
(a)	Roof	(R-3)	kN.m	0.61	1.83	0.92	0.0
	3rd Floor	(3-R)	kN.m	0.70	1.77	0.23	0.0
(b)	3rd Floor	(3-2)	kN.m	0.70	1.77	0.30	0.0
	2nd FLoor	(2-3)	kN.m	0.70	1.77	0.29	0.0
(c)	2nd FLoor	(2-1)	kN.m	0.70	1.77	0.45	0.0
	1st FLoor	(1-2)	kN.m	0.64	1.80	0.44	0.0
(d)	1st Floor	(1-PL)	kN.m	1.87	1.57	0.51	0.0
	Plinth	(PL-1)	kN.m	1.40	0.56	0.00	0.0
(e)	Plinth	(PL-FT)	kN.m	4.83	6.92	0.00	0.0
	Footing	(FT-PL)	kN.m	0.00	0.00	0.00	0.0

(ii) Bending @ X - Axis

	Floor	Between		C27	C 7	C15	C14
(a)	Roof	(R-3)	kN.m	1.26	2.94	2.18	0.27
	3rd Floor	(3-R)	kN.m	1.34	6.91	1.52	0.23
(b)	3rd Floor	(3-2)	kN.m	1.34	6.88	3.63	1.90
	2nd Floor	(2-3)	kN.m	1.34	6.88	2.04	1.22
(c)	2nd Floor	(2-1)	kN.m	1.34	6.88	6.91	4.11
	1st Floor	(1-2)	kN.m	1.70	7.17	3.68	2.49
(d)	1st Floor	(1-PL)	kN.m	1.30	6.23	7.59	5.14
	Plinth	(PL-1)	kN.m	1.34	4.15	1.29	0.32
(e)	Plinth	(PL-FT)	kN.m	4.65	22.15	6.35	1.57
	Footing	(FT-PL)	kN.m	0.00	0.00	0.00	0.00

(E) Design of Column for Axial Load and Moment in each Storey :

(a) COLUMN BETWEEN STOREY R-3

(i) Bending about y - axis (bisecting width of column)

			C27	C7	C15	C14
Section (width x Depth)	b x D	mm	150x225	150x600	225x225	225x150
A_{sc}	N - #	mm	4-12	8-12	4-12	4-12
p %		%	1.34	1.00	0.89	1.34
Floor to Floor height	L_{cc}	mm	3000	3000	3000	3000
Depth of beam at top		mm	300	300	600	375
Unsupported Length	L	mm	2700	2700	2400	2625
Minimum Ecentricity	$e_{y.min}$	mm	20	20	20	20
Axial Load	P_u	kN	37	86	268	160
$M_{uy.min} = P_u \times e_{y.min}$		kN.m	0.74	1.72	5.36	3.2
M_{uy} at top (R -3)			@ 0.61	$ 1.83	0.92	0.0
M_{uy} at Bottom (3 -R)			@ 0.70	$ 1.77	0.23	0.0
Design moment	M_{uy}	kN.M	0.74	1.72	5.36	3.2
Effective Length = L_{eff} = L		mm	2700	2700	2400	2625
L_{eff} / b			18	18	10.67	10.67
Column Type			Long	Long	Short	Short
Additional moment due to Slenderness						
$P_{uz} = .45 x f_{ck} x A_c + .67 x f_y x A_{sc}$		kN	365.45	883	-	-
* $P_{ub} = (k_1+k_2 xp/f_{ck}) x f_{ck} x b x D$		kN	73.86	201.8	-	-
$M_{ey} = (L_{eff}/b)^2 x k x P_u x b/(2000x1000)$		kN.M	1.016	2.45	-	-
Total M_{uy}		kN.M	1.76	4.17	5.36	3.20
d'c / b			0.3	0.3	0.2	0.2
P_u / (f_{ck} bD)			0.073	0.064	0.35	0.316
P/f_{ck}			0.09	0.067	0.59	0.089
Mur / (f_{ck} b^2D)			0.095	0.070	0.085	0.12
Provided M_{ury}		kN.M	7.21	14.17	14.52	13.67

* For of K1 & K2 see Table E - 6

(ii) Bending about x-axis (bisecting Depth of column)

			C27	C7	C15	C14
Section b x D		mm	150x225	150x600	225x225	225x150
A_{sc}	N - #		4-12	8-12	4-12	4-12
p %		%	1.34	1.00	0.89	1.34
Depth of beam at top		mm	300	600	300	300
Unsupported Length L		mm	2700	2400	2700	2700
Minimum Eccentricity	$e_{x.min}$	mm	20	24.8	20	20
Axial Load P_u		kN	37	86	268	160
$M_{x.min1} = P_u \times e_{x.min}$		kN.M	0.74	2.03	5.36	3.20
M_{ux} at top R-3		kN.M	1.26	2.94	2.18	0.27
M_{ux} at bottom 3 - R		kN.M	1.34	6.91	1.52	0.23
Design Moment		kN.m	1.34	6.91	5.36	3.20
Effective Length	L_{eff}	mm	2700	2400	2700	2700
L_{eff} /D			12	4	12	18
Column Type			Short	Short	Short	Long

Bending about x-axis (bisecting Depth of column) Continued ....

	C27	C7	C15	C14
Additional moment due to Slenderness				
P_{uz}				
P_{ub}	365.45	-	-	365.45
M_{ax}	-	-	-	73.86
Total M_{ux}	-	-	-	2.72
Moment of Resistance provided ----	1.34	6.91	5.36	5.92
d'_c/D	See Chart 4	Chart 2	Chart 4	Chart3&4
$P_u/(f_{ck} x b x D)$	0.2	0.10	0.2	0.3
P/f_{ck}	0.073	0.064	0.35	0.316
$M_{ur}/(f_{ck} x b x D^2)$	0.089	0.067	0.059	0.089
Provided M_{urx}	0.125	0.122	0.088	0.087
P_u/P_{uz}	14.24	$$ 98.92	15.04	6.61
Remarks	0.1	-	-	-
α_n	bi-axial	Safe	Safe	Safe
$(M_{ux}/M_{urx})^{\alpha_n} + (M_{uy}/M_{ury})^{\alpha_n}$	1			
Remark	@@ 0.36			
	Safe			

Explanatory Notes :

$ For slender column modified initial moment=.6x1.83-.4x1.77=0.39 >=0.4x1.83 Thus $M_{uy.min}$ is greater than modified initial moment in this case.

@ Since the initial moments are small compared to $M_{uy.min}$ the modified initial moments have not been worked out.

$$ The size of the column is governed by the requirement of the layout and also the reinforcement provided is minimum and hence it is not possible to reduce either the column size or the reinforcement eventhough M_{ux} or M_{uy} is far less then M_{urx} or M_{ury}.

@@ Though the interaction equation gives value far less than unity there is no scope to reduce either the size of the column or steel ,since the section provided is smallest in the module of 75 mm.

(b) Column Between Storey 3-2

(i) Bending about y - axis (bisecting width of column)

			C27	C7	C15	C14
Section	b x D	mm	150x225	150x600	225x300	225x300
A_{sc}	N - #	mm	4-12	8-12	6-12	6-12
p %		%	1.34	1.00	1.34	1.00
Floor to Floor height		mm	3000	3000	3000	3000
Depth of beam at top		mm	375	375	600	450
Unsupported Length L		mm	2625	2625	2400	2550
Axial Load P_u $(=P_r + P_f)$		kN	129	254	501	484
$e_{y.min}$		mm	20	20	20	20
$M_{uy.min} = P_u \times e_{y.min}$		kN.m	2.58	5.08	10.02	9.68
Moment at top		kN.M	0.70	1.77	0.30	0.00
Moment at Bottom		kN.M	0.70	1.77	0.29	0.00
Design moment	M_{uy}	kN.M	2.58	5.08	10.02	9.68
L_{effy} = (0.8 L)		mm	2100	2100	1920	2040
L_{effy} / b		mm	14	14	8.50	9.00
Column Type			Long	Long	Short	Short

Column between storey 3-2 : Bending about y-axis continued ....

		C27	C7	C15	C14
Obtain Additional moment due to Slenderness					
P_{uz}	kN	365.45	883	-	-
P_{ub}	kN	73.86	201.80	-	-
M_{ay}	kN.m	1.54	3.44	-	-
Total M_{uy}	kN.m	4.12	8.52	10.02	9.68
d'_c/b		0.3	0.3	0.2	0.2
$P_u/(f_{ck} \cdot bD)$		0.255	0.19	0.49	0.48
P/f_{ck}		0.09	0.067	0.067	0.067
$M_{ury}/(f_{ck} \cdot b^2D)$		0.09	0.085	0.065	0.065
Provided M_{ury}	kN.m	7.21	16.74	14.80	$ 14.80

(ii) Bending about x-axis (bisecting Depth of column)

Section b x D		mm	150x225	150x600	225x300	225x300
A_{sc}	N - #		4-12	8-12	6-12	6-12
p %		%	1.34	1.00	1.34	1.00
Depth of beam at top		mm	375	600	375	375
Unsupported Length L		mm	2625	2400	2625	2625
$e_{x.min}$		mm	20	24.8	20	20
Axial Load P_u		kN	129	254	501	484
$M_{ux.min} = P_u \times e_{x.min}$		kN.m	2.58	6.30	10.02	9.68
Moment at top		kN.m	1.34	6.88	3.63	1.90
Moment at bottom		kN.m	1.34	6.88	2.04	1.22
@Design moment		kN.m	2.58	6.88	10.02	9.68
L_{effx} =(0.8 L)		mm	2100	1920	2100	2100
L_{effx}/D			9.3	3.2	7.0	7.0
Column Type			Short	Short	Short	Short

Explanatory Notes :

@ Since Design moments about x-axis are less than design moments about y-axis and the sections are safe for bending about y-axis they will be safe for bending about x-axis also. Check for bending about x-axis should be carried out when M_{ux} is greater than M_{uy}.

$ Though the difference between M_{uy} and M_{ury} is large , it is not possible to reduce the size of the column. If the size of the column is reduced to 225mmx225mm keeping the same reinforcement , M_{ury}' works out to be equal to 5.98 kN.m. Hence unsafe. For column size of 225mmx225mm reinforcement required is 8-#12 (p=1.78%) ,M_{ury} works out to be equal to 10.25 kN.m. which is greater than 9.68 kN.m. Hence safe. Now, the choice is left to the designer/contractor to select the section. However, it is recommended to choose a section having low percentage of steel,since the cost of the steel is much higher than the cost of the concrete.

(c) Column Between Storey 2-1

Bending about y - axis

			C27	C7	C15	C14
Section	b x D	mm	150x225	150x600	225x450	225x450
Area of steel	N - #	mm	6-12	8-12	8-12	4-16+2-12
p %		%	2.01	1.00	0.89	1.02
Depth of beam at top		mm	375	375	600	450
Unsupported Length		mm	2625	2625	2400	2550
$e_{y.min}$		mm	20	20	20	20
Axial Load P_u $(=P_r + 2P_f)$		kN	221	422	734	8088
$M_{uy.min} = P_u \times e_{y.min}$		kN.m	4.42	8.44	14.68	16.16
Moment at top (2 - 1)		kN.m	0.7	1.77	0.45	0.0
Moment at Bottom (1 - 2)		kN.m	0.64	1.80	0.44	0.0
Design moment M_{uy}		kN.	4.42	8.44	14.68	16.16
L_{effy} (=0.8 L)		mm	2100	2100	1920	2040
L_{effy} / b		mm	14	14	12	12
Column Type			Long	Long	Short	Short
Obtain Additional moment due to Slenderness						
P_{uz}/A_g (see chart-9)			\$ 12.80	\$ 0.98	-	-
P_{uz}		kN	432	882		
P_{ub}		kN	70.30	201.8	-	-
M_{ay}		kN.m	1.91	4.19	-	-
Total M_{uy}		kN.m	6.33	12.63	14.68	16.16
d'_c/b			0.3	0.3	0.2	0.2
$P_u/(f_{ck}\ bD)$			0.44	0.31	0.48	0.532
p/f_{ck}			0.134	0.067	0.06	0.068
$M_{ury}/(f_{ck}\ b^2D)$			0.090	0.080	0.055	0.048
Provided M_{ury}		kN.m	6.83	16.20	18.79	16.40

(ii) Bending about x - axis (bisecting depth of column)

Depth of beam at top		mm	375	600	375	375
Unsupported Length	L	mm	2625	2400	2625	2625
$e_{x.min}$		mm	20	24.8	20	20
Axial Load	P_u	kN	221	422	734	808
$M_{ux.min}$		kN.m	4.42	10.46	14.68	16.16
Moment at top (2 - 1)		kN.m	1.34	6.88	6.91	4.11
Moment at bottom(1 - 2)		kN.m	1.70	7.17	3.68	2.49
Design moment M_{ux}		kN.m	4.42	10.46	14.68	16.16
L_{effx} (=0.8 L)		mm	2100	1920	2100	2100
L_{effx}/D			9.3	3.2	4.7	4.7
Column Type			Short	Short	Short	Short
Check for moment of resistance provided:						
d'_c/D			0.2	0.1		
$P_u/(f_{ck}\ bD)$			0.44	0.31		
p/f_{ck}			0.134	0.67		
$M_{urx}/(f_{ck}\ b\ D^2)$			0.13	0.122		
M_{urx} provided			14.80	98.82		
M_{ux}/M_{urx}			0.299	-		
M_{uy}/M_{ury}			0.926	-		
P_u/P_{uz}			0.51	-		
Allowable M_{ux}/M_{urx} (from chart-10)			@ 0.3	-		

* Since M_{ux} is nearly equal to M_{uy} and column is safe for bending about y-axis , it will be safe for bending about x-axis also.

Explanatory Notes :

$ Instead of using formula for calculation of P_{uz}, the value of P_{uz}/A_g is obtained directly from Chart 9.

@ The column subjected to bi-axial bending has been designed directly by using Chart-10

(d). Column Between Storey 1-PL

(i) Bending about y-axis (bisecting width of column)

		C27	C7	C15	C14
Section b x D	mm	@ 225x225	150x600	225x600	225x600
Area of steel N - #	mm	6-12	8-16	4-16+4-12	8-16
p %	%	1.34	1.79	0.93	1.19
Height from plinth beam to floor top	mm	3450	3450	3450	3450
Depth of beam at top	mm	375	375	600	450
Unsupported Length L	mm	3075	3075	2850	3000
$e_{y.min}$	mm	20	20	20	20
P_u $(=P_f + 3P_f)$	kN	313	590	967	1132
$M_{uy.min} = P_u \times e_{y.min}$	kN.m	6.26	11.80	19.34	22.64
Moment at top	kN.m	1.87	1.57	0.51	0.00
Moment at Bottom	kN.m	1.40	0.56	0.00	0.00
Design moment M_{uy}	kN.m	6.26	11.80	19.34	22.64
L_{effy} $(=0.8\ L)$	mm	2460	2460	2280	2400
L_{effy}/b		10.9	16.4	10.1	10.67
Column Type		Short	Long	Short	Short
Additional moment due to Slenderness					
P_{uz}/A_g	kN/mm²	15.3	12.1	-	-
P_{uz}	kN	516	1089	-	-
P_{ub}	kN		190.6	-	-
M_{ay}	kN.m		6.54	-	-
Total M_{uy}	kN.m	6.26	18.34	19.34	22.64
d'_c/b		0.2	0.30	0.20	0.20
$P_u/(f_{ck}\ bD)$		0.41	0.44	0.48	0.56
P/f_{ck}		0.089	0.12	0.062	0.079
$M_{ury}/(f_{ck}\ b^2D)$		0.100	0.082	0.060	0.050
Provided M_{ury}	kN.m	17.08	16.60	27.34	22.78
Remarks		safe	$ unsafe	safe	safe
Revised Section. (b x D)	mm	-	225x600		
A_{sc} N - #		@ -	4-16+4-12	-	-
p %	%	-	0.93	-	-
L_{effy}/b		-	10.90	-	-
Column Type		-	Short	-	-
d'_c/b		-	0.30	-	-
$P_u/f_{ck}\ D$		-	0.29	-	-
p/f_{ck}		-	0.062	-	-
$M_{ury}/f_{ck}\,b^2D$		-	0.076	-	-
Provided M_{ury}		-	34.62 (safe)		-

Explanatory Notes : $ It can be seen that the tentative section assumed has worked out to be unsafe. This shows inadequacy of method of determination of section based on equivalent load and need to perform detailed calculations. However it may be noted that in all other cases the section has worked out to be safe.

@ Also, it may be seen that due to marginal increase in the size of the column from 150mmx300mm to 225mmx225mm, the column becomes short, requires less steel and also works out to be safe. The calculations have been given specifically to show the demerits of 150mm wide columns for lower storeys.

Column between storey 1-PL continued ......

Bending about x-axis

		C27	C7	C15	C14
Section b x D	mm	225x225	225x600	225x600	225x600
Area of steel N - #	mm	6-12	4-16+4-12	4-16+4-12	8-16
p %	%	1.34	0.93	0.93	1.19
Height from plinth beam to floor top	mm	3450	3450	3450	3450
Depth of beam at top	mm	375	600	375	375
Unsupported Length L	mm	3075	2850	3075	3075
$e_{x.min}$	mm	20	25.70	26.15	26.15
Axial Load P_u	kN	313	590	967	1132
$M_{ux.min}$	kN.m	6.26	15.16	25.29	29.60
Moment at top	kN.m	1.30	6.23	7.59	5.14
Moment at Bottom	kN.m	1.34	4.15	1.29	0.32
Design moment	kN.m	6.26	15.16	25.29	29.60
L_{effx} (=0.8 L)	mm	2460	2280	2460	2460
L_{effx} /D	mm	10.9	3.8	4.1	4.1
Column Type		Short	Short	Short	Short
Check for momenting resistance provided					
P_{uz}	kN	530	-	-	-
d'_c/D		0.20	0.10	0.10	0.10
$P_u/(f_{ck}\ bD)$		0.41	0.44	0.48	0.56
p/f_{ck}		0.089	0.12	0.062	0.079
$M_{urx}/(f_{ck}\ b\ D^2)$		0.10	0.15	0.06	0.030
Provided M_{urx}	kN.m	17.08	182	72.90	36.45
Remark		biaxial	Safe	Safe	Safe
M_{uy}/M_{ury}		0.37	-	-	-
P_u/P_{uz}		0.59	-	-	-
M_{ux}/M_{urx}		0.37	-	-	-
Allowable M_{ux}/M_{urx} (from chart-10)		0.87	-	-	-
		Safe			

(e) COLUMN BETWEEN STOREY : PL - FT

Bending about y-axis (bisecting width of column)

		C27	C7	C15	C14
Section b x D	mm	225x225	225x600	225x675	225x675
Area of steel N - #	mm	8-12	4-16+4-12	4-16+4-12	10-16
p %	%	1.78	0.93	0.83	1.32
Height from plinth beam to footing	mm	1000	1000	1000	1000
Depth of plinth beam	mm	375	375	375	375
Unsupported Length L	mm	625	625	625	625
$e_{y.min}$	mm	20	20	20	20
Axial Load P_u (=P_r+3P_f+P_p)	kN	399	711	1123	13144
$M_{uy.min}$	kN.m	7.98	14.22	22.46	26.28
Moment at top (PL-FT)	kN.M	4.83	6.92	0.0	0.0
Moment at Bottom (FT-PL)	kN.M	0.0	0.0	0.0	0.0
Design moment	kN.M	7.98	14.22	22.46	26.28
L_{effy} (=0.8 L)	mm	500	500	500	500
L_{effy} / b	mm	2.2	2.2	2.2	2.2
Column Type		Short	Short	Short	Short

Column between Storey PL-FT :- Bending about y-axis continued ....

		C27	C7	C15	C14
d'_c / b		0.2	0.2	0.2	0.2
P_u / (f_{ck} bD)		0.53	0.35	0.49	0.58
p/f_{ck}		0.12	0.062	0.055	0.088
M_{ury} / (f_{ck} b^2D)		0.1	0.09	0.048	0.055
Provided M_{ury}	kN.M	17.09	41.00	24.60	28.19

Bending about x - axis (bisecting depth of column)

		C27	C7	C15	C14
Section (b x D)	mm	225x225	225x600	225x675	225x675
A_{sc}	N - #	8-12	4-16+4-12	4-16+4-12	10-16
Depth of plinth beam	mm	375	375	375	375
unsupported Length L	mm	625	625	625	625
$e_{x.min}$	mm	20	21.25	23.75	23.75
Axial Load P_u	kN	399	711	1123	1314
$M_{ux.min}$	kN.m	7.98	15.11	26.67	31.21
Moment at top	kN.m	4.65	22.15	6.35	1.57
Moment at Bottom	kN.m	0.0	0.0	0.0	0.0
Design moment M_{ux}	kN.m	7.98	22.15	26.67	31.21
L_{effx} (=0.8 L)	mm	500	500	500	500
L_{effx} /D	mm	2.2	0.8	0.7	0.7
Column Type		Short	Short	Short	Short
d'_c/D		0.2	0.1	0.1	0.1
P_u / (f_{ck} bD)		0.53	0.35	0.49	0.58
p/f_{ck}		0.12	0.062	0.055	0.088
M_{ur} / (f_{ck} b D^2)		0.1	0.102	0.05	0.064
Provided M_{urx}	kN.m	17.09	124	77	98

8.6.7 Approximate Method of Computation of Loads on Columns :

(a) When a structure cannot be divided into plane 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.

(b) Unit Ultimate Loads : These are given in Table. A-1 but are reproduced for ready reference.

Slab:Roof/Floor 9 kN/m² Wall:External con.Block 150mm Internal 112.5mm brick
Balconies(SS) 10 kN/m² 1.0 m.high 6.75 kN/m 2 m.high 9.0 kN/m
Bath & W.C. 10.5 kN/m² 2.0 m high 13.50 kN/m 2.7m high 12.0 kN/m
Loft 8 kN/m² 3.0 m high 20.00 kN/m 3.0m.high 13.5 kN/m
Stair 15 kN/m² 3.45m high 23.00 kN/m 3.45m high 15.5 kN/m
Chajja 2.25 kN/m² Grill - 1.0 m. high 1.5 kN/m
\- 2.7 m. high 4.0 kN/m
Balcony parapet 80 mm thick - 3 kN/m
Beam rib Self weight : Size 150x300 mm 1.1 kN/m. , 150x375 mm 1.5 kN/m.
(taking D_f=100 mm) Size 150x450 mm 2.0 kN/m. , 150x600 mm 2.8 kN/m.

Computation of Load on Beams : (A) Roof Beams

In the following

Type 1 refers to - Load transfered from one-way slab.
Type 2 refers to - Trapezoidal Load transferred from two-way slab.
Type 3 refers to - Triangular Load transferred from two-way slab.

Beam No.	Span: m	Load Transferred from slab: Lf	Typ	Rt	Typ	Wall: kN/m	Self: kN/m	Load calculations & Beam End Rections.	Total load kN.
B1(SS)	3.4	-	-	S1	2	3	1.1	9[(3.4-3.35)+3.4]3.35/4+3x3.4+1.1x3.4 = 26+10.2+3.74 = say VL = 20 kN & VR = 20 kN.	40
B3 (SS)	3.4	S1	2	S4	1	-	1.1	26+9(2.4x3.4)/2+1.1x3.4 = 26+36.72+3.74 = say VL = 33 kN & VR = 33 kN	66
B6 (SS)	3.4	S4	1	S4	1	-	1.1	36.72+9(2.5x3.4)/2+1.1x3.4 = 36.72+38.25+3.74 = say VL = 40 kN & VR = 40 kN	80
B2 (SS)	3.2	-	-	S2	1	5.25	1.1	9x1.6x3.2/2+5.25x3.2+1.1x3.2 = 23.04+16.8+3.52 = say VL = 22 kN & VR = 22 kN	44
B4 (SS)	3.2	S2	1	S5	3	-	1.1	23.04+9x3.3²/4+3.52 = 23.04+23.04+3.52 = say VL = 25 kN & VR = 25 kN	50
B7 (SC)	3.4	S4	1	S4	1	-	1.1	38.25+38.24+3.74 = VL =.45x80 = 36 kN & VR =.6x80 =48kN	80
B8 (CS)	3.2	S5	3	S5	3	-	1.1	23.04+23.04+3.52 = say VL =.6x50= 30 kN & VR = .45x50=22kN	50
B9 (CC)	2.5	-	-	-	-	3	1.1	3x2.5+1.1x2.5 = 7.5+2.75 = say VL = 5 kN & VR = 5 kN	10
B10(CC)	2.4	-	-	-	-	3	1.1	3x2.4+1.1x2.4 = 7.2+2.644 = say VL = 5 kN & VR = 5 kN.	10
B11(SC)	3.35	-	-	S1	3	3	1.1	9x3.35²/4+3x3.35+1.1x3.35 = 25.25+10.05+3.69 = say VL =.45x40 = 18 kN & VR =.6x40=24 kN	40
B13(CC)	4.3	-	-	S5	2	-	1.5	Total Ld=9[(4.3-3.2)+4.3]3.2/4+1.5x4.3= Pt.Load 40kN from B6 @ 1.8m from LH end VL = 46/2+40x2.5/4.3 = 46 kN VR = 46+40-46 = 40 kN	46
B14(SC)	3.35	S1	3	-	-			Same as B11 + Pt load from B2(22kN) VR = 40/2+22x2.35/3.35)x1.2 = 43 kN VL = 40/2+22x1/3.35)x0.9 = 24 kN.	
B15a(F)	1.0	-Cap slab				13.5	2.8	13.5x1+9x1x2.4/2+2.8+22(B2)+5(B20) = VL = 54 kN.	54
B16a(F)	1.0	-	-	-	-	3	2.8	3x1+2.8x1 = 6 kN + 22 kN(from B2) =	28
B17(SS)	2.4	-	-	-	-	3	1.1	Load from chajja 2.25 kN/m Total Load =(3+2.25)x2.4 + 1.1x2.4 = VL = 8 kN & VR = 8 kN	16
B18	2.4	Details worked out separately. VL = 41 kN & VR = 41 kN							
B15	4.3	Details worked out sepatately. VL = 91 kN.& VR = 79 kN							
B16	4.3	Details worked out separately. VL = 48 kN & VR = 38 kN							
B19	2.4	Details worked out separately. VL = 49 kN & VR = 49 kN							

S = Simply Supported , C = Continuous

Explanatory Notes :

Beam B18 : End shear at simply supported end of stairs slab –

w_u = 15(2.825+1.075) - 40.38 = say 19 kN/m

Load on B18=Load due to stairs(19kN/m)+wall load(13.5kN/m+self(1.1)

VL = VR = (19 + 13.5 + 1.1)x2.4/2 = 41 kN

Beam B15 : The loading consists of load transferred from S4 , S5 and internal wall 3m high +self weight + load from B18 (41 kN)acting at distance of 2.3m from left hand end.

W1 = load from (S5 + S4 +Self) over the whole span.

= 9x[4.3-3.2)+4.3]x3.2/4+9x4.3x2.4/2+2.8x4.3 = 38.88+46.44+12.04 = 98 kN.

W2 due to wall = 13.5x2.3 = 31 kN acting over a length of 2.3m from LH end.

Concentrated load of 41 kN from B18 acting at distance of 2.3 m from LH end.

∴ VR = 31x2.3/(2x4.3)+98/2+41x2.3/4.3 = 79 kN.

VL = 31+98+41-79 = 91 kN (Ignoring effect of fixed end moments)

Beam B16 :- W1 = Load from S5 +Self acting over the whole length. :

= 38.88 +12.04 = 51 kN (for details see B15)

W2 = Load from S4 acting over the length of 1.75 m from LH end.

= 9x1.75x2.4/2 = 19 kN.

W3 = Parapet wall acting over a length of 2.55m from RH end.

= 3x2.55 = 8 kN

Concentrated load from B17 (8 kN) acting at an distance of 1.75m from LH end

VR = 51+19+8+8-48 = 38 kN.

These values have been entered in the Table above.

Beam B19 : End shear at overhanging end of stairs slab :-

Taking moments about LHS.

w_u = 15(2.825+1.075)²/(2x2.825) = 40.38 kN./m

hence, VL = VR = 40.38x2.4/2 = 49 kN

Computations of End Reaction of Floor Beams :

In the cases where weight of full height of wall is taken into account, the self weight of beam is not added separately. The difference between the unit weight of concrete and the unit weight of masonary is neglected.

Beam No.	Span	Load transferred from slab Lf	Typ	Rt	Typ	Wall kN/m	Self kN/m	Load calculations & Beam End Rections.	Total load kN.
B1 (SS)	3.4	-	-	S1	2	20	-	Total Load=9[3.4-3.35)+3.4]x3.35/4+ +20x3.4 = 28 + 68 = VL = VR = 47 kN	94
B5 (SS)	2.3	S3	1	S3	1	9	2.0	Total Load =[10.5+8]x[1.05+1.35)/2]x2.3 +(9 + 2)x2.4 = VL = VR = 38 kN	76
B12(SS)	2.4	-	-	S3	1	9	2.0	Total Load=9x1.1x2.4/2+(9+2)x2.4 =38 + Pt.Load of 38 kN from B5 acting at 1.05m from RH end VR = 38/2 + 38x1.05/2.4 = 36 kN VL = 38 + 38 -36 = 40 kN	
B10(SS)	2.4	-	-	-	-	20	-	20 x 2.4 = Pt.Ld of 38kN from B5 at 1.05m from RH end VR = 48/2 + 38x1.05/2.4 = 41 kN VL = 48 + 38 - 41 = 45 kN	48

End reactions from Floor Beams continued

Beam No.	Span	Load transferred from slab: Lf	Typ	Rt	Typ	Wall kN/m	Self kN/m	Load calculations & Beam End Rections.	Total load kN
B3 (SS)	3.4	S1	2	S3	1	13.5	-	26+(10.5+8)x(1.05/2)x2.3 + 13.5x3.4 =	94
								Pt.Ld of 40kN from B12 at 1.1m from RH end	
								VL = 94/2 + 40x1.1/3.4 = 60 kN	
								VR = 94 +40 - 60 = 74 kN	
B6 (SS)	3.4	S3	1	S4	1	13.5	-	9x2.5x3.4/2+(10.5+8)x(1.35/2)x2.3+	
								13.5x3.4 = 38.25 + 28.72 + 45.9 =	113
								Pt.Ld of 36kNfrom B12 at 1.1m from RH end	
								VL = 113/2 + 36x1.1/3.4= 68 kN	
								VR = 113 + 36 - 68 = 81 kN	
B7 (SC)	3.4	S4	1	S4	1	13.5	-	38.25+38.25+13.5x3.4 =	122
								VL =.45x122 =55 kN & VR =.6x122 =73kN	
B2 (SS)	3.2	-	-	S2	1	5.25	1.5	9x1.6x3.2/2 + 5.25x3.2 + 1.5x3.2	
								= 23.04+16.8+4.8 =Say:	44
								VL = 22 kN & VR = 22 kN	
B4 (SS)	3.2	S2	1	S5	3	20	-	23.04+9x3.2^2/4+20x3.2=23.04+23.04+6.4=	110
								VL = 55 kN & VR = 55 kN	
B8 (CS)	3.2	S5	3	S5	3	13.5	-	23.04 + 23.04 + 13.5x3.2 =	90
								VL = .6x90 = 54 kN & VR=.45x90= 40 kN	
B9 (SC)	3.25	-	-	-	-	20	-	20 x 2.5 =	50
								VL =.45x50 = 22 kN & VR =.6x50= 30 kN	
B11(SS)	3.35	-	-	S1	3	20	-	9x3.35^2/4+20x3.35 = 25.25 + 67 =	92
								VL = 92/2 = 46 kN & VR = 46 kN	
B14(SC)	3.35	S1	3	-	-	20	-	S1+wall+Pt.Ld 22kN(from B2)at 1m from RHS	
								VL=.9[22x1.0/3.35+(25.25+67)/2]=47 kN	
								VR=1.2[22x2.35/3.25+(25.25+67)/2]=74kN	
B17(SS)	2.4	-	-	-	-	5.25	1.1	5.25x2.4+1.1x2.4=16kN , VL = VR = 8kN	
B18(SS)	2.4	Stairs					1.5	1.5x2.4x2.825/2 + 1.5x2.4 =	54
								VL = 27 kN & VR = 27 kN	
B19(SS)	2.4	Stairs+landing					1.5	15x2.4x2.825/2+15x1x 2.4+1.5x2.4 =	90
								VL = 45 kN & VR = 45 kN	
B20(SS)	2.4	Grill					1.1	4x2.4+1.1x2.4=12kN, VL = VR = 6 kN	
B13(CC)	4.3	S3	1	S5	2			9x1.1/2x4.3+9x[(4.3-3.2)+4.3]3.2/4	
								+13.5x4.3= 21.28 + 38.88 + 58.05 =	118
								Pt.Load 81 kN (B6)at 1.8m from LH end:	
								VR=118/2+81x1.8/4.3=93 kN	
								& VL=118+81-93=106 kN	
B15(CC)	4.3	S5	2	S4	1	20	-	W1 = 38.88+20x4.3=125 kN	
								W2=9x2x2.4/2=22kN for 2m from RH end:	
								Pt.Ld. 27kN(B18) at 2m from RH end	
								VL=125/2+27x2/4.3 + 22x1/4.3 = 80 kN	
								VR = 125+22+27-80 — = 94 kN	
B16(CC)	4.3	S5	2	S4	1	-	-	W1=125 kN as obtained above	
								W2=9x1.75x2.4/2=19kN for 1.75m from LH end	
								Pt.Load of 8kN(B17)at 1.75m from LH end	
								VL=125/2+8x2.55/4.3+19(4.3-1.75/2)/4.3=82kN	
								VR = 125+8+19-82 = 70 kN	
B15a	1.0	Fixed				18	2.8	Pt.Load of 28 kN from B2 & B20 at free end	
								VR = 18x1(22+6)+2.8x1 = 49 kN	
B16a	1.0	Fixed				18	2.8	Pt.Load of 22 kN(from B2)at free end:	
								VL = 18x1 + 2.8x1 + 22 = 43 kN	

Computation of end reaction of Plinth Beams :

The plinth beams only carry wall load and its self weight. Beam B2,B5,B12,B17,B18 & B20 have not been provided at plinth level. Beam B20 & B21(between column C15 & C16) of size 225mmx300mm have been provided to act as stiffner for the frame. These beams carry their self weight only.

The external walls of height 3.45 m are 225 mm wide and the supporting plinth beams are of size 225mmx375mm. The load due to wall and self weight of beam = 1.5(20x0.225x3.45)+1.5(25x0.225x0.375)= 26.6 kN/m

The internal walls are having thickness of 150mm and height of 3.45m. The beam has size of 225 mm x 375 mm.

The ultimate load due to internal wall including self weight of beam
= 1.5(20x0.15x3.45)+1.5(25x0.225x0.375)
= 18.7 kN/m

The End Reactions of Plinth Beams are worked out - as detailed below.

Beam No.	Span m.	Wall thickness: mm	Total Load kN	Beam End Reactions. kN
B1(SS)	3.4	225	26.6x3.4 = 90 kN	VL = 45 kN & VR = 45 kN
B3(SS)	3.4	150	18.7x3.4 = 64 kN	VL = 32 kN & VR = 32 kN
B6(SS)	3.4	150	18.7x3.4 = 64 kN	VL = 32 kN & VR = 32 kN
B7(SC)	3.4	150	18.7x3.4 = 64 kN	VL=.45x64=29 kN & VR=.6x64=38 kN
B4(SS)	3.2	225	26.6x3.2 = 86 kN	VL = 43 kN & VR = 43 kN
B8(CS)	3.2	150	18.7 x 3.2 = 60 kN	VL=.6x60=36 kN & VR=.45x60=27 kN
B9(CC)	2.5	225	26.6x2.5 = 67 kN	VL = 67/2 = 34 kN & VR = 34 kN
B10(CC)	2.4	225	26.6 x 2.4 = 64 kN	VL = 64/2 = 32 kN & VR = 32 kN
B11(SC) &B14(SC)	3.35	225	26.6 x 3.35= 90 kN	VL=.45x90 =40kN & VR =.6x90=54kN
B13(CC)	4.3	150	18.7x4.3=80 kN + Pt.Load of 32 kN(B6)1.8m from LH end	VR = 80/2+32x1.8/4.3 = 53 kN VL = 80+32 - 53 = 59 kN
B15a(F) &B16a(F)	1.0	225	26.6 x 1.0 = 27 kN	VR = 26.6 x 1.0 = 27 kN
B15(CC) B16(CC)	4.3	225	26.6 x 4.3 = 114 kN	VL = 114/2 = 57 kN & VR = 57 kN
B19(SS) &B20(SS)	2.4	-	1.5(25x.225x.3)=6kN	VL = VR = 6/2 = 3 kN

Computation of Load on Column

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.

```
        1
        |
  4 --- C --- 2
        |
        3
```

COLUMNS IN TOP STOREY

Loads Transferred by Roof Beams (Approximate method)

Cate-gory	Column Mark	Beam End Forces in kN from directions 1	2	3	4	Total kN	Tank Load kN	Total Rounded Pr kN
I	C14	40	30	40	48	158	-	158
II	C 7	28	-	38	25	91	-	91
	C13	5	36	5	-	46	-	46
	C15	48	-	79	22	149	105	254
	C19	5	40	5	-	50	-	50
	C21	5	33	24	-	62	-	62
	C22	46	25	43	33	147	-	147
	C23	91	49	54	25	219	105	324
III	C27	18	20	-	-	38	-	38
	C28	24	-	-	20	44	-	44

COLUMNS IN INTERMEDIATE STOREY

Loads Transferred by Floor Beams (Approximate Method)

Cate-gory	Column Mark	Beam End tonses in kN from directions 1	2	3	4	Total kN	Total Pf kN
I	C14	93	54	93	73	313	313
II	C 7	43	-	70	55	168	168
	C13	30	55	30	-	115	115
	C15	82	-	94	40	216	216
	C19	22	68	41	-	131	131
	C21	45	60	46	-	151	151
	C22	106	55	74	74	309	309
	C23	80	45	49	55	229	229
III	C27	46	47	-	-	93	93
	C28	47	-	-	47	94	94

COLUMNS IN FIRST STOREY

Loads Transferred by Plinth Beams (Approximate Method)

Category	Column Mark	Beam End shears in kN from directions 1	2	3	4	Total kN	Total Rounded Pf kN
I	C14	53	36	53	38	180	180
II	C 7	27	-	57	43	127	127
	C13	34	29	34	-	97	97
	C15	57	3	57	27	144	144
	C19	34	32	32	-	98	98
	C21	32	32	54	-	118	118
	C22	59	43	54	32	188	188
	C23	57	3	27	43	130	130
III	C27	40	45	-	-	85	85
	C28	40	-	-	45	85	85

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	Category II							Category III	
	C14	C7	C13	C15	C19	C21	C22	C23	C27	C28
Top Storey P_r kN	158	91	46	254	50	62	147	324	38	44
Int. Storey P_f kN	313	168	115	216	131	151	309	229	93	94
1st Storey P_p kN	180	127	97	144	98	118	188	130	85	85

(B) Loads at Each Storey LEVEL in kN. (Approximate Method)

		C14	C7	C13	C15	C19	C21	C22	C23	C27	C28
Top Storey P_r	kN	158	91	46	254	50	62	147	324	38	44
3rd Storey P_r+P_f	kN	471	259	161	470	181	213	456	553	131	138
2nd Storey P_r+2P_f	kN	784	427	276	686	312	364	765	782	224	232
1st Storey P_r+3P_f	kN	1097	595	391	902	443	515	1074	1011	317	326
Plinth $P_r+3P_f+P_p$	kN	1277	722	488	1046	541	633	1262	1141	402	411
Add 10%	kN	128	72	49	105	54	63	126	114	40	41
Total Load	kN	1405	794	537	1151	595	693	1388	1255	442	452

Explanatory Note: It will be seen that the total load at plinth level matches well with those obtained using accurate method in Sec.8.6.4. The final loads have been increased arbitrarily by 10% to make allowance for fixity, slenderness and approximations involved in computations. A comparison of final loads obtained with those worked out in Sec.8.6.5(e) will show that the loads obtained by approximate method are marginally on higher side and hence can be used for design of foundation.

Schedule of Columns

Storey	Column	Section b(mm) x D(mm)	Main Steel N - #	p %	Lateral Ties 0(mm)-S(mm)
R-3	C14	225 x 150	4 - 12	1.34	6 - 150
	C 7	150 x 600	8 - 12	1.00	6 - 150
	C15	225 x 225	4 - 12	0.89	6 - 190
	C13,C19,C21	150 x 225	4 - 12	1.34	6 - 150
	C22	150 x 600	8 - 12	1.00	6 - 150
	C23	150 x 600	4 - 16	0.89	6 - 150
	C27 & C28	150 x 225	4 - 12	1.34	6 - 150
3-2	C14	225 x 300	6 - 12	1.00	6 - 190
	C 7	150 x 600	8 - 12	1.00	6 - 150
	C15	225 x 300	6 - 12	1.34	6 - 190
	C13,C19,C21	150 x 300	4 - 12	1.34	6 - 150
	C22	150 x 600	8 - 12	1.00	6 - 150
	C23	150 x 600	6-16+2-12	1.59	6 - 150
	C27,C28	150 x 225	4 - 12	1.34	6 - 150
2-1	C14	225 x 450	4-16+2-12	1.02	6 - 190
	C7	150 x 600	8 - 12	1.00	6 - 150
	C15	225 x 450	8 - 12	0.89	6 - 190
	C13,C19,C21	150 x 450	4 - 16	1.19	6 - 150
	C22	225 x 600	4-16+4-12	0.93	6 - 190
	C23	225 x 600	6-16+2-12	1.06	6 - 190
	C27,C28	150 x 225	6 - 12	2.01	6 - 150
1-PL	C14	225 x 600	8 - 16	1.19	6 - 225
	C7	225 x 600	4-16+4-12	0.93	6 - 190
	C15	225 x 600	4-16+4-12	0.93	6 - 190
	C13,C19,C21	225 x 450	4-16+2-12	1.02	6 - 190
	C22	225 x 600	8-16+2-12	1.36	6 - 190
	C23	225 x 600	8 - 16	1.19	6 - 225
	C27,C28	225 x 225	6 - 12	1.34	6 - 190
PL-FT	C14	225 x 675	10 - 16	1.32	6 - 225
	C7	225 x 600	4-16+4-12	0.93	6 - 190
	C15	225 x 675	4-16+4-12	0.83	6 - 190
	C13,C19,C21	225 x 450	4-16+2-12	1.02	6 - 190
	C22	225 x 675	10 - 16	1.32	6 - 225
	C23	225 x 675	8 - 16	1.32	6 - 225
	C27,C28	225 x 225	8 - 12	1.78	6 - 190

8.7 DESIGN OF FOOTINGS

8.7.1 Categorisation of Footings

All footings have been designed for axial loads as the bearing capacity of the soil is reasonably low.

8.7.2 Grouping of Footings

Footings of the columns having same sizes and variation of load of about 5% are grouped together and designed for the maximum load (see Sect.8.6.4) in that group.

Accordingly the footings have been divided into the following groups:-

Group	Maximum Load	Design Load	Column Size	Column Mark
I(a)	399 kN	400 kN	225 x 225	C27, C28
I(b)	654 kN	660 kN	225 x 450	C13,C19,C21
I(c)	711 kN	720 kN	225 x 600	C7
I(d)	1180 kN	1200 kN	225 x 675	C15,C23
I(e)	1314 kN	1320 kN	225 x 675	C14,C22

8.7.3 Design of Footing

Footings are designed for soil bearing capacity of 250 kN/m² as per the procedure given in Sect.5.10. The design is given in the tabular form and values of all important intermediate steps have been given.

I Size of Footing : :

Group	P_u kN	q kN/m²	b mm	D mm	Req.A_f m²	B_f mm	L_f mm	Prov.A_f m²	w_u kN/m²	Proj-x mm	Proj-y mm
(a)	400	250	225	225	1.173	1100	1100	1.21	330.60	437.5	437.5
(b)	660	250	225	450	1.936	1300	1525	1.98	332.91	537.5	537.5
(c)	720	250	225	600	2.20	1325	1700	2.25	319.64	550	550
(d)	1200	250	225	675	3.52	1675	2125	3.56	337.08	725	725
(e)	1320	250	225	675	3.872	1775	2225	3.95	334.23	775	775

II Depth of Footing from Bending Moment Considerations :

Group	M_{ux} kN.m.	M_{uy} kN.m.	Col. offset mm	Required dx mm	Required dy mm	Total D_f mm	Provided dx mm	Provided dy mm	$D_{f,min}$ mm
(a)	34.8	34.80	50	246.00	238.00	325	271	263	150
(b)	62.52	73.34	50	304.89	253.80	375	321	313	150
(c)	64.06	82.19	50	308.60	238.16	400	346	338	150
(d)	148.39	188.25	50	469.65	342.56	525	470	460	150
(e)	178.16	233.33	50	514.16	381.37	575	520	510	150

III Area of Steel and Check for Two-way Shear

Group	Required Steel A_{stx} mm²	Required Steel A_{sty} mm²	# mm	Requd. L_d mm	Prov. L_d mm	:	2(B1+L1) mm	d1 mm	V_{uc} kN	V_{uD} kN
(a)	408	425	8	456	501	:	1952	238.5	451	321
(b)	653.6	737	8	456	512		2602	271.9	685	523
(c)	602	737	8	456	550	:	3002	286.5	729	551
(d)	1094	1267	10	570	700	:	3676	327.0	966	932
(e)	1179	1416	10	570	750	:	3876	396.0	1239	1023

IV Check for One-way Shear - Bending about x-axis (Parallel to short side).

Group.	B2 mm	A_{x2} mm²	A_{stx} (Prov.) mm²	P_{tx} %	V_{ucx} kN	V_{uDx} kN	# - No mm	Sx mm
(a)	767	188001	502	0.267	67.72	60.50	8 - 10	117
(b)	867	233150	804	0.345	93.70	93.84	8 - 16	83
(c)	917	241542	653	0.27	87.40	86.41	8 - 13	106
(d)	1163	359232	1256	0.315	145.50	144.53	10 - 16	108
(e)	1263	394806	1256	0.32	153.97	151.19	10 - 16	115

to the following groups:-

lumn Size	Column Mark
25 x 225	C27, C28
25 x 450	C13,C19,C21
25 x 600	C7
25 x 675	C15,C23
25 x 675	C14,C22

capacity of 250 kN/m² as per
iven in the tabular form and
en given.

ov.A_f m²	Wu kN/m²	Proj-x mm	Proj-y mm
1.21	330.60	437.5	437.5
1.98	332.91	537.5	537.5
2.25	319.64	550	550
3.56	337.08	725	725
3.95	334.23	775	775

ent Considerations :

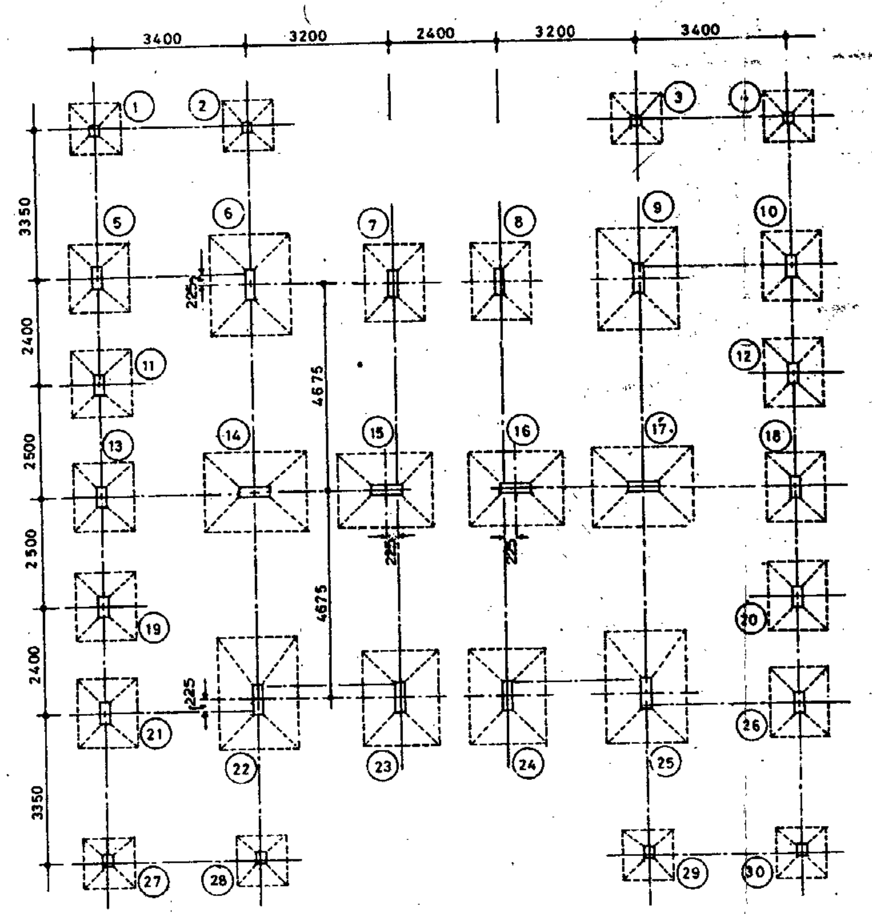
Total D_f mm	Provided dx mm	dy mm	$D_{f.min}$ mm
325	271	263	150
375	321	313	150
400	346	338	150
525	470	460	150
575	520	510	150

o-way Shear

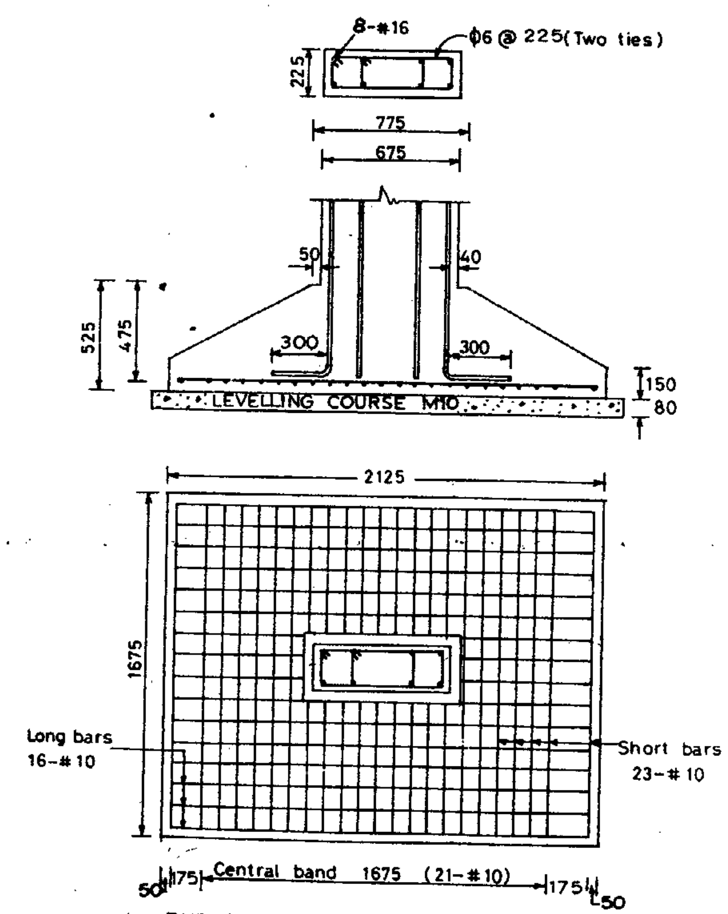
2(B1+L1) mm	d1 mm	Vuc kN	VuD kN
1952	238.5	451	321
2602	271.9	685	523
3002	286.5	729	551
3676	327.0	966	932
3876	396.0	1239	1023

-axis (Parallel to short side).

Vucx kN	VuDx kN	# - No mm	Sx mm
67.72	60.50	8 - 10	117
93.70	93.84	8 - 16	83
87.40	86.41	8 - 13	106
145.50	144.53	10 - 16	108
153.97	151.19	10 - 16	115



PLAN OF FOUNDATION



TYPICAL COLUMN FOOTING (C23)

Schedule of Columns and Footings

Column No. Between Storey	C1, C2, C3, C4, C27, C28, C29, C30	C5, C10, C11, C12, C13, C18, C19, C20, C21, C26	C7, C8	C14, C17	C15, C16	C6, C9, C22, C25	C23, C24
R-3	150 x 225 4- # 12 ϕ 6 @ 150	150 x 225 4- # 12 ϕ 6 @ 150	150 x 600 8- # 12 ϕ 6 @ 150	150 x 225 4- # 12 ϕ 6 @ 150	225 x 225 4- # 12 ϕ 6 @ 190	150 x 600 8- # 12 ϕ 6 @ 150	150 x 600 4- # 16 ϕ 6 @ 150
3-2	150 x 225 4- # 12 ϕ 6 @ 150	150 x 300 4- # 12 ϕ 6 @ 150	150 x 600 8- # 12 ϕ 6 @ 150	225 x 300 6- # 12 ϕ 6 @ 190	225 x 300 6- # 12 ϕ 6 @ 190	150 x 600 8- # 12 ϕ 6 @ 150	150 x 600 6-#16+2-#12 ϕ 6 @ 150
2-1	150 x 225 6- # 12 ϕ 6 @ 150	150 x 450 4- # 16 ϕ 6 @ 150	150 x 600 8- # 12 ϕ 6 @ 150	225 x 450 4-#16+2-#12 ϕ 6 @ 190	225 x 450 8- # 12 ϕ 6 @ 190	225 x 600 4-#16+4-#12 ϕ 6 @ 190	225 x 600 6-#16+2-#12 ϕ 6 @ 190
1-PL	225 x 225 6- # 12 ϕ 6 @ 190	225 x 450 4-#16+2-#12 ϕ 6 @ 190	225 x 600 4-#16+4-#12 ϕ 6 @ 190	225 x 600 8-# 16 ϕ 6 @ 225	225 x 600 4-#16+4-#12 ϕ 6 @ 190	225 x 600 8-#16+2-#12 ϕ 6 @ 190	225 x 600 8 # 16 ϕ 6 @ 225
PL-FT	225 x 225 8- # 12 ϕ 6 @ 190	225 x 450 4-#16+2-#12 ϕ 6 9 190	225 x 600 4-#16+4-#12 ϕ 6 @ 190	225 x 675 10- # 16 ϕ 6 @ 225	225 x 675 4-#16+4-#12 6 @ 190	225 x 675 10- # 16 ϕ 6 @ 225	225 x 675 8- # 16 ϕ 6 @ 225
Footing	1100x1100	1300x1525	1325x1700	1775x2225	1675x2125	1775x2225	1675x2125
Df	325	375	400	575	525	575	525
Df-min	150	150	150	150	150	150	150
Long bars	10 - # 8	16 - # 8	13 - # 8	16 - # 10	16 - # 10	16 - # 10	16 - # 10
Short bars	10 - # 6	21 - # 8	20 - # 8	22 - # 10	23 - # 10	22 - # 10	23 - # 10

Notes :
- Concrete Mix M15
- Clear cover for slab Rein. 15mm, Beam 25mm
- Clear cover for column 40mm, Footing 50mm
- # Represents Tor Steel Fe 415 & represents Mild Steel Fe 250
- First stirrup in beam shall be 50mm away from the face of support.

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	Scale as shown	Checked by	Structural Engineer

V Check for One-way Shear - Bending about y-axis (Parallel to long side).

Group.	L2 mm	A_{y2} mm^2	A_{sty} (Prov.) mm^2	P_{ty} %	V_{ucy} kN	V_{uDy} kN	# mm	No	Sy mm
(a)	751	182455	502	0.275	66.60	63.56	8	10	117
(b)	1076	268958	1056	0.392	114.47	113.97	8	21	74
(c)	1276	287453	1005	0.350	116.45	115.23	8	20	87
(d)	1595	445117	1806	0.405	191.40	189.82	10	23	115
(e)	1695	483262	1728	0.358	197.70	197.00	10	22	103

VI Schedule of Footing :

Group	P_u kN	b mm	D mm	B_f mm	L_f mm	D_f mm	$D_{f.min}$ mm	# mm	Nx	Sx mm	# mm	Ny	Sy mm
(a)	400	225	225	1100	1100	325	150	8	10	117	8	10	117
(b)	660	225	450	1300	1525	375	150	8	16	83	8	21	74
(c)	720	225	600	1325	1700	400	150	8	13	106	8	20	87
(d)	1200	225	675	1675	2125	525	150	10	16	108	10	23	115
(e)	1320	225	675	1775	2225	575	150	10	16	115	10	22	103

Chapter - 9

Design of Portal Frame

9.1 INTRODUCTION

A portal frame is a frame having its elements rigidly connected at the joint. 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 singlestorey or multi-storey and as regards to the number of spans they are single span and multispan. Some of the forms of portal frames in R.C. construction are shown in Fig.9.1.

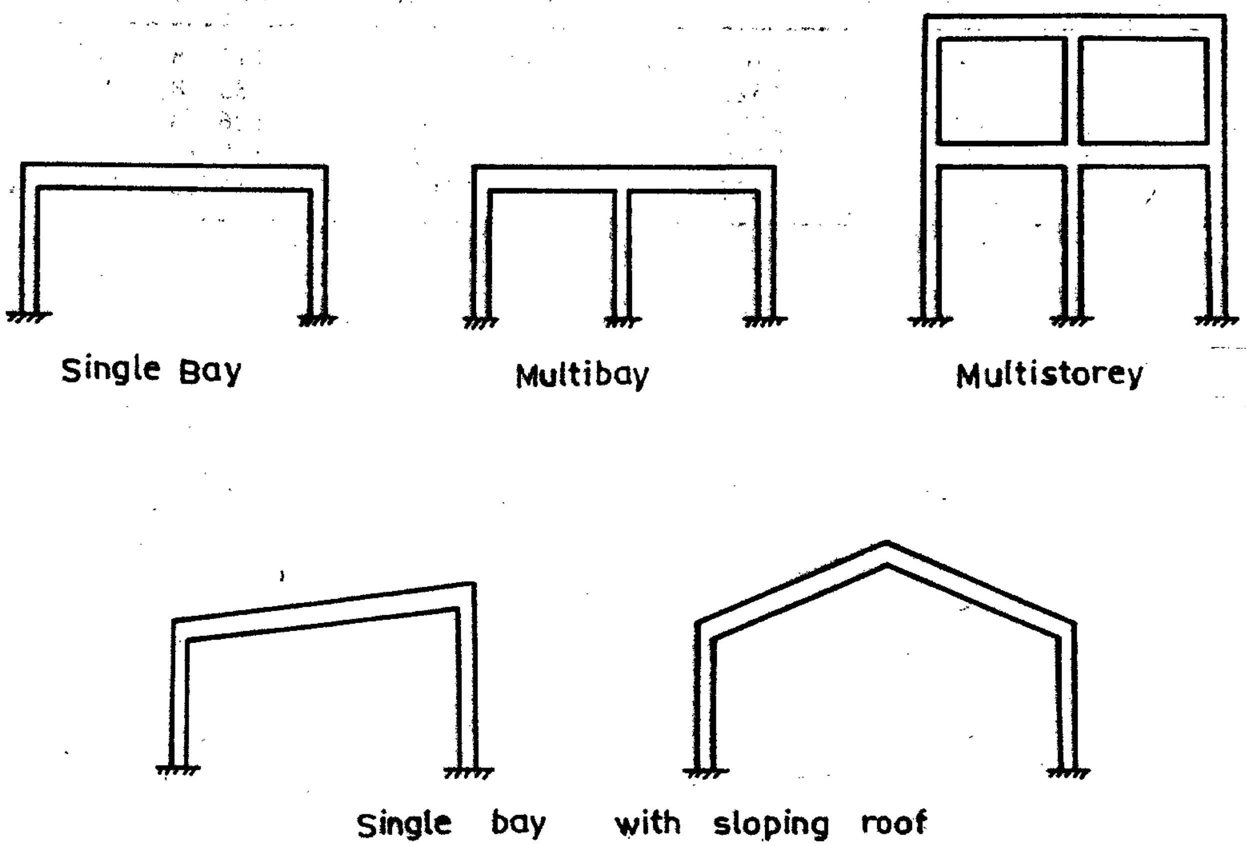


Fig. 9.1 Different types of R.C.Portal Frames.

For ware-houses 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. 9.2.

9.2 ANALYSIS AND DESIGN OF PORTAL FRAMES

9.2.1 Introduction

Single-storey portal farmes with flat roof have span of 12 to 15 metres and spaced at interval of 3 to 4 m. The columns of the frame are generally supported on isolated footing ,in which case the end condition may be taken as hinged or fixed. If the footing is resting on normal soil , 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.

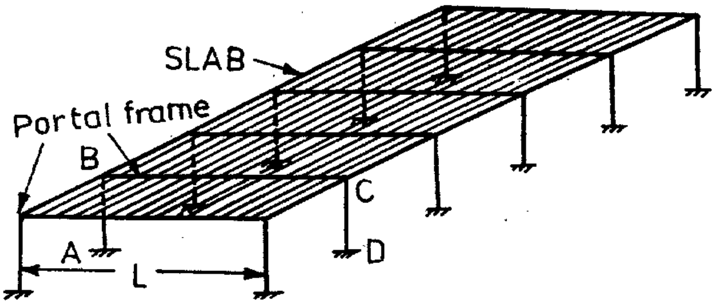


Fig. 9.2 Portal Frame with R.C. Slab Roof

9.2.2 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 becomes more 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 0.5 to 0.6 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 the beam. Now, if the slab is cast monolithically over the beam, the beam acts as a flanged section in the midspan 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.3.2.5). The assumption of rectangular section 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.

9.2.3 Methods of Analysis

The portal frames can be analysed 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 no partical influence on the results.

The usual example of a rigid frame is a symmetrical portal frame shown in Fig. 9.2 . The columns AB and CD have the same cross section and length. If the loading on the beam is symmetrical (say loaded by U.D.L.) 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 analysed 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. The stiffness factor for the fixed based column will be 4EI/L. The example given in part-A of the next section illustrates the method of analysis and design. In part-B the same example is worked out allowing redistribution of moments to understand the economics and advantages of redistribution of moments.

9.3 DESIGN OF FIXED BASE PORTAL FRAME

The roof of an assembly hall 20m long and 12m wide betweencentres of columns is supported by a fixed based R.C.portal frame spaced at 4m apart. The height of the column from top of footing upto the centre of beam is 5.5m. The columns are laterally braced at a height of 2m above plinth level. Design the roof slab and an intermediate portal frame for the following additional data :

Live load	0.75 kN/m^2
Floor finish	1.5 kN/m^2
Depth of foundation	1.5 m below G.L.
Soil bearing capacity	400 kN/m^2
Materials	Concrete M15 steel Fe415
Design philosophy	Limit State Method conforming to IS: 456-1978
Design assumption	All members of the frame are rigid jointed.

(A) Design of Portal Without Redistribution of Moments

(B) Design of Portal With 30% Redistribution of Moments.

DATA :

Effective Span of portal frame		=	12m
Spacing of portal frame		=	4m
Height of Columns above footing		=	5.5m
Live load on roof		=	0.75 kN/m^2
Floor finish		=	1.5 kN/m^2
Height of plinth		=	0.30 m above G.L.
Depth of foundation		=	1.25 m below G.L.
Soil bearing capacity		=	400 kN/m^2
Concrete grade	f_{ck}	=	15 N/m^2
Main steel grade	f_y	=	415 N/m^2

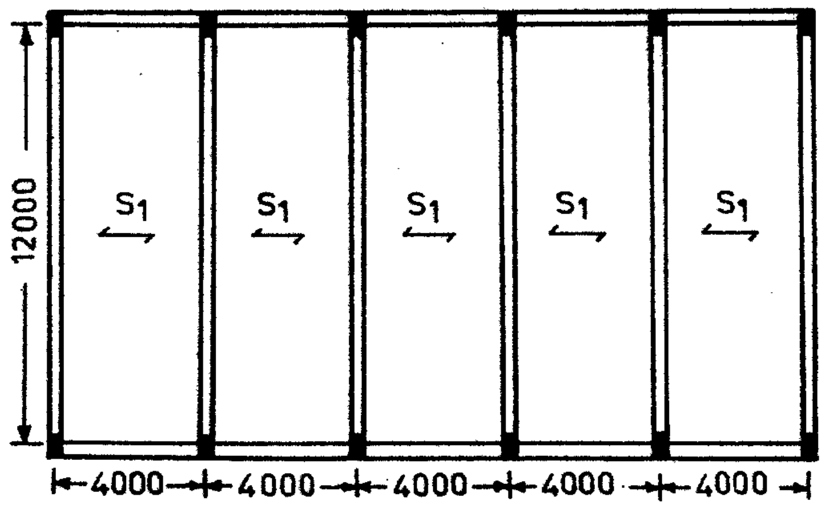


Fig. 9.3 Structural Plan of Assembly Hall

9.3.1 Design of Slab - S1

The slab is designed as a continuous slab supported by portal frames. Thus the span of the slab is 4m. Since the number of spans are more than two, the slab is designed using I.S. code coefficients.

Step No.		Design Calculations
1.	Type	: One-way continuous slab with 4 equal spans.
2.	Span	: $L = 4m = 4000mm$
3.	Imposed load	: Live load, $LL = 0.75\ kN/m^2$ Floor finish, $FF = 1.50\ kN/m^2$
4.	Trial Depth	: This is decided by deflection criteria Basic L/d ratio for continuous slab $= r_b = 26$ Assuming $p_t = 0.3\%$, Modification factor $\alpha_1 = 1.4$ Allowable L/d ratio $= r_a = \alpha_1 r_b = 1.4 \times 26 = 36.4$ Required $d = L/r_a = 4000 / 36.4$ say 110 mm. Assuming effective cover $= 20$ mm Required total depth $= 110 + 20 = 130$ mm.
5.	Loads	Consider one metre width of slab (i.e.b = 1m) Self weight + floor finish $= w_d = 25 \times 0.13 + 1.5 = 4.75\ kN/m$ Live load $w_l = 0.75\ kN/m$ Total working load $w = 5.50\ kN/m$

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	1/24	– 1/12
Live load α_l	1/10	– 1/9	1/12	– 1/9
Bending moment kN.m	11.3	– 13.4	6.25	– 11.5

These ultimate moments at different sections have been calculated using the equation :

$$M_u = 1.5\ (\alpha_l \times w_d L^2 + \alpha_l \times w_l\ L^2)$$

At middle of outer span: (i.e. at Section – 1)

$$M_{u1} = 1.5\ (\ 4.75 \times 4^2/12 + 0.75 \times 4^2/10) = 11.3\ kN.m.$$

At penultimate support: (i.e. at Section – 2)

$$M_{u2} = 1.5\ (4.75 \times 4^2/10 + 0.75 \times 4^2/9) = 13.4\ kN.m.$$

At middle of inner span: (i.e. at Section – 3)

$$M_{u3} = 1.5\ (4.75 \times 4^2/24 + 0.75 \times 4^2/12) = 6.25\ kN.m.$$

At intermediate support : (i.e. at Section – 4)

$$M_{u4} = 1.5\ (\ 4.75 \times 4^2/12 + 0.75 \times 4^2/9) = 11.5\ kN.m.$$

Absolute maximum B.M. = $M_{u.max}$ = 13.4 kN.m.

7. Check for Concrete Depth:

$M_{ur.max} = R_{u.max}\ bd^2$, For slab, b = 1000 mm.

$M_{ur.max} = 2.07 \times 1000 \times 110^2 \times 10^{-6} = 25.05$ kN.m > 13.4 kN.m ∴ .O.K.

8. Main steel:

This is obtained at 4 different sections using equation :

$$A_{st} = (0.5\ f_{ck}\ /\ f_y)\ [1 - \sqrt{1 - 4.6M_u / (f_{ck}\ bd^2)}] \times bd$$

$$= (0.5 \times 15 / 415)\ [1 - \sqrt{1 - 4.6\ M_u / (15 \times 1000 \times 110^2)}] \times 1000 \times 110.$$

The area of steel required is shown in step 11.

9. Check for Deflection:

Required maximum A_{st} at midspan = A_{st1} = 308 mm^2

Required p_t = 308 x 100 / (1000 x 110) = .28% < 0.3% assumed. ∴ .O.K.

10. Distribution Steel:

For Fe415 grade, A_{st} = 0.12% of bD = .12 x 1000 x 130/100 = 156 mm^2

Provide # 8 at 320mm c/c, Area provided = 157 mm^2 > 156 mm^2

11. Detailing of Reinforcement:

Section	1	2	3	4
Required A_{st} in mm	308	373	164	314
Diam and spacing	#8-160	#8-320 + #8-600	#8-300	#8-600 +#8-600
Provided A_{st} in mm^2	314	241	167	167
Required extra A_{st} in mm^2	_	132	_	147
Diam & spacing of extra bars	_	#8-300	_	#8-300
Provided extra steel in mm^2	_	167	_	167

Note : Spacing of extra steel has been provided at 300mm c/c instead of required spacing of 340mm for ease of placing of bars to save the labour.

9.3.2 Determination of Cross-Sectional Dimensions

Step No.	Design Calculations
1. Span :	Beam span = 12m, Column height = 5.5 m.
2. Loads from slab :	For intermediate portal ,
	Slab left = s_1 , w_{s1} = 0.5 w L_x = 0.5 x 5.5 x 4 = 11 kN/m
	Slab Right = s_1 , w_{s2} = 0.5 w L_x = 0.5 x 5.5 x 4 = 11 kN/m
	Total working load transferred from slab = 11 + 11 = 22 kN/m

3. Beam Section : Assume $b = 250mm$.

Depth of beam is worked out based on Span/Depth ratio and support moment equal to 0.5 times simply supported moment for superimposedload transferred from slab.

Assume L/D = 15 , Depth of beam = 12000/15 = 800mm

Approximate support moment = $0.5 \times (1.5 \times 22 \times 12^2 / 8) = 297$ kN.m.

Depth for balanced section = $d = \sqrt{297 \times 10^6 / (2.07 \times 250)} = 758mm$

Assume beam section 250mm x 750mm.

Comments : Since the moment of resistance varies with square of depth it is advantageous to adopt more depth than 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, eventhough it is uneconomical.

4.Total Load: Beam self weight = $25 (0.75 - 0.13) \times 0.25 = 3.87$ kN/m

Total Ultimate Load = $w_u = 1.5 (22 + 3.87) = 38.8$ kN/m = say 39 kN/m.

Fixed end moment = $w_u L^2/12 = 39 \times 12^2/12 = 468$ 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. 9.2.2). 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 moment distribution process is given for one case. The advantage of odd span symmetry is taken and the rotational stiffness factor for beam is taken to be equal to 2EI/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/5.5=0.727	.894	.81
	BC	2EI/12 =0.167		.19

* R.S.F. = Rotational Stiffness Factor and * D.F. = Distribution Factor.

MEMBER	AB	BA	BC	M_E
Distribution factors	---	.81	.19	---
Initial F.E.M. Distributed moments Carry over moment	--- 189.5	--- +379	–468 +89	
Final moments	189.5	379	–379	323

Mid-span moment = $w_u L^2 / 8 - 379 = 39 \times 12^2 / 8 - 379 = 323$ kN.m

The results of all cases are given in the following table :

CASE	I_{BC} / I_{AB}	M_{AB}	M_{BA}	M_{BC}	M_E
I	1	189.5	379	–379	523
II	2	160.5	321	–321	381
III	3	139	278	–278	424

Comments : From the above table it can be seen that if I_{BC}/I_{AB} (i.e.moment of inertia of the beam is equal to moment of inertia of column) is unity, 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 flanged section will not be obtained. On the other hand if I_{BC}/I_{AB} is large (say 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 will act as a singly reinforced section at support and the minimum bottom steel required to be continued to support will not be usful in resisting compression. Further 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 is given by ; $b \times 750^3/12 = 2\ (\ b \times D^3/12\ )$

$D = 750/\sqrt[3]{2} = 595\text{mm}$

Assume column section of 250mm x 600mm

$$\therefore\ I_{BC} / I_{BA} = \frac{b \times 750^3 / 12}{b \times 600^3 / 12} = 1.95$$

Now, for the assumed values of sections of beam and column the design moments are worked out

Joint	Member	R.S.F.	Sum	D.F.
B	BA	4EI / 5.5 = .727	1.052	0.69
	BC	2E (1.95I) / 12 = .325		0.31

MEMBER	AB	BA	BC	M_E
Distribution factors	---	.69	.31	
Initial F.E.M. Distributed moments Carry over moment	--- 161.5	 323 	–468 145 	
Final moments	161.5	323	–323	+379

Mid-span moment = $39 \times 12^2 / 8 - 323 = 379$ kN.m

The details of portal frame and bending moment diagram is shown in Fig. 9.4

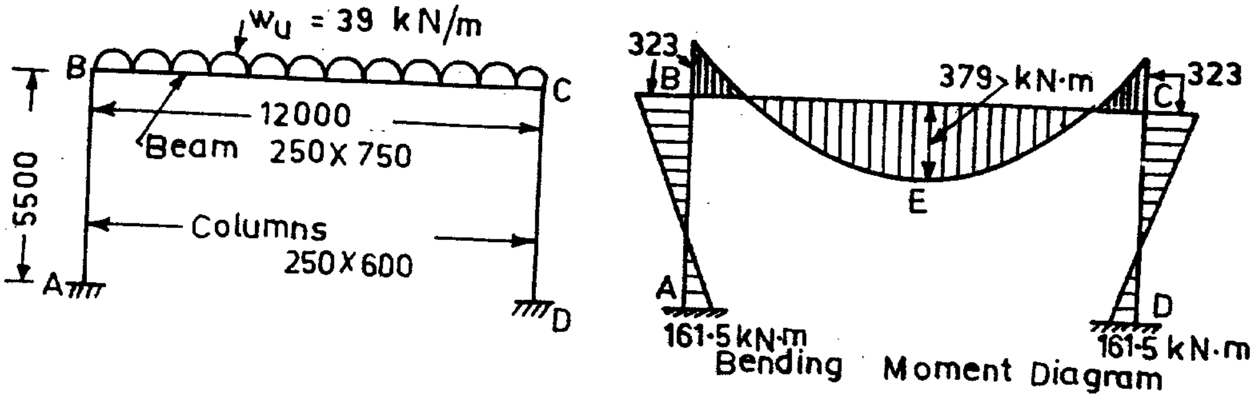


Fig. 9.4 Fixed Base Portal Frame

9.3.3 Design of Portal Without Redistribution of Moments.

Design of Beam

Effective Span = 12 m

Ultimate load w_u = 39 kN/m

Section 250mm x 750mm   Effective depth = 750 − 35 = 715mm.

(a) Mid-span section.

Design sagging moment = 379 kN.m

b_f = 0.7 x 12000 / 6 + 6 x 130 + 250 = 2430mm < c/c spacing (=4000mm)

For $x_u = D_f$,

M_{ur} = 0.36 x 15 x 2430 x 130 (715 −0.42 x 130) x 10^{-6} = 1126 kN.m > 379 kN.m

$\therefore\ x_u < D_f < x_{u.max}$ (=343mm)

Required A_{st} $= (0.5 \times 15/415)\,[1-\sqrt{1-4.6 \times 379 \times 10^6/(15 \times 2430 \times 715^2)}]\ 2430 \times 715$

= 1505 mm^2

Provide 5 - # 20mm , Area provided = 1570 mm^2

Check for width , Required width = 5 x 20 + 6 x 25 = 250mm ∴ O.K

Check for cover , Required cover = 25 + 20 / 2 = 35mm (=assumed value)

Bar curtailment :

Mimimum number of bars required to be extended into support

= A_{st} / 4 = 5 / 4 = 2 Nos.

Curtail 2 bars of 20mm.

No. of bars to be continued into the support = 3

Area of 3 bars of 20mm diameter = 942.48 mm^2

M_{ur} of 3 bars = $0.87\ f_y\ A_{st}\left(d - \dfrac{f_y A_{st}}{f_{ck}\, b}\right)$

= 0.87x 415 x 942.48 x [715− (415x942.48)/(15x250)] x 10^{-6} = 207.8 kN.m

Let x be the distance of point of curtailment from support (TPC)

$234x - 39x^2 / 2 - 323 = 207.8$

$x^2 - 12x + 27.22 = 0$ $\therefore$ x = 3.08m

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 = 3080 – 715 = 2365mm

= say 2.35m from support.

(b) Support section:

Design moment at support = 323 kN.m

$M_{ur.max}$ = 2.07 x 250 x 715^2 x 10^{-6} = 264.56 kN.m < 323 kN.m

∴ Section is doubly reinforced.

Balance moment = M_{u2} = 323 – 264.56 = 58.44 kN.m

Tension Steel:

A_{st1} = 264.56 x 10^6 / [0.87 x 415 x (715 – 0.42 x 0.48 x 715)] = 1283.6 mm^2

A_{st2} = 58.44 x 10^6 / [0.87 x 415 (715 – 35)] = 238.03 mm^2

Total tension steel = A_{st1} + A_{st2} = 1283.6 + 238.03 = 1521.63 mm^2

Provide 5 – # 20mm , Area provided = 1571 mm^2 > 1521.63 mm^2

Compression Steel:

For d_c/d = 35/715 = 0.05 and k_u = 0.48, f_{sc} = 355 N/mm^2

A_{sc} = 0.87 x 415 x 238.03 / (355 – 0.45 x 15) = 246.8 mm^2

Reinforcement available is 3 – # 20 mm i. e. 942 mm^2 ∴ O.K.

Points of contraflexures :

End reaction = 39 x 12 / 2 = 234 kN.

$x_{max} = R_A / w$ = 234/39 = 6m

Points of contraflexure from left support ,

$$x_1 = x_{max} - \sqrt{x^2_{max} - 2M_A / w}$$

$$= 6 - \sqrt{36 - 2x323/39}$$

$$= 1.59 \text{ m.}$$

$$x_2 = 6 + 4.41 = 10.41 \text{ m.}$$

Provide 5– #20mm bars at top of support beyond point of contraflexure for a distance

> d (=715mm)

> 12 x dia of bar (=240mm)

> Clear span / 16 (=705mm)

∴ Provide top bars 5 – #20mm for distance of (1.59 + 0.715 = 2.305 m) 2.3 m from support.

Note : It is possible to curtail approximately 40% of reinforcement at top of support at a distance equal to (TPC/2+d) or L_d whichever is greater. But since the negative moment region will be small appreciable saving in steel will not be obtained. Hence all the bars are terminated at distance of (TPC+d) from support.

Check for Development Length:

$M_1 / V_1 + L_0 > L_d$ or $M_1 = V_1 (L_d - L_0)$

V_1 = shear force at point of contraflexure = 234 – 39 x 1.59 = 171.99 kN.

L_d = 57Ø = 57x20 = 1140 mm.

L_0 = Greater of [12Ø (=240mm) or d(= 715mm.)]

∴ M_1 = 171.99 (1.14 – .715) = 73.1 kN.m

Required number of bars at point of contraflexure

$= N_{max}\ M_1/M_{max} = 5 \times 73.1/323 = 1.13 < 2 \quad \therefore$ O.K.

Design of Shear Reinforcement:

$V_{u.max} = 234$ kN $\quad V_{uD} = 234 - 39(0.6/2 + 0.715) = 194.41$ kN

The area of tension steel available at top of support is 5 – #20mm. But the bottom tension steel available beyond TPC isonly 3 – #20mm Since the zone of shear reinforcement is likely to be greater than the distance of TPC from support only 3 – #20mm givingarea equal to 942.48 mm^2 is taken for computation of shear resisted by concrete (i.e. V_{uc}).

$p_t\% = 100 \times 942.8 / (250 \times 715) = 0.53\%, \quad \tau_{uc} = 0.472\ N/mm^2$

$V_{uc} = 0.472 \times 250 \times 715 / 1000 = 84.37$ kN.

$V_{usv.min} = 0.35 \times 250 \times 715 / 1000 = 62.56$ kN

$V_{ur.min} = 84.37 + 62.56 = 146.93\ kN < V_{uD}\ (=194.41\ kN)$

$V_{us} = V_{uD} - V_{uc} = 194.41 - 84.37 = 110.04$ kN

Assuming #8mm 2-legged stirrups of Fe 415 $(A_{sv} = 100.5\ mm^2)$

Spacing , $s = 0.87 \times 415 \times 100.5 \times 715 / (110.04 \times 1000) = 236mm$ say $= 230mm$

Length of shear zone, $L_{sl} = (234 - 146.93) / 39 = 2.23m >$ TPC (=1.59m)

Provide #8mm 2–legged stirrups at 230mm c/c for distance of 2.3m from support.

Area of tension steel at mid–span $= 1570.8\ mm^2$

$p_t\% = 100 \times 1570.8 / (250 \times 715) = 0.879\%$

$\tau_{uc} = 0.55 + (0.58 - 0.55)(.879 - .8) / 0.1 = 0.574\ N/mm^2$

$V_{uc} = 0.574 \times 250 \times 715/1000 = 102.6$ kN

Length of nominal shear reinforcement zone : $L_{s3} = 0.5 \times 102.6 / 39 = 1.31$ m say 1.3 m.

Length of minimum shear reinforcement zone : $L_{s2} = L/2 - L_{s1} - L_{s3} = 6 - 2.3 - 1.3 = 2.4$ m.

Spacing of # 8mm 2–legged stirrups,

$s = 415 \times 100.5 / (0.4 \times 250) = 417$ mm say 400 mm

Provide # 8mm 2–legged stirrups at 400mm c/c for length of 2.4m and for remaining distance of 1.3m provide 6mm 2–legged nominal stirrups at 450 mm c/c.

Check for Deflection:

Basic L/d ratio = 26

% of steel required at mid span = 1505 x 100/ (2430 x 715) = 0.09%

Modification factor $\alpha_1 = 2$

For flanged section, $b_w/b_f = 250/2430 = 0.1 \quad \therefore \alpha_2 = 0.8$

For span above 10m, $\alpha_3 = 10/\text{span} = 10/12 = 0.833$

Allowable L/d ratio $= 26 \times 2 \times .8 \times .833 = 34.65$

Required effective depth = 12 x 1000 / 34.65 = 345 mm < 715 mm. $\therefore$ Safe

Comments : *Experiments have shown that considerable local compressive stresses appear at the edge of inner corner of column–beam junction. These local stresses get reduced by provision of haunches at the inner corners.*

Design of Column :

(a)Top section :

Ultimate axial load	P_u	= 234 kN
Ultimate moment	M_{ux}	= 323 kN.m.
Section of column		= 250 mm x 600 mm.
Height of column		= 5.5 m
Unsupported length of column		= 5500 – 750/2 = 5125 mm.

Bending about x-axis (bisecting depth of column)

e_{minx} = 5125 / 500 + 600 / 30 = 30.25 mm

M_{minx} = $P_u x e_{minx}$ = 234 x 30.25 / 1000 = 7.07 kN.m < 323 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 unsupported length of column.

L_{eff} = 5125 mm.

L_{eff}/D = 5125/600 = 8.54 < 12 ∴ Column is short.

d_c/D = 50 / 600 = 0.083

$P_u/(f_{ck}\ bD)$ = 234 x 1000 / (15 x 250 x 600) = 0.104

$M_{ux} / (f_{ck}\ bD^2)$ = 323 x 10^6 / (15 x 250 x 600^2) = 0.24

From interaction diagram Chart – 1 and Chart – 2 by interpolation

p/f_{ck} = .134 ∴ p = 0.134 x 15 = 2.01%

∴ A_{sc} = 2.01 x 250 x 600 / 100 = 3015 mm^2

Provide 10 – #20mm bars distributed equally on each face and additional bars 2 – #12mm at mid–depth of section so that the spacing of longitudinal bars measured along the periphery of the column does not exceed 300 mm. Area provided = 3368mm^2

Bending about y-axis (bisecting width of column)

The column is braced at lintel level by beam of size 250mm x 300mm at height of 2m above plinth level. Assuming thickness of footing equal to 0.55 m. unsupported length of column about y–axis = (2 + 1.25 + 0.3 – 0.55) = 3m.

e_{miny} = 3000/500 + 250/30 = 14.33mm < 20mm. ∴ e_{miny} = 20mm

M_{miny} = P_u x e_{miny} = 234 x 20/1000 = 4.68 kN.m.

$L_{effy}/$ b = 3000/250 = 12 ∴ column is short

d_c/b = 50/250 = 0.2

$P_u/\ (f_{ck}\ bD)$ = 234 x 1000/(15 x 250 x 600) = 0.104

$M_{uy}/\ (f_{ck}\ b^2D)$ = 4.68 x 10^6/ (15 x 250^2 x 600) = 0.008 say 0.01

From interaction diagram Chart –4, p/f_{ck} is very small ∴ safe.

(b) Bottom section :

Ultimate axial load = 234 + 25x0.25x0.6 x (5.5 – 0.75/2) x 1.5 = 262.8 kN. say 263 kN

M_{ux} = 161.5 kN.m.

d_c/D = 50 / 600 = 0.083

$P_u/(f_{ck}\ bD)$ = 263 x 1000/(15 x 250 x 600) = 0.1169

$M_{ux}/(f_{ck}\ bD^2)$ = 161.5 x 10^6/(15 x 250 x 600^2) = 0.12

Using interaction diagram Chart–1 & Chart–2

p/f_{ck} = 0.0533 ∴ p = 0.0533 x 15 = 0.8%

Area of steel = 0.8 x 250 x 600 / 100 = 1200 mm^2

Provide4 – #20mm with 2 nos. on each face and additional bars 2 – #12mm at mid-depth so that the spacing of longitudinal bars measured along the periphery of column does not exceed 300mm.

Area provided = 1482 mm^2 > 1200 mm^2

The top section of the column requires: 10nos. of 20mm diameter bars while the bottom section of the column need only 4 nos. of 20mm diameter of bars. Therefore, 6 nos. of 20mm diameter of bars can be theoratically cutailed at a distance of h/3 (= 5.5/3 m) from top where the bending moment will be equal to M/2. Allowing for a bond length of 45 times diameter, the actual point of curtailment will be at a distance of (5500/3 + 750/2 + 45 x 20 = 3108mm) 3.1 m from top.

∴ Curtail 6 – #20mm at 3.1 m from top.

In order to ensure necessary rigidity of a joint, part of the beam reinforcement is inserted into the column not less than 30 times the diameter from the bottom face of the beam, and part of the column reinforcement is embedded in the beam which also party serves as tension reinforcement for support section of the beam.

Transverse Reinforcement :

Minimum diameter = 20/4 = 5mm Using 6 mm lateral ties.

Spacing shall be least of (16 x 20 or 48 x 6 or 250mm)

∴ Provide 6 mm lateral ties at 250mm c/c.

Design of Footing

Ultimate load at column base	$= P_u = 263$ kN
Ultimate moment at column base	$= M_{ux} = 161.5$ kN.m.
Size of column	= 250mm x 600mm
Safe bearing capacity of soil	$= 400$ kN/m^2

(a) Proportioning of base size :

Ultimate load transferred from column at base $= P_u = 263$ kN.

Self weight of footing 10% = 26.3 kN.

Total ultimate load = 289.3 kN.

Safe Bearing capacity of soil	$= f_b = 400$ kN/m^2
Area of footing required	$= A_f = 289.3/(1.5 \times 400) = 0.482$ m^2
Bending moment at column base	$= M_{ux} = 161.5$ kN.m
Eccentricity of load at column base	$= M_{ux}/P_u = e = 161.5/263 = 0.614$ m = 614mm

Provide offset of 100 mm for seating of formwork for column.

Size of plain concrete pedestal = 450 mm x 800 mm x 300 mm deep.

Area provided by pedestal = 0.45 x 0.8 = 0.36 m^2

For concrete pedestal

$$\tan\alpha = 0.9\sqrt{\frac{100q}{f_{ck}} + 1}$$

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/sq.mm

= 263/ (1.5 x 0.36 x 1000) = 0.487 N/mm^2

$\tan\alpha = 0.9\sqrt{(100 \times 0.487 / 15) + 1} = 1.85$

$\tan\alpha$ provided = 300 / 100 = 3 > 1.85 ∴ Safe.

Provide width of footing equal to width of pedestal $= B_f = 450$ mm

Length of footing required $= A_f/B_f = 0.482/0.45 = 1.07$ m.

Provide eccentric footing such that the C.G. of load from column coincides with the C.G. of footing resulting in uniform pressure distribution.

Minimum length of footing to effect uniform pressure distribution

$= 2 \ (e + D/2 + \text{offset})$

$= 2 \ (614 + 600/2 + 100)$

$= 2028$ mm

Provide footing of size 450mm x 2100mm.

Area of footing provided $= 2.1 \times 0.45 = 0.945 \ m^2 > 0.482 \ m^2$

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 $= 6e = 6 \times 0.614 = 3.68$ m.

So, the minimum size of footing required $= 3.7\text{m} \times 0.45\text{m}$,

Area of footing $= 1.665 \ m^2 > 0.945 \ m^2 >> 0.482 \ m^2$

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 equation : $L_f = (2 \ M_u / P_u + 4 \ P_u / (1.5 \times 3 \times f_s \times B_f)$

For $B_f = 0.45$,

$L_f = 2 \times 161.5/263 + 4 \times 263/ (1.5 \times 3 \times 400 \times 0.45)$

$= 2.527$ m = say 2.53 m

Length of footing in contact with soil $= 3(L/2 - e) = 3 \ (2.53/2 - 0.614) = 1.953$m.

Area of footing $= 2.53 \times 0.45 = 1.138 \ m^2 > 0.935 \ m^2$

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 non-uniform.

Minimum projection of the footing beyond column face $= 1050 - 614 - 400 = 36$mm

Maximum projection of the footing beyond column face $c_x = 1050 + 614 - 400 = 1264$mm

Intensity of ultimate soil pressure $= w_u = 263/.945 = 278.3 \ kN/m^2$

(b) Depth of footing from bending moment considerations :

Bending moment at face of pedestal

$M_{ux} = w_u \times B_f \times c_x \times c_x/2$

$= 278.3 \times 0.45 \times 1.264 \times 1.264/2$

$= 100.04$ kN.m.

Required effective depth $= \sqrt{M_{ux}/(R_{u.max} \ b)}$

$= \sqrt{100.04 \times 10^6/(2.07 \times 450)}$

$= 327.71$mm.

Provide total depth of 550mm

Effective depth provided $= d = 550 - 60 = 490$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. 9.5

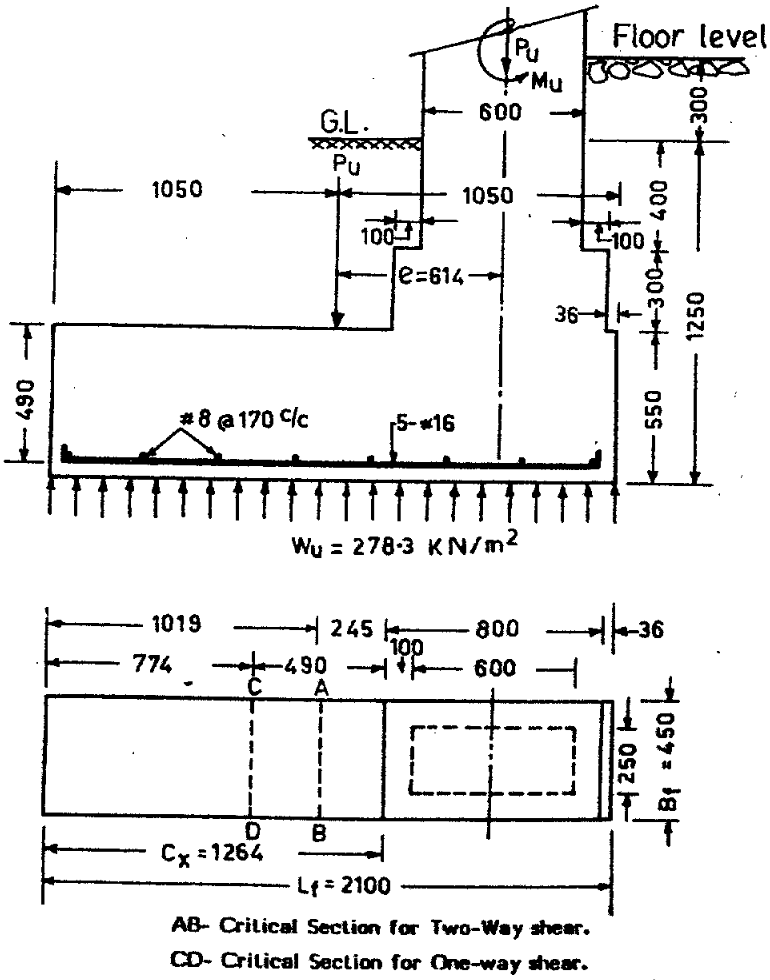


Fig. 9.5 Rectangular Eccetric Pad Footing

(c) *Check for Two-way Shear* :

Critical section for two-way shear at distance d/2(i.e.245 mm) from the face of pedestal.

Perimeter at critical section = width of footing = 450 mm

Area resisting shear = 450x490 = 220500 mm²

Shear strength of concrete :

$$\tau'_{uc} = 0.25\sqrt{15} = 0.968 \text{ N/mm}^2$$

$$k_s = 0.5 + 450/800 <= 1$$

$$\therefore \quad k_s = 1$$

$$\tau_{uc} = \tau'_{uc} \; k_s$$

$$= 0.968 \times 1$$

$$= 0.968 \text{ N/mm}^2$$

Shear resistance of concrete $= V_{uc2} = 0.968 \times 220500/1000$

$= 213.44$ kN

Design Shear = V_{uD2} $= w_u (c_x - d/2) B_f$
$= 278.3 \ (1.264 - 0.245) \times 0.45$
$= 127.61 \text{ kN} < V_{uc2} \ (= 213.44 \text{ kN})$

(d) Area of Steel and Check for Development Length :

Required area of steel $= (.5 \times 15/415) \left[1 - \sqrt{1 - 4.6 \times 100.04 \times 10^6/(15 \times 450 \times 490^2)}\right] 450 \times 490$
$= 612.9 \text{ mm}^2$

Provide 4 - #16mm, Area provided $= 804 \text{ mm}^2 > 612.9 \text{ mm}^2$

Clear distance between bars $= [(450 - 2 \times 40)/(4 - 1)] - 16$
$= 107\text{mm}$

Development required (L_d) requd. $= [0.87 \times 415/(4 \times 1.6)] \times 16$
$= 902 \text{ mm}$

Provide 90^0 bend (L_d) available $= 800 + 8 \times 16 = 928\text{mm} > 902\text{mm}$

(e) Check for One-way Shear :

Critical section at distance d from the face of pedestal = 490mm

Design shear = V_{uD} $= w_u B_f (c_x - d)$
$= 278.3 \times 0.45 \times 0.774$
$= 96.93 \text{ kN}$

Shear resisted by concrete :

$p_t\% = 100 \times 804 / (450 \times 490) = 0.365\%$ (See Table 4.2)

$\tau_{uc} = 0.41 \text{ N/mm}^2$

Shear resistance of concrete section

$V_{uc} = 0.41 \times 450 \times 490 / 1000$
$= 90 \text{ kN} < V_{uD} \ (= 96.93 \text{ kN})$ $\therefore$ unsafe

Increase the steel i.e. Provide 5 – #16mm

Area provided = 1005 mm^2

$p_t\% = 100 \times 1005 / (450 \times 490) = 0.456\%$

$\tau_{uc} = 0.4468 \text{ N/mm}^2$

$V_{uc} = 0.4468 \times 450 \times 490 / 1000$
$= 98.52 \text{ kN} > V_{uD} \ (= 96.93 \text{ kN})$ $\therefore$ Safe

(f) Distribution steel :

Area required = $0.12 \times 450 \times 550 / 100 = 297 \text{ mm}^2$

Provide # 8 @ 170c/c Area Provided = 296 mm^2

Details of footing :

Size of pedestal : 800mm x 450mm x 300mm (deep)
Size of footing : 2100mm x 450mm x 550mm (deep)
Main steel : 5 - #16mm
Distribution steel : #8 @ 170 mm c/c

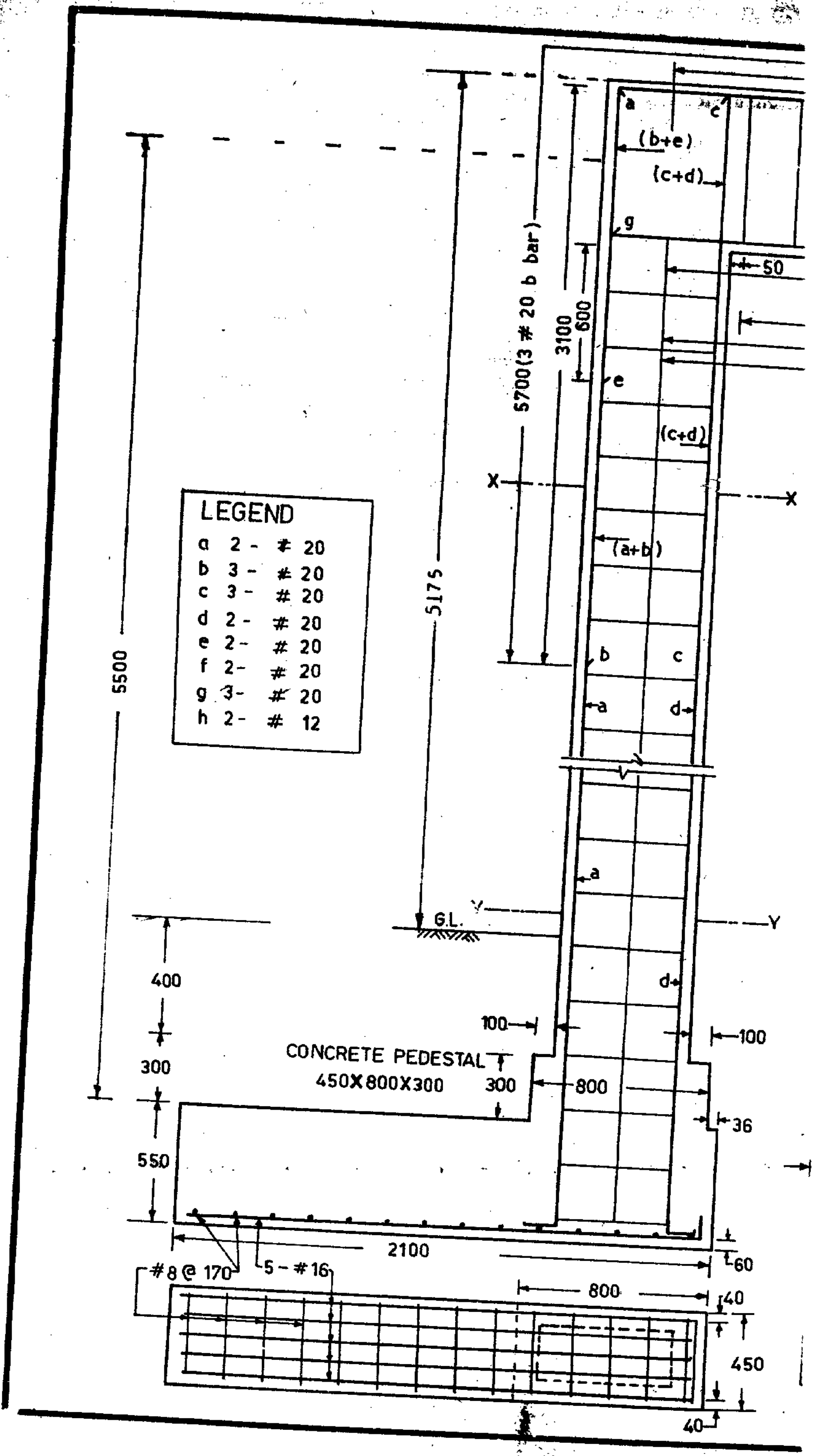
The detailed drawing of fixed base portal frame without redistribution of moments is shown in Fig. 9.6

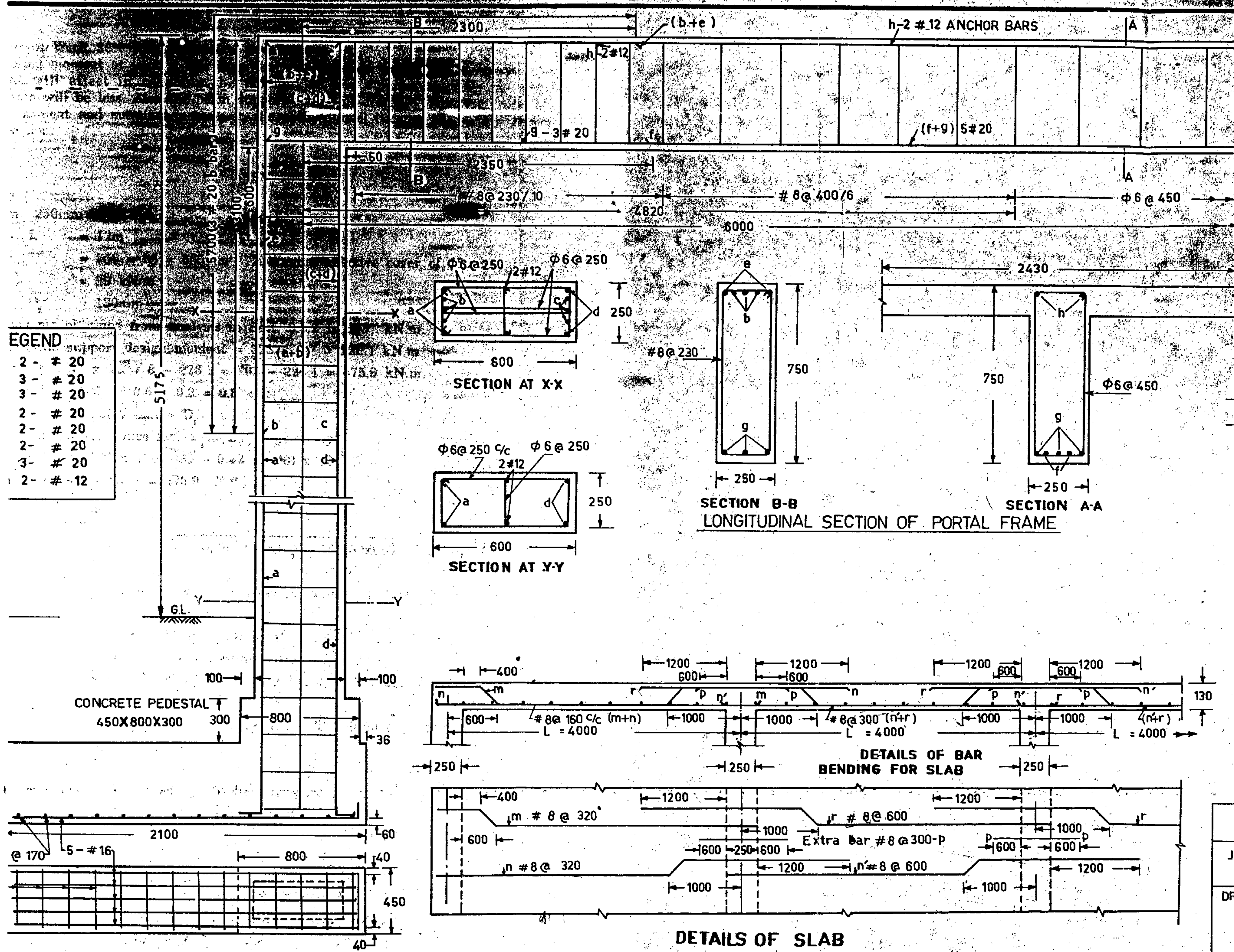
Co

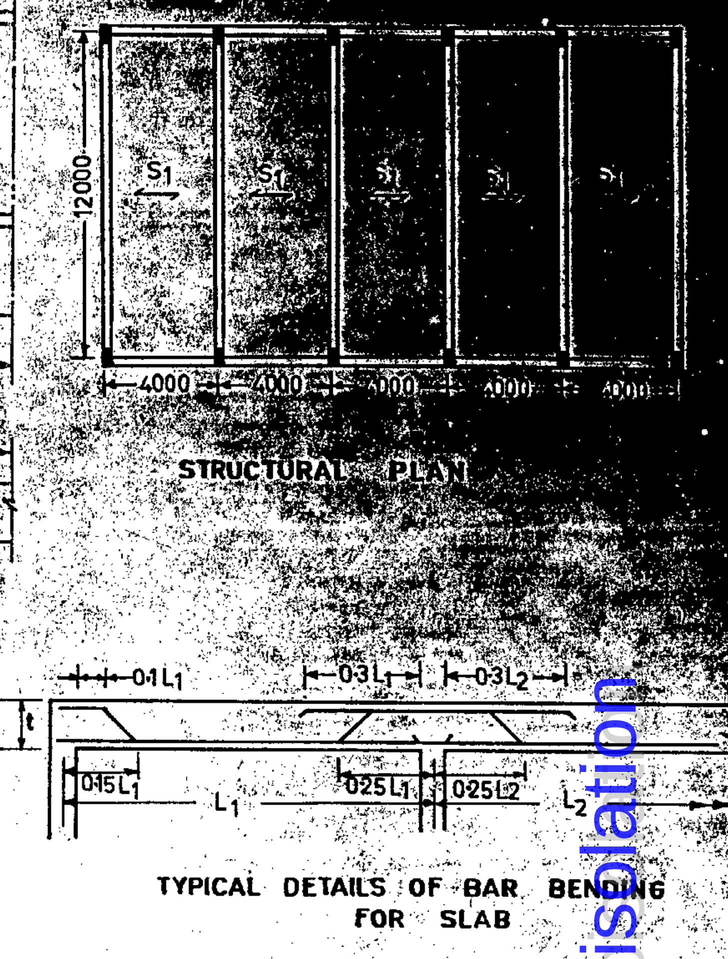
0x490)]^2 450 x 490

ı

ents is shown







NOTES :

1. CONCRETE MIX M 15
2. CLEAR COVER FOR SLAB - 15 mm
   BEAMS - 25 mm
   COLUMNS - 40 mm
   FOOTINGS - 60 mm
3. # REPRESENT TOR STEEL Fe 415
4. Φ REPRESENT MILD STEEL Fe 250
5. FIRST STIRRUP IN BEAMS IS 50 mm AWAY FROM FACE OF SUPPORT
6. FOR P.C.C. USE M 10 CONCRETE
7. ALL DIMENTIONS ARE IN mm

FIG. 9.6

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9.3.4 Design of Portal With 30% Redistribution of Moments :-
Eventhough 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 be less. 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 250mm x 750mm,

Span L = 12m

Effective depth = 750 – 65 = 685mm Assuming effective cover of 65mm.

Ultimate load = 39 kN/m

b_f = 2430mm , D_f = 130mm

The support moments obtained from analysis in Sect. 9.3.2 is 323 kN.m.

When dM = 30%, the support design moment = 323 x 0.7 = 226.1 kN.m

Beam span moment = 39 x 12^2 / 8 – 226.1 = 702 – 226.1 = 475.9 kN.m.

$k_{u.limit}$ = 0.6 – dM / 100 = 0.6 – 0.3 = 0.3 < $k_{u.max}$

$x_{u.limit}$ = 0.3 x 685 = 205.5 mm > D_f

The ultimate moment of resistance for $x_u = D_f$ is given by

(M_{url}) = 0.36 x 15 x 2430 x 130 (685 – 0.42 x 130) x 10^{-6}

= 1075.4 kN.m. > Mu (= 475.9 kN.m)

$\therefore x_u < D_f < x_{u.limit}$

$\therefore$ Section is under-reinforced.

Required A_{st} = (0.5 x 15/415) [1 – $\sqrt{1 - (4.6x475.9x10^6)/(15x2430x685^2)}$] 2430 x 685

= 1991.1 mm^2

Provide 5 – #20mm + 1 – #25mm in two rows, Area provided = 1571 + 491 = 2062 mm^2.

Bar Curtailment :

Curtail 2 – #20mm + 1 – #25mm , Area = 1119 mm^2

Moment of resistance of 3 – #20mm bars continued into support

= M_{ur} = 0.87 x 415 x 942.48 [685 – (415 x 942.48)/(15 x 250)] x 10^{-6}

= 197.6 kN.m.

Let x be the distance of theoretical point of curtailment (TPC) from support

Then, 234 x –39 $x^2/2$ – 226.1 = 197.6

$\therefore$ x = 2.22m.

Distance of Actual Point of Curtailment (APC) from support

= 2.22 – 0.685 = 1.535m = say 1.53m

b) Support section :

Design moment at support = 226.1 kN.m.

$M_{ur.limit} = R_{u.limit}\, bd^2$, $R_{u.limit}$ = 0.36 x 15 x 0.3 (1–0.416 x 0.3) = 1.478 N/mm^2

$M_{ur.limit}$ = 1.4178 x 250 x 685^2 x 10^{-6} = 166.3 kN.m < 226.1 kN.m.

$\therefore$ Section is doubly reinforced.

M_{u2} = 226.1 – 166.3 = 59.8 kN.m.
A_{st1} = 0.36 x 15 x 250 x (0.3 x 685) / (0.87 x 415) = 768.4 mm^2
A_{st2} = 59.8 x 10^6/[0.87 x 415 (685 – 35)] = 254.8 mm^2
Total tension steel = A_{st} = 768.4 + 254.8 = 1023.2 mm^2
Provide 3 – #20mm + 2 – #12mm , Area provided = 1168 mm^2

Compression steel :

For d_c/d = 35 / 685 = 0.051 and $k_{u.limit}$ = 0.3 , f_{sc} = 353 N/mm^2
A_{sc} = 0.87 x 415 x 254.8 / (353 – 0.45 x 15)
= 265.7 mm^2

Reinforcement available at bottom is 3 – #20mm , Area provided > 265.7mm^2
End reaction = 39 x 12 / 2 = 234 kN. , x_{max} = R_A / w_u = 239 / 39 = 6m.

Points of contraflexure :

$$x_1 = 6 - \sqrt{36 - 2 \times 226.1 / 39} = 1.06 \text{ m.}$$

Top tension bars to be continued beyond points of inflection shall be greater of :
d(= 685) or 12 x dia.(= 300) or clear span/16 [= (12000 – 600)/16 = 713)]
Provide top bars 3 – #20mm for distance of (1.06 + 0.713 = 1.773) 1.78m from support and 2 – #12 will be continued throughout the span and will also act as anchor bars.

Design of Shear Reinforcement :

$V_{u.max}$ = 234 kN. , V_{uD} = 234 – 39 (0.6 / 2 + .685) = 195.58 kN.

Comments : Eventhough the reinforcement at support is 3 – #20 mm + 2 – #12mm, only 3 – #20mm is taken for calculation of V_{uc} because the shear zone will be more than the negative moment region and the number of bars available at bottom are only 3 – #20mm upto the point of curtailment.

Area of tension steel at top considered = 3 – #20mm , Area = 942.5 mm^2
p_t% = 100 x 942.5 / (250 x 685) = 0.55%
τ_{uc} = 0.48 N/mm^2
V_{uc} = 0.48 x 250 x 685 / 1000 = 82.2 kN
$V_{usv.min}$ = 0.35 x 250 x 685 / 1000 = 59.94 kN.
$V_{ur.min}$ = 82.2 + 59.94 = 142.14 kN < V_{uD} (=195.58 kN)
Minimum stirrups are not sufficient.
Shear to be resisted by stirrups

= V_{usv} = V_{uD} – V_{uc}=195.58 – 82.2 = 113.38 kN

Using # 8mm 2 – legged stirrups,
Spacing = 0.87 x 415 x 100.53 x 685/(113.38 x 1000)
= 219.3 mm
Provide #8mm 2–legged stirrups at 220 c/c
Length of shear zone, L_{s1} = (234 – 142.14) / 39 = 2.35m > TPC (= 2.22m)

Area of tension steel available beyond shear zone = 5 – #20mm + 1 – #25mm = $2062 mm^2$

$p_t\% = 100 \times 2062 / (250 \times 685) = 1.2\%$

Design shear strength of concrete (for pt = 1.2% & M15) $\tau_{uc} = 0.63 N/mm^2$

Shear resisted by concrete $V_{uc} = 0.63 \times 250 \times 685 / 1000 = 107.88$ kN.

Zone of nominal shear reinforcement = $L_{s3} = 0.5 \times 107.88/39 = 1.38$m.

Zone of minimum shear reinforcement = $L_{s2} = 6 - 2.35 - 1.38 = 2.27$m.

Spacing of #8 mm 2–legged stirrups, s = 415 x 100.5 / (0.4 x 250) = 417mm

Provide #8mm 2–legged design stirrups at 220mm c/c for length of 2.35m and #8mm 2–legged stirrups at 400 c/c for length of 2.27m and for remaining distance of 1.38m provide 6mm 2–legged nominal stirrups at 450mm c/c.

Design of column :

(a) Top Section :

Ultimate axial load P_u = 234 kN

Ultimate moment M_{ux} = 226.1 kN.m.

Section of column = 250mm x 600mm

Height of column = 5.5m

Unsupported length of column = 5.5–0.75/2 = 5.125m.

Bending about x-axis (bisecting depth of column)

$e_{minx} = 5125 / 500 + 600/30 = 30.25$mm

$M_{minx} = P_u \times e_{minx} = 234 \times 30.25/1000 = 7.07$ kN.m < 226.1 kN.m

Assuming , L_{eff} = 5125mm

$L_{eff}/D = 5125/600 = 8.54 < 12$ ∴ Column is short

$d_c/D = 50/600 = 0.083$

$P_u / (f_{ck}\ bD) = 234 \times 1000/(15 \times 250 \times 600) = 0.104$

$M_u/(f_{ck}\ bD^2) = 226.1 \times 10^6/(15 \times 250 \times 600^2) = 0.167$

From interaction diagram Chart – 1 and Chart – 2 by interpolation

$p/f_{ck} = 0.0833$, ∴ $p = 0.0833 \times 15 = 1.25\ \%$

$A_{sc} = 1.25 \times 250 \times 600/100 = 1875\ mm^2$

Provide 6 – #20mm + 2 – #12mm , Area provided = 1885 + 226 = 2111 mm^2

Additional 2 – #12mm bars are provided at mid-depth of the section to limit the spacing of longitudinal bars measured along the periphery of the column.

(b) At bottom section :

Ultimate axial load = 234 + 25 x 0.25 x 0.6 (5.5–.75/2) x 1.5 = 262.8 kN say 263 kN.

Ultimate moment = M_{ux} = 226.1/2 = 113.05 kN.m. > M_{minx}

$d_c/D = 50/600 = 0.083$

$P_u/(f_{ck}\ bD) = 263 \times 1000/(15 \times 250 \times 600) = 0.117$

$M_{ux}/(f_{ck}\ bD^2) = 113.05 \times 10^6/(15 \times 250 \times 600^2) = 0.084$

Using interaction Chart – 1 & Chart – 2

$p/f_{ck} = .0282$, $p = 0.423\% < 0.8\%$ ∴ $p = 0.8\%$

Area of steel = 0.8x250x600/100 = 1200 mm^2

Provide 4 – #20 with 2 Nos. on each face and 2 – #12 at mid section of the column.

Area provided = 1482 mm^2 > 1200 mm^2

Provide 6mm lateral ties at 250 c/c.

Design of Footing :

Ultimate load at column base	= 263 kN
Bending moment at column base	= 113.05 kN.m
Size of column	= 250 mm x 600 mm
Safe bearing capacity of soil	= 400 kN/m^2

(A) Proportioning of Base size :

Ultimate load transferred from column at base = P_u = 263 kN

Add self weight of footing at 10% = 26.3 kN

Total ultimate load = 289.3 kN.

Area of footing required = A_f = 289.3/(1.5 x 400) = 0.482 m^2

Bending moment at column base = M_{ux} = 113.05 kN.m.

Eccentricity of load at base = 113.05 x 1000 / 263 = 430mm

Provide concrete pedestal of size 450mm x 800mm x 300mm deep.

Width of footing = B_f = 450mm

Minimum length of footing = 2 (e + D/2 + 100) = 2 (430 + 300 + 100) = 1660mm

Length of footing required = L_f = A_f/B_f = .482 / 0.45 = 1.07m < 1.66m

Area of footing provided = 1.66 x 0.45 = 0.747 m^2

Projection of footing beyond face of pedestal = c_x = 860mm

Ultimate upward intensity of soil pressure = w_u = 263/0.747 = 352 kN/m^2

b) Depth of Footing from Bending Moment Considerations :

Bending moment at face of pedestal = $w_u \; B_f \; c_x^2/2$

= 352 x 0.45 x $.86^2/2$

= 58.58 kN.m

Required effective depth = $\sqrt{58.58 \times 10^6/(2.07 \times 450)}$ = 251mm

Provide total depth of 450mm , effective depth = 450 – 60 = 390mm

c) Check for Two-way Shear :

Critical section is at distance d/2 from pedestal

Perimeter at critical section = width of footing = 450mm

Area resisting shear = 450x390 = 175500 mm^2

Shear Strength of Concrete :

$\tau_{uc} = k_s \; (0.25 \sqrt{f_{ck}})$

Where , k_s = (0.5 + 450/800) < 1 $\therefore$ k_s = 1

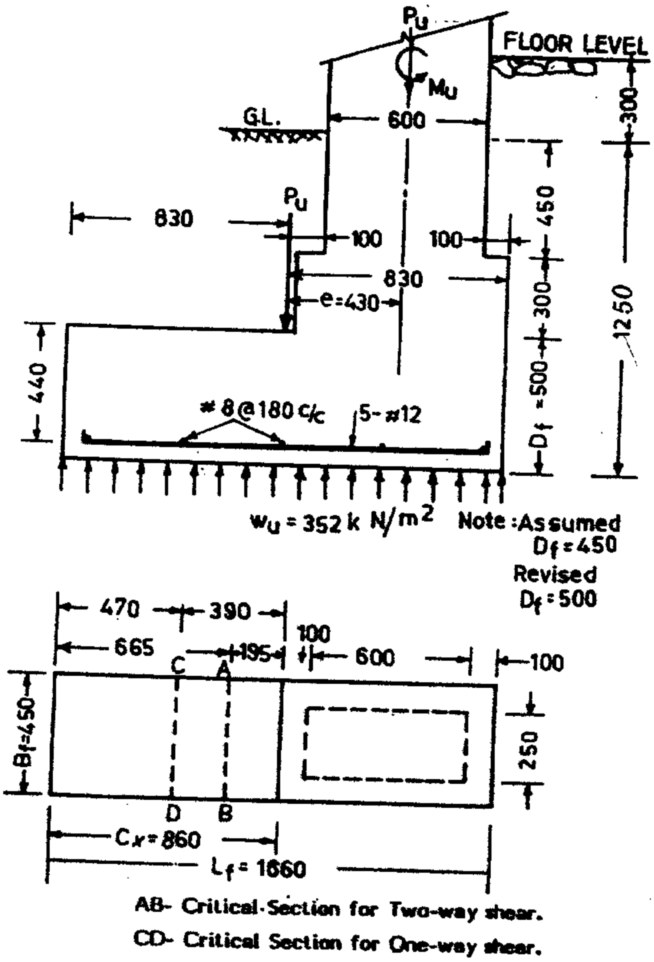


Fig. 9.7 Rectangular Eccentric Footing

τ_{uc2} = 1 $(0.25\sqrt{15})$ = 0.968 N/mm^2

Shear resistance of concrete = V_{uc2} = 0.968 x 175500/1000 = 169.88 kN

Design shear force = V_{uD2} = 352 x 0.45 x (860 − 195)/1000 = 105.34 kN < V_{uc2} ∴ Safe

(d) Area of Steel :

Required area of steel = (0.5x15/415) $[1-\sqrt{1-(4.6\text{x}58.58\text{x}10^6)/(15\text{x}450\text{x}390^2)}]$ 450x390 = 448mm^2

Minimum area of tension reinforcement= .85 x 450 x 390 / 415 = 359mm^2 < 448 mm^2

Provide 5 − #12, Area provided = 565.5mm^2

Clear distance between bars = (450−2x40)/(5−1)−12 = 80mm > 50mm

(e) Check for One-way Shear :

Critical section at distance d from face of pedestal

= 860 − 390 = 470mm

Design shear = V_{uD} = 352 x 0.45 x 0.47 = 74.45 kN

Shear carried by concrete :

p_t% =100 x 565.5/(450 x 390) = 0.32% , τ_{uc} = 0.38 + 0.05 x 0.02/0.1 = 0.39 N/mm^2

Shear resistance of concrete section

= V_{uc} = 0.39 x 450 x 390/1000 = 68.4 kN < V_{uD} (=74.45 kN) ∴ Unsafe

Revise the Depth :

Provide total depth of 500mm , d = 500 – 60 = 440mm

Design shear = V_{uD} = (352 x 0.45) x (860 – 440)/1000 = 66.52 kN

$p_t\%$ = 100 x 565.5/(450 x 440) = 0.28% , τ_{uc} = 0.368 N/mm^2

V_{uc} = 0.368 x 450 x 440/1000 = 72.86 kN > 66.52 kN ∴ Safe

Distribution steel :

Area required = 0.12 x 450 x 500 / 100 = 270mm^2

Provide #8 at 180 c/c , Area provided = 279mm^2

Details of footing :

Size 2100mm x 450mm x 500mm (deep)

Main steel 5 – #12mm

Distribution steel #8 at 180c/c.

9.4 DESIGN OF HINGED BASE PORTAL FRAME

The analysis of a hinged base portal frames is similar to that of fixed base portal frame except that the stiffness of the column willbe 3EI/L instead of 4EI/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 details are left to the reader. The only part that differs from that of fixed base portal frame is 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 in the next section.

9.4.1 Design of Hinge and Foundation

Design the hinge between portal frame and its foundation for the following data.

Ultimate load from column	= 255 kN.
Ultimate shear at hinge	= 57.2 kN.
Distance of hinge from base	= 1.5 m.
Size of column	= 250 mm x 600 mm.

Also design a suitable foundation if bearing capacity of the soil is 200 kN/m^2. Use concrete grade M15 and steel grade Fe 415.

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 = 255 kN.

Area of hinge required = 255 x 1000/(1.5 x 15/2) = 22667mm^2

Width of hinge = Area at hinge/width of column = 22667/250 = 90.7mm.

Provide hinge of size 250mm x 100mm.
Length of the hinge < 2 x smallest dimension at hinge cross-section
Provide Length of hinge = 120mm.
Ultimate shear at the hinge = M_{BA}/L_{AB} = 57.2 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 9.8, the set A will be in tension while the other set B will be in compression.
For equilibrium : $(A_t \sin\phi)\ 0.87\ f_y = H$
Where, A_t = total area of steel at the hinge
ϕ = inclination of reinforcing bars with vertical.
Providing minimum effective cover = 20 mm

$\tan\phi = (50 - 20)/60 = 0.5$, $\phi = 26.56^\circ$, $\sin\phi = 0.447$

$A_t = 57.2 \times 10^3/(.87 \times 415 \times 0.447) = 354.4\ mm^2$

Provide 2 bars of 12mm dia.on each side of hinge and 10mm diameter closely spaced ties or stirrups to effect gradual dispersal of high stresses in constriction.

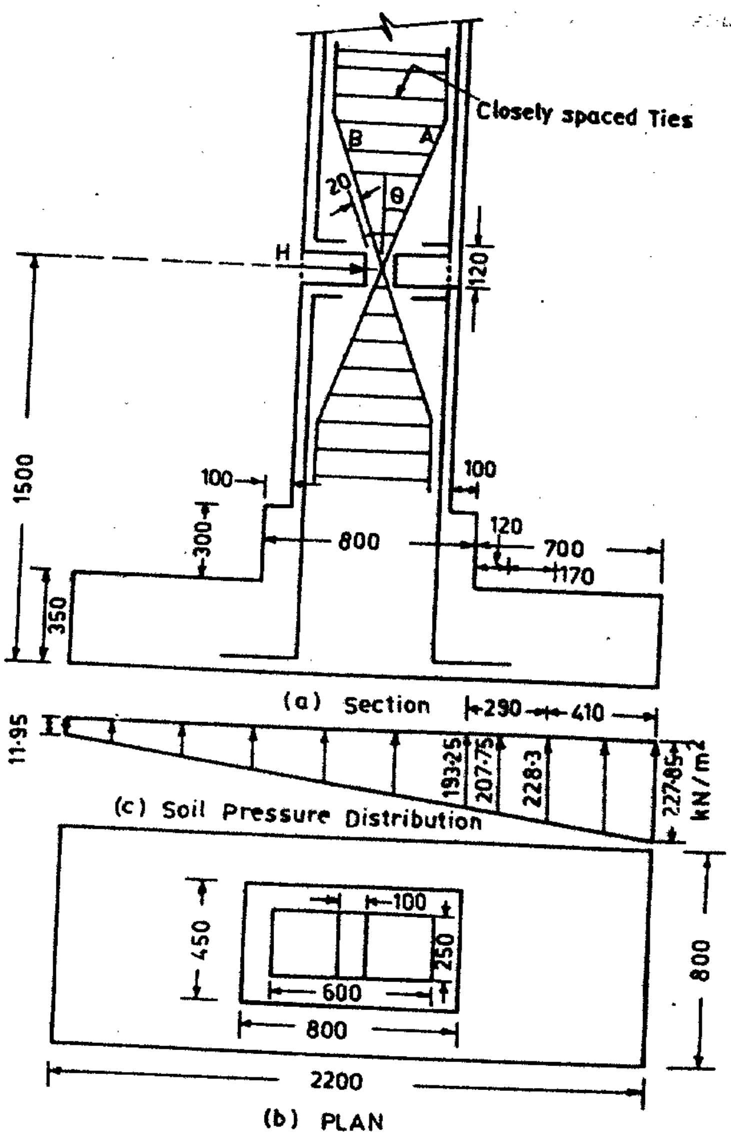


Fig. 9.8 Hinged Base Portal Frame

Design of Footing :

Ultimate load on column $= P_u = 255$ kN.

Add self weight of footing at 10% $= 25.5$ kN.

Total ultimate load $= 280.5$ kN.

Area of footing required $= 280.5/(1.5 \times 200) = 0.935\ m^2$

Ultimate moment at base = Shear at hinge x Distance of hinge from base

$M_u = 57.2 \times 1.5 = 85.8$ kN.m.

Eccentricity $e = M_u/P_u = 85.8 \times 10^3/280.5$

$= 306$ mm

Minimum Length of the footing for no tension condition $= 6e = 6 \times 306$

$= 1836$ 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 0.8 m x 2.2 m

Area of footing provided $= A_f = 0.8 \times 2.2 = 1.78\ m^2 > 0.935\ m^2$

Provide concrete pedestal of size 450mm x 800mm x 300mm deep

Maximum pressure intensity at base

$P'_{u.max} = 280.5/(0.8 \times 2.2) + 85.8/(.8 \times 2.2^2/6)$

$= 159.38 + 132.95 = 292.33\ kN/m^2$

$p'_{u.min} = 159.38 - 132.95 = 26.43\ kN/m^2 > 0$

Maximum intensity of pressure at working load

$p_{max} = 292.33/1.5 = 195\ kN/m^2 <$ Soil Bearing Capacity ($= 200\ kN/m^2$) $\therefore$ Safe

Ultimate upward intensity of soil pressure is given by

$p_{u.max} = 255/(0.8 \times 2.2) + 85.8/(.8 \times 2.2^2/6)$

$= 144.9 + 132.95 = 277.85\ kN/m^2$

$p_{u.min} = 144.9 - 132.95 = 11.95\ kN/m^2$

Maximum pressure intensity at the face of pedestal

$= 277.85 - (277.85 - 11.95) \times 700/2200$

$= 193.25\ kN/m^2$

Depth of Footing from B.M. Considerations :

Ultimate moment at face of pedestal

$= M_{ux} = (193.25 \times 0.7 \times 0.8)\ 0.7/2 + 84.6 \times .7 \times .8 \times 0.7/3$

$= 48.93$ kN.m.

Required effective depth $= \sqrt{48.93 \times 10^6/(2.07 \times 450)}$

$= 229$mm

Provide total depth of 300mm

Effective depth provided $= 300 - 60 = 240$mm

Check for Two-way Shear :

Critical section is at distance d/2 (i.e. 120mm) from face of pedestal

Stress at critical section = 277.85 − (277.85 − 11.95) x 580/2200

= 207.75 N/mm^2

Perimeter at critical section = (1040 + 690) X 2 = 3460 mm

Area resisting shear = 3460x 240 = 830400 mm^2

Shear Strength of Concrete :

$\tau'_{uc} = 0.25\sqrt{15} = 0.968\ N/mm^2$

$k_s = 0.5 + 450 / 800 <= 1$ $\quad\therefore k_s = 1$

$\tau_{uc} = 1 \times 0.968 = 0.968\ N/mm^2$

Shear resistance of concrete = V_{uc2} = 0.968 x 830400/1000

= 803.8 kN.

Design shear = V_{uD2} = (277.85 + 207.75) / 2 x .58 x .8

= 112.66 kN < V_{uc2} (=803.8 kN) ∴ Safe

Area of Steel :

Required area of steel

$= (0.5 \times 15/415)\ [1 - \sqrt{(1 - 4.6 \times 48.93 \times 10^6)/(15 \times 800 \times 240^2)}]\ 800 \times 240$

= 620 mm^2

Provide 4 − #16mm , Area provided = 804 mm^2

Check for one-way shear :

Critical section is at distance d (= 240 mm) from the face of pedestal

Distance of critical section from edge of footing

= 700 − 240 = 460 mm

Stress at critical section = 277.85 − 265.9 x 460/2200

= 222.25 N/mm^2

Design shear force = V_{uD} = (277.85 + 222.25) x 0.46 x 0.8/2

= 92 kN.

$p_t\%$ = 100 x 804/(800 x 240)= 0.42% ,

$\tau_{uc} = 0.436\ N/mm^2$ (See Table 4.2)

Shear resistance of concrete

= Vuc = 0.436 x 800 x 240/1000 = 83.71 kN < 92 kN ∴ Unsafe.

Revise the depth :

Provide total depth of 350mm

∴ d = 350 − 60 = 290mm

Stress at critical section = 277.85 − 265.9 x (700 − 290)/2200

= 228.3 N/mm^2

Design shear force = V_{uD} = (277.85 + 228.3) x 0.41 x 0.8/2 = 83 kN.

$p_t\% = 100 \times 804/(800 \times 290) = 0.346\ \%$

$\tau_{uc} = 403\ N/mm^2$

$V_{uc} = 0.403 \times 800 \times 290/1000 = 93.5\ kN > V_{uD}$

Distribution steel :

Required area $= 0.12 \times 800 \times 350/100$

$= 336\ mm^2$

Provide #8 @ 150 c/c

Details of Footing :

Size of pedestal	: 800mm x 450mm x 300mm deep
Size of footing	: 2200mm x 800mm x 350mm deep
Main Steel	: 4 – #16mm.
Distribution steel	: #8 @ 150 c/c

Design of Porch.

10.1 INTRODUCTION

Porch is normally provided over an entrance of a building with slab projecting out for parking of car or from aesthetics considerations or for using it for seat out. The porch slab is normally provided at lintel level that is at about 2.2 m from the ground floor level. Sometimes it is also provided at the floor level. The projection of the slab is about 2.5 m from the outer face of the wall.

10.2 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. 10.1a).

(b) Cantilever slab supported over beam, which is rigidly connected with columns (Fig.10.1b).

(c) Slab simply supported on the beams with supporting end-beam resting on cantilever end of floor beams (Fig. 10.1c).

(d) Slab simply supported over the cantilever portion of floor beam (Fig. 10.1d).

(e) Slab supported along its edges by beams (Fig. 10.1e).

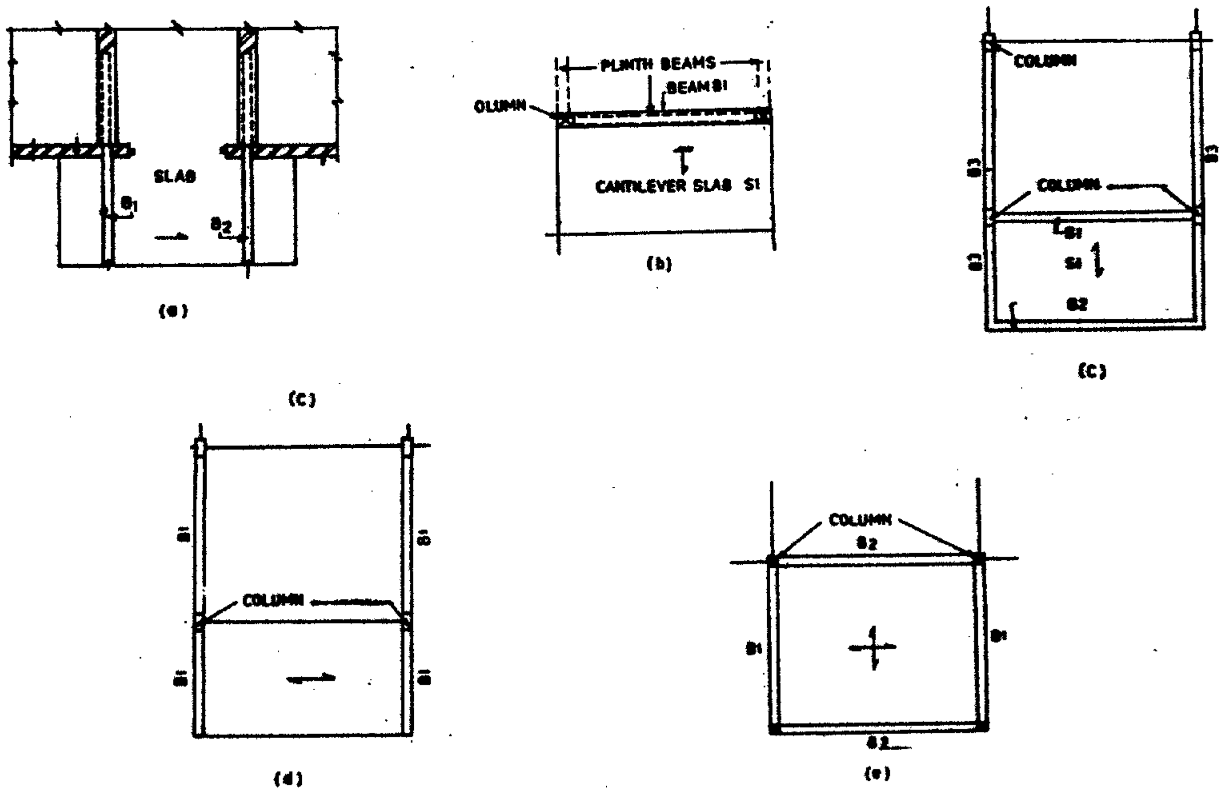


Fig. 10.1 Different layouts of Porch

10.2.1 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. 10.1a). The overhanging portion of the slab helps to reduce the midspan 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 counter weight required 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.

10.2.2 Cantilever Slab Supported over Beams which are Rigidly Connected with Columns (Fig. 10.1b)

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. 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 the 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.

10.2.3 Slab Simply Supported on Beams with Supporting End-beam Resting on Cantilever ends of Floor Beams

Fig. 10.1c 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.

10.2.4 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. 10.1d. 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. 10.2.3 and will carry the load as detailed therein with the change that the cantilever portion of the beams B3 will carry a uniformly distributed load transferred from the porch slab.

10.2.5 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. 10.1e.This type of layout is very simple but generally not preferred from aesthetics considerations. This is suitable when very large size of porch is required to be provided.

10.3 ILLUSTRATIVE EXAMPLES

Ex. 10.3.1 A cantilever porch of size 2500 mm wide x 5000 mm long 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 3m. centre to centre so that the slab overhangs for a length of lm 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. 10.2)

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 250 mm thick.

Assume the following data:

Live load = 0.75 kN/m^2, Load due to finish = 0.8 kN/m^2
Concrete grade M20 and Steel grade Fe 415

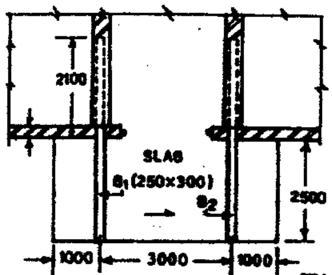


Fig. 10.2

Solution :

f_{ck} = 20 N/mm^2, f_y = 415 N/mm^2, LL = 0.75 kN/mm^2,

FF = 0.8 kN/m^2, $k_{u \cdot max}$ = 0.48, $R_{u \cdot max}$ = 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 = 1.6
   Basic L/d = 26 , Allowable L/d = 26 x 1.6 = 41.6
   Required effective depth = 3000/41.6 = 72 mm say 80 mm
   Provide total depth = 100 mm ∴ d = 100 – 20 = 80 mm
3. *Loads* : Consider lm width of the slab i.e. b = 1000 mm
   Working load/m = D.L. of slab+FF+LL = 25x0.10+.8+.75
   = 4.05 kN/m
   Ultimate load = w_u = 1.5 x 4.05 = say 6.1 kN/m

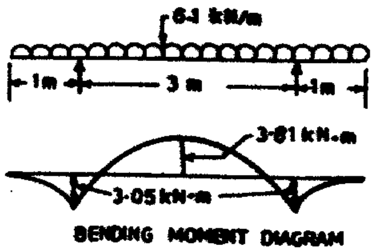


Fig. 10.3

4. *Design Moments* :
   Maximum –ve moment at support :
   $= w_u L_1^2/2 = 6.1 \times 1^2/2 = 3.05$ kN.m.
   Support reaction $= R_A = 6.1 \times 5/2 = 15.25$ kN
   Maximum +ve moment at midspan :
   $= 15.25 \times 3/2 - 6.1 \times 2.5^2/2 = 3.81$ kN/m.
   Point of contraflexure : Let the distance of point of contraflexure be x from A
   $M_x = R_A.x - w_u(1+x)^2/2 = 0$ ∴ $15.25 \times x - 6.1 \times (1+x)^2/2 = 0$
   ∴ x = 0.38 m or x = 2.62 m
5. *Check depth from B. M. Criteria* :
   $M_{ur \cdot max} = R_{u \cdot max}\, bd^2 = 2.76 \times 1000 \times 80^2 \times 10^{-6} = 17.66$ kN m >> 3.81 kN.m
   ∴ Section is under–reinforced.
6. *Main Steel* :
   Area of Steel of midspan $= A_{st} = \dfrac{0.5 \times 20}{415}\left[1-\sqrt{1-\dfrac{4.6 \times 3.81 \times 10^6}{20 \times 1000 \times 80^2}}\right] 1000 \times 80$
   $= 136$ mm^2
   Minimum reinforcement $= A_{st.min} = 0.12 \times 1000 \times 100/100 = 120$ mm^2

Using #8 mm bars (Area = 50 mm^2)

Spacing = 1000 x 50/136 = 367 mm < = (450 mm or 3d = 240 mm) whichever is less
$\therefore$ Spacing = 240 mm $\therefore$ Provide #8 mm bars at 240 mm c/c.

$$\text{Area of steel at support} = \frac{0.5 \times 20}{415}\left[1-\sqrt{1-\frac{4.6 \times 3.05 \times 10^6}{20 \times 1000 \times 80^2}}\right]1000 \times 80$$

$$= 108 \text{ mm}^2 < A_{st.min} (= 120 \text{ mm}^2)$$

$$\therefore A_{st} = 120 \text{ mm}^2$$

Development length = 57ϕ = 57 x 8 = 456 mm.

Provide #8mm bars at 240mm 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 500 mm (> L_d) from the centre of support.

7. *Distribution Steel* :

Ast = 1.2D = 1.2 x 100 = 120 mm^2
Using #8 mm bars, spacing = 1000 x 50/120 = 416 mm < = 5d (= 400 mm)
Provide #8mm bars at 400mm c/c.

8. *Check for deflection*

(a) At mid–span :

$(p_t)_{requd}$ = 100 x 136/(1000 x 80) = 0.17 % < 0.2% $\therefore$ Safe.

(b) At support :

Basic L/d = 7, $(p_t)_{support}$ = 100 x 108 / (1000 x 80) = 0.135%
Modification factor = 1.8.
$\therefore$ $(d)_{requd}$ = 1000/ (7 x 1.8) = 79 mm< 80 mm. $\therefore$ Safe.

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 a higher nominal shear stress and hence is safe against shear.

b. Design of Beam

1. *Span* :

Cantilever span = L = 2.5 m = 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, the beam will act as a T-beam.

Assume the depth of the beam = D = 300 mm, Assuming effective cover = 35 mm,
Effective depth = d = 300 – 35 = 265 mm.
Width of beam = Thickness of wall = 250 mm.
$\therefore$ Assume 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 + .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

$\therefore b_f = 2500/6 + 6 \times 100 + 250 = 1266$ 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)

$\therefore xu < D_f$

5. *Main Steel* :

$$Ast = \frac{0.5 \times 20}{415}\left[1-\sqrt{1-\frac{4.6 \times 53.44 \times 10^6}{20 \times 1266 \times 265^2}}\right] 1266 \times 265$$

= 580 mm^2

$A_{st.min} = 0.85 \times bd/f_y = 0.85 \times 250 \times 265/415 = 136$ mm^2 < Ast. (= 580 mm^2)

Provide 3 bars of # 16 mm, Area provided = 603 mm^2 > 580 mm^2

Check for the width : Required width = (3 x 16 + 4 x 25) = 148 mm < 250 mm.
All bars can be accommodated in one row.

Curtailment of steel :

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 x 2/3 = 35.62 kN.m

$M_{ux} = w_u\ x^2/2 = 17.10 \times x^2/2 = 35.62$

$\therefore x = 2.04$m.

Hence, curtail 1 bars at a distance = [(2500 – 2040) + 12 x 16]
= 652 mm

Curtail 1 bars at a distance of 920 mm [(= 652 + 265) > L_d] 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.

Now, the stabilizing load on the beam from slab of adjoining rooms and walls is given to be equal to 35 kN/m.

Stabilizing moment = (35 x x) x 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

$17.5x^2 = 2 \times 35.63 \quad \therefore x = 2.02$ 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

$\therefore p_t\% = 100 \times 603/(250 \times 265) = 0.9\%, \quad \tau_{uc} = 0.6$ N/mm^2 for M20 (Table.4.2)

$\therefore V_{uc} = \tau_{uc}$ bd = 0.6 x 250 x 265/1000 = 39.75 kN

Shear resisted by minimum stirrups

$V_{usv.min}$ = 0.35 x 250 x 265/1000 = 23.19 kN.

$\therefore V_{ur.min} = V_{uc} + V_{usv.min}$ = 39.75 + 23.19 = 62.94 kN > $V_{u.max}$ (= 42.75 kN)

$\therefore$ Minimum shear reinforcement is sufficient.

Using ø6 mm 2-legged stirrups of grade Fe 250

Spacing = 250 x (2 x 28)/ (0.4 x 250) = 140 mm < = (450 mm or 0.75 x 265)

$\therefore$ Provide ø6mm 2-legged stirrups at 140 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. $A_{st} = 2 \times 201 = 402$ 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, $\therefore$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

$\therefore$ Required d = 2500/(7 x 1.85 x 0.8) = 241 mm < 265 mm $\therefore$ Safe.

The details of reinforcement are shown in Fig. 10.4

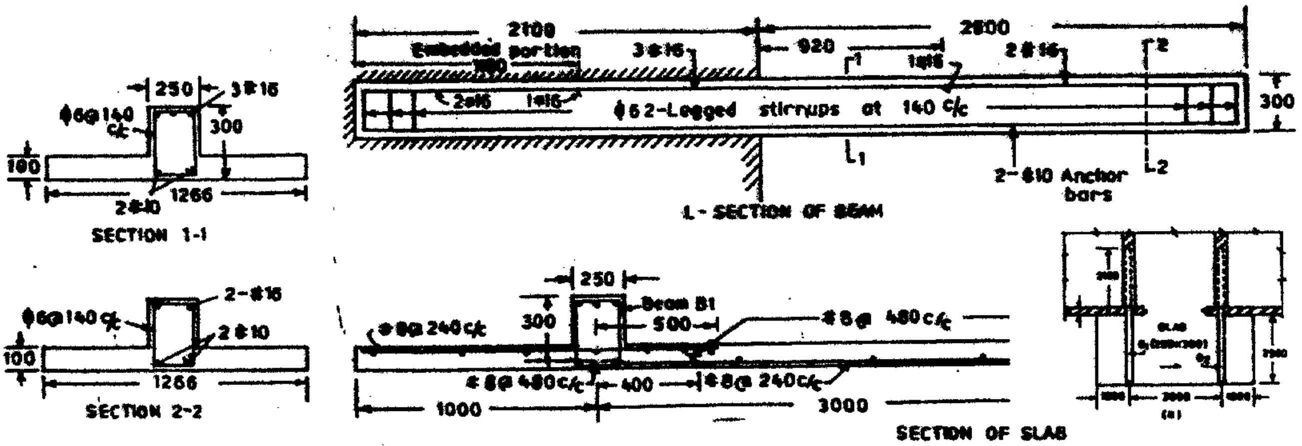


Fig. 10.4

Ex. 10.3.2 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, overhangs 2500 mm beyond the face of the beam. The beam is supported by columns and the construction is monolithic. (See Fig. 10.5)

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,

Floor to floor height is 3.4 m. Plinth beams are provided in both orthogonal directions (See Fig. 10.5)

Concrete grade M20 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)

C. 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 = $\alpha_1 \times 7 = 2 \times 7 = 14$

   Required d = L/14 = 2500/14 = 178.6 mm.

   Assuring 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 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 (since unapproachable) = 0.75 kN/m

   Total = 5.3 kN/m

   $\therefore$ Ultimate load/m = w_{u1} = 5.3 x 1.5 = 7.95 kN/m.

   Maximum −ve moment at support = $M_u = w_{u1} L_1^2/2 = 7.95 \times 2.5^2/2 = 24.84$ kN.m

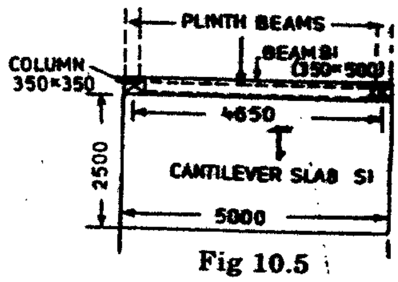


Fig 10.5

5. *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

   $\therefore$ Section is under-reinforced.

6. Area of Steel

$$A_{st} = \frac{0.5 \times 20}{250}\left[1-\sqrt{1-\frac{4.6 \times 24.84 \times 10^6}{20 \times 1000 \times 180^2}}\right] 1000 \times 180 \quad = 666 \text{ mm}^2$$

Using ø10 mm bars, area = 78.5 mm^2

Spacing = 1000 x 78.5/666 = 117 mm say 110 mm < = (450 mm or 3d = 540 mm)

Provide ø 10mm bars at 110 mm c/c

Area provided = 1000 x 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.

$(M_u)_{x=1.6} = 7.95 \times 1.6^2/2 = 10.176$ kN.m

Total depth of the slab at 1.6 from free end = 100 +1.6 x (200 − 100)/2.5 = 164 mm.

$\therefore$ d = 164 − 20 = 144 mm.

Required $A_{st} = 0.5 \times 20/250\ [1 - \sqrt{1 - 4.6 \times 10.176 \times 10^6/(20 \times 1000 \times 144^2)}\ ]\ 1000 \times 144$

$= 335\ mm^2 < (1/2 \times 713)$ 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 + 12ø = 900 + 120 = 1020 mm.

b) 900 + d = 900 + 144 = 1044 mm.

$\therefore$ Curtail 50% steel at a distance of 1050 mm from the face of support.

7. *Distribution Steel* :

Area required = 1.5 D = 1.5 x (200 + 100)/2 = 225 mm^2

Using ø6mm @ 120 mm c/c.

8. *Check for deflection* :

p_t = 100 x 666/ (1000 x 180) = 0.37%, < 0.4% $\therefore$ Safe.

9. *Check for development length* :

Required L_d = [0.87 x 250/(4 x 1.2)] ø = 45.3ø say 46ø

= 46 x 10 = 460 mm.

a) The available length of curtail bars from support upto 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.

(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 midspan region. Since the beam is subjected to equilibrilum 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 $w_uL^2/16$ at support and midspan assuming partial fixity.

1. *Span* : L_2 = 5000 − 350 = 4650 mm = 4.65 m

Assume the size of the beam = 350 mm x 500 mm $\therefore$ d = 500 − 35 = 465 mm.

2. *Loads (Ultimate)* :

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,

$M_u = w_u L_2^2/16 = 26.5 \times 4.65^2/16 = 35.81$ 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

$\therefore$ Torsional moment at support $= T_u = 28.32 \times L_2/2 = 28.32 \times 4.65/2$
$= 65.84$ kN.m

Note **: 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 = 2.2 m
Rotational 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

Lenght 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 = 350 mm = 500 mm , L = 4650 mm

a) Torsional stiffness of beam $= K_t = \dfrac{T}{\phi} = \dfrac{\beta D b^3 G}{L}$

$G = E / [2(1 + \upsilon)]$ Taking $\upsilon = 0.15$ $G = 0.43$ E $D/b = 500/350 = 1.428$

β the coefficient for rectangular section (Ref.) corresponding to $D/b = 1.428$ is equal to*
$= 0.166 + (0.196 - 0.166) \times 0.228/0.3 = 0.1888$

$\therefore$ 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 $= K_b = 4\ E \times (1/12 \times 350 \times 500^3)/4650 = 3.136\ E \times 10^6$ N.mm

Distribution factor for Beam for torsion $= K_t/(K_t + K_g + K_u) = 0.374/(0.374 + 2.27 + 4.17) = 0.055$

$\therefore$ Distributed moment in beam $= 65.84 \times 0.055 = 3.62$ kN.m.

$\therefore$ 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 $= K_b/(K_b + K_g + K_u) = 3.136/(3.136 + 2.27 + 4.17) = 0.327$

$\therefore$ Distributed moment in beam $= 47.75 \times 0.327 = 15.61$ kN.m.

$\therefore$ Bending moment in beam $= 47.75 - 15.61 = 32.14$ kN.m. ($\cong w_u L^2/16 = 35.81$ kN.m.)

$\therefore$ 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 midspan section for bending moment of 39.48 kN.m. 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 midspan 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 mm^2 for M20 mix.

Vertical shear $= V_u = 26.5 \times 4.65/2 = 61.61$ kN.

* See Timoshenko, S.P. and Goodier, J.N., "Theory of Elasticity", McGraw-Hill publications.

Equivalent shear $= V_{ue} = V_u + 1.6\ T_u/b_w$ (Eq. 4.2.9a)

$= 61.61 + 1.6 \times 65.84/0.35 = 362.6$ kN.

Shear stress $= \tau_{ue} = V_{ue}/(b_w d) = 362.6 \times 1000/ (350 \times 465)$

$= 2.23\ N/mm^2 < 2.8\ N/mm^2$ $\therefore$ 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 $> M_u$

$\therefore$ Section is singly reinforced.

5. *Main Steel* :

(a) Support Section :

Equivalent bending moment $= M_{uel} = M_u + M_t$

where, $M_t = T_u\ (1+ D/b_w)\ /1.7$

$= 65.84 \times (1 + 500/350)\ /1.7 = 94.06$ kN.m.

$\therefore M_{uel} = M_u + M_t$ (Eq. 4.2.–8a)

$= 35.81 + 94.06 = 129.87$ kN.m

$$\text{Required } A_{st} = \frac{0.5 \times 20}{250}\left[1-\sqrt{1-\frac{4.6 \times 129.87 \times 10^6}{20 \times 350 \times 465^2}}\right] 350 \times 465$$

$= 1445\ mm^2$

Provide 2 - ø12 mm + 4 - ø20 mm, Area provided $= 226 + 1256 = 1482\ mm^2 > 1445\ mm^2$

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. 4.2.–8b)

$$A_{sc} = \frac{M_{e2}}{0.87\ f_y\ (d - d_c)} = \frac{58.25 \times 10^6}{0.87 \times 250 \times (465 - 35)} = 623\ mm^2.$$

(b) Mid Span section:

The twisting moment at midspan section is zero. Since partial fixity is assumed at support the midspan 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.

$$\text{Required } A_{st} = \frac{0.5 \times 20}{250}\left[1-\sqrt{1-\frac{4.6 \times 35.81 \times 10^6}{20 \times 350 \times 465^2}}\right] 350 \times 465 = 365\ mm^2$$

Provide 2 – ø16 mm + 1 – ø12mm, Area provided = 628 mm^2

$p_t\% = 100 \times 628/(350 \times 465) = 0.38\%$,

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 $= 0.1 \times bD/100 = 0.1 \times 350 \times 500 / 100 = 175\ mm^2$

Provide 1 bar of ø12 mm on each face at mid depth giving total area = 226 mm^2 at spacing $(500-2 \times 25-2 \times 12)/2 = 213$ mm which is less than 300 mm

Curtailment of Top steel :

At 1.2 m. 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 $1.2^2/2$ = 19.04 kN.m.

Shear force = 61.61 – 26.5 x 1.2 = 29.81 kN

M_{ue} = 31.86 x (1 + 500/350)/1.7 + 19.04 = 64.55 kN.m.

$$\text{Required Ast} = \frac{0.5 \times 20}{250}\left[1-\sqrt{1-\frac{4.6 \times 64.55 \times 10^6}{20 \times 350 \times 465^2}}\right] 350 \times 465 = 673 \text{ mm}^2.$$

Curtail 2 No. 20 mm diameter bars. Balance area consisting of (2 – ø12 + 2– ø20) = 851 mm^2

Extent the bars for a distance of effective depth.

∴ Actual point of curtailment = 1.2 + 0.465 = say 1.7 m from support.

Design for Shear

Maximum equivalent shear, V_{ue} = 362.6 kN.

Area of steel at support = 1596 mm^2

$$p_t\% = \frac{100 \times 1596}{350 \times 465} = 0.98\%, \ \tau_{ue} = 0.616 \text{ Mmm}^2 \text{ for M20 mix}$$

V_{uc} = 0.616 x 350 x 465/1000 = 100.25 kN

$V_{usv\,min}$ = 0.35 x 350 x 465/1000 = 56.96 kN

∴ $V_{ur.min}$ = 100.25 + 56.96 = 157.21 kN < V_{ue} (= 362.6 kN)

∴ Desing shear reinforcement is required.

Shear resisted by stirrups

= V_{us}=V_{ue} – V_{uc} = 362.6 – 100.25 = 262.35 kN.

Now spacing of stirrups

$$s = \frac{0.87 f_y A_{sv} d}{V_{us}}$$

$$\text{Where , } V_{use} = \left(\frac{T_u}{b_1} + \frac{V_u}{2.5}\right) d/d1 \qquad \textbf{(Eq.4.2.9c)}$$

or = V_{ue} – V_{uc} whichever is greater

Now V_{ue} – V_{uc} = 262.35 kN.

b_1 = 350 – 2 (25 + 20/2) = 280 mm , d_1 = 500 – 2 (25 + 20/2) = 430 mm , d = 465 mm

$$\therefore V_{us} = \left(\frac{65.84}{0.28} + \frac{61.61}{2.5}\right) \times \frac{465}{430}$$

= 280.93 kN > 264.6 kN

∴ Vuse = 290.35 kN.

Using 10mm 4-legged stirrups, Area = 4x 78.5 = 314 mm^2

$$s = \frac{0.87 \times 250 \times 314 \times 465}{280.93 \times 1000} = 113\text{mm say 110 mm} \qquad \textbf{(Eq.4.2.10)}$$

Now $s <= (0.75d$ or x_1 or $(x_1 + y_1)/4$ or 300 mm.) whichever is less.

$x_1 = 350 - 2(25 - 10/2) = 310$ mm

$y_1 = 500 - 2 \times 25 + 10 = 460$ mm

$$\frac{x_1 + y_1}{4} = \frac{310 + 460}{4} = 192.0 \text{ mm}$$

$0.75d = 0.75 \times 465 = 348$ mm

least value = 192 mm

$\therefore$ Provide $s = 110$ mm < 192 mm

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.

$\therefore$ for $p_t = 0.38\%$, $\tau_{uc} = 0.43$ N/mm^2 , for M20 mix

$V_{uc} = 0.43 \times 350 \times 465/1000 = 69.98$ kN

$V_{usv.min} = 0.35 \times 350 \times 465/1000 = 56.96$ kN

$V_{ur.min} = 69.98 + 56.96 = 126.94$ kN.

Equating the equivalent shear force = $V_u + 1.6\ T_u/b_w$ at distance x from support to $V_{ur.min}$ we get,

$(61.61 - 26.5\,x) + 1.6\,(65.84 - 28.32\,x)/0.35 = 126.94$

$\therefore 155.96\,x = 241.79$

$\therefore x = 1.51$ m

Using ø6 mm 2-legged stirrups.

Spacing = $250 \times 56/(0.4 \times 350) = 100$ mm

Provide ø6mm 2-legged minimim stirrups at 100mm c/c from 1.51m from each support towards mid-span.

The reinforcement details are shown in Fig. 10.6

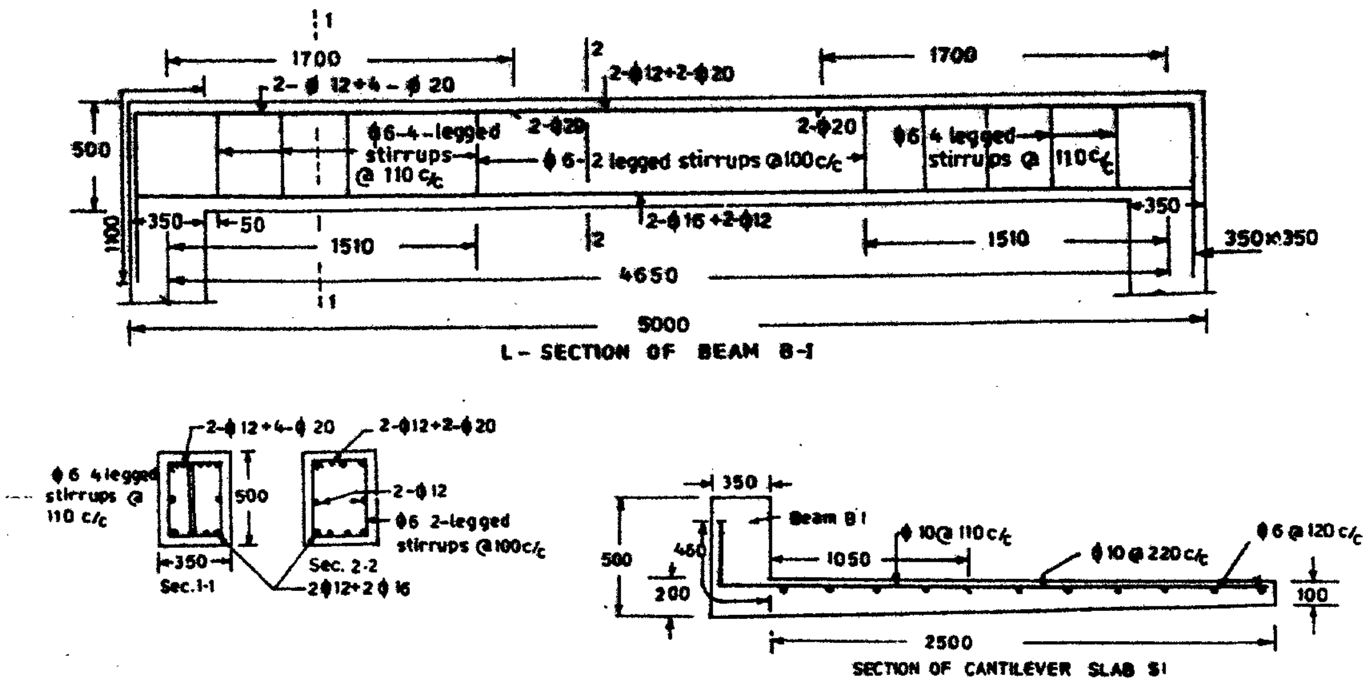


Fig 10.6

Ex. 10.3.8 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 1m 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 beams B3 which are provided at the level of porch slab as shown in Fig. 10.7. The thickness of the wall is 200 mm and size of column is 200 mm x 400 mm. Use concrete M15 and 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.

LL on porch slab= 0.75 kN/m^2 , Load due to finish = 0.8 kN/m^2

Given : f_{ck} = 15 N/mm^2 f_y = 415 N/mm^2

Floor to floor height = 3.4 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

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.

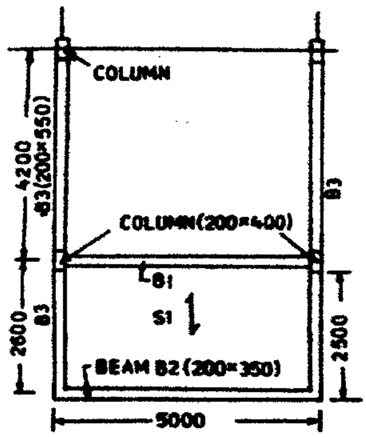


Fig. 10.7

1. *Span* : 2500 + 400/2 − 200/2 = 2600 mm.
2. *Trial Design* : Assuming p_t = 0.2% Modification factor = 1.6

$$\therefore \text{(d) requd} = \frac{2600}{20 \times 1.6} = 81.2 \text{ mm}$$

Provide total depth of 110mm ∴ d= 110 − 20 = 90 mm.

3. *Load* : Consider 1 m width of slab

Self weight of slab = 25 x 0.11 = 2.75 kN/m
Weight due to finish = 0.80
Live Load = 0.75
Total working load = 4.3kN/m
Ultimate load = 1.5 x 4.3 = 6.45 kN/m

4. *Design Moment* :

$M_u = w_u L^2/8 = 6.45 \times 2.6^2/8 = 5.45$ kN.m

5. *Check for depth* from B.M. Considerations :

$M_{ur.max} = 2.07 \times 1000 \times 90^2 \times 10^{-6} = 16.76$ kN.m > 5.45 kN.m

6. *Main Steel* :

$$\text{Ast} = \frac{0.5 \times 15}{415}\left[1-\sqrt{1-\frac{4.6 \times 5.45 \times 10^6}{15 \times 1000 \times 90^2}}\right] 1000 \times 90 = 177.5 \text{ mm}^2$$

Using # 8mm bars
Spacing = 1000 x 50/177.5 = 281mm < = (3 x 90 or 450mm) whichever is less.
∴ Spacing = 270 mm. ∴ Provide # 8mm bar @ 270mm c/c.

7. *Distribution Steel.* :

Using 6 mm diameter bar of grade Fe250
Required area = 0.15 x 1000 x 110/100 = 165 mm^2
Spacing = 1000 x 28/165 = 169mm say 160mm
Provide ø6mm at 160mm c/c

8. *Check for deflection* :

$(p_t)_{requd} = 100 \times 177.5/(1000 \times 90) = 0.197\% < 0.2\%$ assumed.

The actual depth provided is 90mm which is greater than required depth of 84.4mm ∴ Safe

(B) Design of Beam B2

Assume b = 200 mm. D = 350 mm ∴ d = 350 – 35 = 315 mm

1. Span : L = 5000 – 200 = 4800 mm
2. Loads : Slab load/m = 1.5 (25D + LL + FF) x 2.6/2

= 1.5 (25 x 0.11 + 0.75 + 0.8) x 2.6/2 = 8.4 kN/m

Weight of facial wall = (25 x 0.08 x 1) x 1.5 = 3.0 kN/m

Self wt. = 1.5 x 25 x 0.20 x (0.35 – 0.11) =1.80 kN/m

Total ultimate load = w_u=13.20 kN/m

3. *Design moment* :

$M_u = w_u L^2/8 = 13.20 \times 4.8^2/8 = 38$ kN.m.

Since the slab is provided at the bottom of the beam it will act as a rectangular section.

$M_{ur.max} = 2.09 \times 200 \times 315^2 \times 10^{-6} = 41.1$ kN.m $< M_u$ (= 38.0 kN.m.)

$$\text{Required } A_{st} = \frac{0.5 \times 15}{415}\left[1-\sqrt{1-\frac{4.6 \times 38.0 \times 10^6}{15 \times 200 \times 315^2}}\right] 200 \times 315 = 407 \text{ mm}^2$$

Provide 4 – # 12mm bars, **Area provided = 452 mm²**

4. *Design for shear* :

$V_{u.max} = w_u L/2 = 13.20 \times 4.8/2 = 31.68$ kN.

$p_t = 100 \times 452/(200 \times 315) = 0.72\%$ $\tau_{uc} = 0.554$ N/mm² for M15 mix (Table. 4.2)

$V_{uc} = \tau_{uc}\,bd = 0.554 \times 200 \times 315/1000 = 34.9$ kN

$V_{usv.min} = 0.35 \times 200 \times 315/1000 = 22.05$ kN.

$V_{ur.min} = 34.9 + 22.05 = 56.95$ kN. $> V_{u.max}$.

∴ Minimum stirrups are sufficient.

Using ø6mm 2-legged Mild steel stirrups

$$\text{Spacing} = \frac{f_y A_{sv}}{0.4\,b} = \frac{250 \times (2 \times 28)}{0.4 \times 200}$$

= 175mm < = (0.75 x 265 or 450mm)

∴ Provide ø6mm 2-legged stirrups at 175mm c/c.

(C) Design of Beam B3

1. *Type* : Overhanging beam ABC6.9 m long is simply supported at A and B over a span of 4.2 m and cantilever span BC equal to 2.6 m

2. *Span* : AB = 4.2 m and BC = 2.5 – 0.2/2 + 0.4/2 = 2.6 m.

3. *Section* : Assumed section of beam 200 mm x 550 mm ∴ d = 550 – 35 = 515 mm.

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. The beam will act as a rectangular beam.

4. *Loads* :

Span AB : w_{AB} = self weight + floor and wall load of 40 kN/m

= 25 x 0.2 x 0.55 + 40 = 42.75 kN/m

Ultimate load = w_{uAB} = 1.5 x 42.75 = 64.13 kN/m.

Span BC : Self weight + reaction from beam B2 at end C

Ultimate load = 1.5 [(25 x 0.2 x (0.55 – 0.11)] = 3.3 kN/m and concentrated load of 31.68 kN at C

= 3.3 kN/m over length of 2.6 m and 31.68 kN at C

5. *Design Moments* :

$M_{uB} = 3.3 \times 2.6^2/2 + 31.68 \times 2.6 = 93.52$ kN/m

$R_{uA} = 64.13 \times 4.2/2 - 93.52/4.2 = 112.40$kN

$x_{u.max} = 112.4/64.13 = 1.75$ m from A

The point where +ve BM is maximum is at distance 1.75 m from A

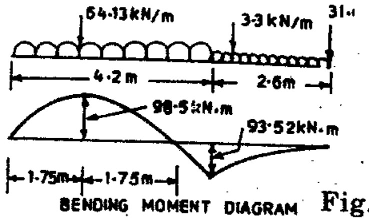


Fig. 10.8

$(M_u)_{max} = 112.4 \times 1.75 - 64.13 \times 1.75^2/2 = 98.5$ kN.m at D

Point of contraflexure, $x_1 = 2\, x_{u.max} = 3.5$ m from A or 0.7 m from B (See Fig 10.8)

6. *Check Depth from B.M. Considerations* :

$M_{ur.max} = 2.07 \times 200 \times 515^2 \times 10^{-6} = 109.8$ kN.m > 98.5 kN.m

∴ The Section is singly reinforced.

7. *Main Steel Span AB* :

(a) Midspan section :

Required A_{st} at D $= \frac{0.5 \times 15}{415}\left[1-\sqrt{1-\frac{4.6 \times 93.52 \times 10^6}{15 \times 200 \times 515^2}}\right] 200 \times 515$

$= 640 \text{mm}^2$

Provide 4 – # 16mm bars. Area provided = 804mm²

Required A_{st} at B $= \frac{0.5 \times 15}{415}\left[1-\sqrt{1-\frac{4.6 \times 93.52 \times 10^6}{15 \times 200 \times 515^2}}\right] 200 \times 515$

$= 600 \text{ mm}^2$

Provide 2– #12mm + 2 – # 16mm at top of support B, Area provided =628 mm²

Curtailment of Bars :

Minimum positive moment reinforcement required to be extended at bottom into the support at

– discontinuous end = $A_{st}/3 = 640/3 = 213$ mm²

– continuous end = $A_{st}/4 = 640/4 = 160$ mm²

Now, the requirement of bars from consideration of development length as per Cl.25.2.3.3 is examined.

At point of contraflexure, the compression reaction does not exist

∴ $L_d < M1/V + L_{ex}$

It two bars of 16mm are to be curtailed the M_{ur} of remaining 2 – #16 mm is given by :

$M_1 = 0.87 \times 415 \times 402\,[515 - 415 \times 402/(15 \times 200)] \times 10^{-6} = 66.7$ kN.m.

SF at point of contraflexure = 112.4 – 64.13 x 3.5 = – 112.06 kN

= 112.06 kN in magnitude

L_{ex} = 12ø or d whichever is greater

= 12x16 or 515 mm whichever is greater

∴ L_{ex} = 515 mm

for M15 and Fe415 $L_d = \frac{0.87 f_y}{4(\tau_{bd} \times 1.6)} \phi = \frac{0.87 \times 415}{4(1 \times 1.6)} = 57\phi$ $= 57 \times 16 = 912$ mm

$\therefore$ 912 < = 66.7 x 10^3/ 112.06 + 515
< = 1110 mm $\therefore$O.K.
$\therefore$ 2 bars of 16 mm can be curtailed. i.e. 50% reinforcement will be curtailed.
$\therefore$ Moment of resistance at point of cut-off = M1 = 66.7kN.m.
Theoretical point of cut-off (TPC) from left support.
Let x_1 be the distance of TPC from A

$R_A x_1 - w_u x_1^2/2 = 66.7$

$112.4\, x_1 - 64.13\, x_1^2/2 = 66.7$

$x_1^2 - 3.5\, x_1 + 2.08 = 0$

$\therefore x_1 = 0.76$ m or $x_2 = 2.74$ m

$\therefore$ Actual point of cut-off $= x_1 - d = 760 - 515 = 245$ mm

$\therefore$ 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 = $x_2 + d$ = 2.74 + 0.515 say 3.25 m from A

Curtail 2– #16 mm bottom bars at distance of 3.25 m from support A 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 (= **516 mm**) (ii) **12ø (=192 mm)** (iii) clear span/16 (= 4000/16)
$\therefore$ Top bars 2 – #16 will be provided for a length of (700+515) say 1.22 m from support B in span AB

Design for Shear

Span AB
(a) *Support A*

V_{AB} = 112.4 kN. A_{st} = 4 – #16 = 804 mm^2

p_t = 100 x 804/(200 x 515) = 0.78%, τ_{uc} = 0.546 N/mm^2 (Table 4.2)

V_{uc} = 0.546 x 200 x 515/1000 = 56.24 kN.

$V_{usv.min}$ = 0.35 x 200 x 0.515 = 36.05 kN.

$\therefore V_{ur.min}$ = 56.24 + 36.05 = 92.29 kN

V_{uD} = 112.4 – 64.13 (200/2 + 515)/1000 = 72.96 kN < $V_{ur.min}$ (= 92.24 kN)

$\therefore$ Minimum shear reinforcement is sufficient.

Using ø6mm 2-legged stirrups
Spacing = 250 x 56/(0.4 x 200) = 175 mm (< =.75 x 515 = 386mm)

(b) *Support B*

V_{BA} = 64.13 x 4.2/2+93.52/4.2=157 kN Area of steel at top = A_{st} = 2– #12 + 2#16 = 628 mm^2

p_t = 100 x628/(200 x 515) = 0.60%, τ_{uc} = 0.5 N/mm^2 for M 15

V_{uc} = 0.5 x 200 x 0.515 = 51.5 kN , $V_{usv.min}$ = 36.05 kN. as obtained in (a)

$V_{ur.min}$ = 51.5 + 36.05 = 87.55 kN, V_{uD} = 157 – 64.13 (200/2 + 515)/1000 = 117.56 kN

Since $V_{uD} > V_{ur.min}$, design shear reinforced is required.

Using ø6mm 2-legged stirrups

$V_{us} = V_{uD} - V_{uc}$ = 117.56 – 51.5 = 66.06 kN

Spacing = 0.87 x 250 x (2 x 28) x 515/(66.06 x 1000)
= 95.0 mm say 90 mm.

L_{s1} = (157 – 87.55)/64.13 = 1.1m

Cantilever Span BC

$V_{BC} = 31.68 + 3.3 \times 2.6 = 40.26$ kN. $< V_{ur.min}$ (= 87.55 kN)

∴ Minimum Stirrups are sufficient.

∴ Provide ø6mm 2-legged stirrups at 90 mm c/c from support B towards A for distance of 1.1m and rest provide ø6mm 2 legged stirrups at 175 mm c/c.

Curtailment of top bars 2 – #16 mm

Moment of resistance of 2 – #12 mm is given by :

$M_{ur} = 0.87 \times 415 \times 226\,[515 - 415 \times 226/(15 \times 200)] \times 10^{-6} = 39.47$ kN.m.

Let x be the distance from cantilever end where Mu = 39.47 kN.m.

∴ $31.68x - 3.3\,x^2/2 = 39.47$ ∴ x = 1.34 m.

Actual point of curtailment of 2 – #16 mm bars = 1.34 – 0.515 = say 0.8 m. from cantilever end or 1.8 m from B.

Check for deflection. :

L/d= 4200 /515 = 8 < 20

or L/d = 2600/515 = 5 < 7   ∴ Safe.

The reinforcement details are shown in Fig. 10.9

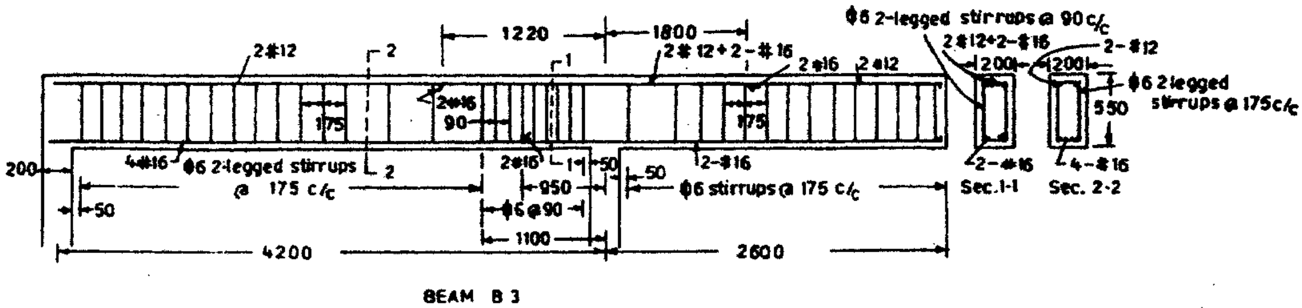


Fig. 10.9

Appendices.

APPENDIX A

Table A-1 Load Data for Residential Building

1. *Values of Load due to Floor/Roof Finish (FF), Live Load (LL) & Total Depth of the Slab (D) assumed for different types of Slabs for Computing its Self Weight*

(a) Roof : FF = 1.5 kN/m² & LL = 1.5 kN/m², D = 120 mm
(b) Floor : FF = 1.0 kN/m² & LL = 2.0 kN/m², D = 120 mm
(c) Bath & W.C: FF = 2.5 kN/m² & LL = 2.0 kN/m², D = 100 mm
(d) Loft : FF = .75 kN/m² & LL = 2.0 kN/m², D = 100 mm
(e) Balconies-
Cantilever: FF = 1.0 kN/m² & LL = 3.0 kN/m², D = 150 mm
Simply supported: FF = 1.0 kN/m² & LL = 3.0 kN/m², D = 100 mm
(f) Stairs : FF = 1.0 kN/m² & LL = 3.0 kN/m², D = 130 mm
Rise = 167 mm & Tread= 250 mm , sec∅= $\sqrt{R2+T2}$/T=1.2

2. *Load On Slab*

(a) Roof :Working load = q = Self+LL+FF = 25x.12+1.5+1.5 = 6 kN/m²
Ultimate Load = q_u = 1.5 x 6 = 9 kN/m²
(b) Floor:Working load = q = Self+LL+FF = 25x.12+1.0+2.0 = 6 kN/m²
Ultimate load = q_u = 1.5 x 6 = 9 kN/m²
(c) Bath & W.C. :
Working load = q = Self+LL+FF = 25x.10+2.0+2.5 = 7 kN/m²
Ultimate load = q_u = 1.5 x 7 = 10.5 kN/m²
(d) Loft :Working load = q = Self+LL+FF = 25x.10+2.0+.75 =5.25kN/m²
Ultimate Load = q_u = 1.5 x 5.25 =say 8 kN/m²
(e) Balcony- (i) Cantilever:
Working load = q = self+LL+FF = 25x.15+3.0+ 1. = say 8 kN/m²
Ultimate load = q_u = 1.5 x 8 = 12 kN/m²
(ii) Simply supported:
Working load = q = self+LL+FF = 25x.10+3.0+1.0 = 6.5 kN/m²
Ultimate load = q_u = 1.5 x 6.5 = say 10 kN/m²
(f) Stair:Working load = q = 25x.13x1.2+25x.167/2+3+1 = say 10 kN/m²
Ultimate load = q_u = 1.5x10 = 15 kN/m²

3. *Loads due to Walls Per Metre Height Per Metre Length :*

(a) Brick masonry: (i) 225 mm or 230 mm thick (9") , 250 mm with plaster.
Working load = q = 20 x .25 = 5 kN/m/m
Ultimate load = q_u = 1.5 x 5 = 7.5 kN/m/m
(ii) 112.5 mm (9/2") thick ,150 mm with plaster.
Working load = q = 20x.150 = 3 kN/m/m
Ultimate load = q_u = 1.5 x 3 = 4.5 kN/m/m
(b) Solid concrete block 150 mm (6") thick , 175 mm with plaster.
Working load = q = 24 x .175 = say 4.5 kN/m/m
Ultimate load = q_u = 1.5 x 4.5 = 6.75 kN/m/m

4. *Load due to R.C.C. Parapet Wall 80 mm Thick :*

Working load = q = 25 x .08 = 2 kN/m/m
Ultimate load = q_u = 1.5 x 2.0 = 3 kN/m/m

5. *Load due to R.C.C. Grill 80 mm Thick With 50% Openings*

Working load = q = 25 x .08/2 = 1.0 kN/m/m
Ultimate load = q_u = 1.5 x 1.0 = 1.5 kN/m/m

6. *Load due to R.C.C. Chajjas 100 mm Thick & 600 mm wide*

Working load = q = 25 x.1 x.6 = 1.5 kN/m
Ultimate load = q_u = 1.5 x 1.5 = 2.25 kN/m

7. *Load due to Beam Rib, Assuming Minimum Depth of Slab Equal to 100 mm*

	Self weight w_{ub} (Ultimate) in kN/m						
Total Depth of beam in mm	300	375	450	525	600	675	750
Width of beam 150 mm	1.1	1.5	2.0	2.4	2.8	3.2	3.6
Width of beam 225 mm	1.7	2.3	3.0	3.6	4.2	4.8	5.4

APPENDIX B

Table B-1 Bending Moment Coefficients for Standard Cases

The bending moment coefficients for some common beams types and loading cases are presented in the following Tables.For detailed information the reader is advised to refer Author's Handbook.

(a) Single Span Beams

Sr.No	Beam Type	Loading	Maximum B.M.	
1	Cantilever :	Point load W at free end.	$-WL$	at fixed end
2	Cantilever :	U.D.Load,w	$-wL^2/2$	at fixed end
3	Cantilever :	Triangluar load,w at fixed end.	$-wL^2/6$	at fixed end
4	Cantilever :	Couple M at free end	M	throughout
5	Simply Supported Beam:	UDL,w	$wL^2/8$	at midspan
6	Simply Supported Beam:	Central Point Load,W	$WL/4$	at midspan
7	Simply Supported Beam:	Triangular load w at centre,	$wL^2/12$	at midspan
8	Simply Supported Beam:	Two equal loads W at distance 'a' from ends.	Wa	between loads
9	Simply Supported Beam:	Two equal and opposite couple M at the ends.	M	throughout
10	Simply Supported Beam:	Non-central load W at distance 'a' from left end and 'b' from right end.	Wab/L	under the load
11	Fixed Beam:	UDL , w	$-wL^2/12$ $+wL^2/24$	at supports at midspan
12	Fixed Beam:	Central Point Load ,W	$-WL/8$ $+WL/8$	at supports at midspan.
13	Fixed Beam:	Triangular Load w at centre	$-5wL^2/96$ $+wL^2/32$	at supports at midspan
14	Fixed Beam:	Two Equal Loads W each at distance 'a' from ends	$Wa(L-a)/L$ $+Wa^2/L$	at supports at midspan
15	Fixed Beam:	Non central Load W at distance 'a' from left	$-Wab^2/L^2$ $-Wa^2b/L^2$	at left end at right end
16	Propped Cantilever:	UDL , w	$-wL^2/8$ $+9wL^2/128$	at fixed end max. in span
17	Propped Cantilever:	Central Point Load W	$-3WL/16$ $+5WL/32$	at fixed end at mid span
18	Propped Cantilever:	Two equal loads W at equal distance 'a' from supports.	$-WL/16$ & $+2WL/a$	at fixed end max. in span
19	Propped Cantilever:	Triangular Load w at fixed end.	$-wL^2/15$ $+3wL^2/100$	at fixed end max. in span
20	Propped Cantilever:	Noncentral point load W at distance 'a' from fixed end.	$-Wa(L-a)(2L-a)/2L^2$	at fixed end.

Table B-1 (continued)

(b) Continuous Beam with Equal Spans Loaded by Uniformly Distributed Load (w) Maximum Bending Moment Coefficients without Redistribution of Moments.

No.of Span	Load Type	1 Midspan	Supp	2 Midspan	Supp	3 Midspan	Supp	4 Midspan	Supp	5 Midspan
2	DL	0.070	-0.125	0.070	-	-	-	-	-	-
	LL	0.096	-0.125	0.096	-	-	-	-	-	-
3	DL	0.080	-0.100	0.025	-0.100	0.080	-	-	-	-
	LL	0.101	-0.117	0.075	-0.117	0.101	—	—	—	—
4	DL	0.077	-0.107	0.036	-0.072	0.036	-0.107	0.077	—	—
	LL	0.099	-0.121	0.081	-0.107	0.081	-0.121	0.099	—	—
5	DL	0.078	-0.105	0.033	-0.080	0.046	-0.080	0.033	-0.105	0.078
	LL	0.100	-0.120	0.080	-0.111	0.086	-0.111	0.080	-0.120	0.100

(Serial Number of Span from Left.)

(c) Continuous Beams : General

I. Maximum Moment Coefficients allowing Redistribution of Moments. (Approximate values).

		Support	Midspan
1. End Spans : Conventional Practice			
	Uniformly Distributed Load	-1/10	1/10
	Point Load	-1/6	1/6
According to ACI-318, and C.P.110 - UDL		-1/ 9	1/11
2 Intermediate Spans: Conventional Practice			
	Uniformly Distributed Load	-1/12	1/12
	Point Load	-1/8	1/7
According to ACI-318 and C.P.110 - UDL		-1/10	1/14

II. Maximum Shear Coefficients for UDL

	End Support	Penultimate Support	Inner Support
According to I.S.Code			
for Dead Load	0.40	0.60 : 0.55	0.50
for Live Load	0.45	0.60 : 0.60	0.60
Conventional Practice			
for Total Load	0.45	0.60 : 0.50	0.50

(d) Continuous Beam with Three or More Equal Spans Allowing Redistribution of Moments (Values As Per Codes) .

	Supports End	Supports Penultimate	Supports Inner	Midspans Outer	Midspans Inner
(i) I.S.Code : Dead Load	0	-1/10	-1/12	1/12	1/24
Live Load	0	-1/9	-1/9	1/10	1/12
(ii) C.P.110 : All loads	0	-1/9	-1/10	1/11	1/14
(iii) A.C.I.Code:-All loads					
End Beam Supports	-1/24	-1/10	-1/11	1/11	1/16
End Column supports	-1/16	-1/10	-1/11	1/14	1/16

(e) Effect of End Moment on Intermediate Spans of a Continuous Beam.

Support Moment Coefficents.

Serial No.of Support next to end support

No.of Spans.	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

Note:- Bending Moment = B.M.CoefxWL Shear Force = S.F.Coefx W

Where, W = Total load on beam and L = Span

Table B-2 Bending Moment Coefficients for Two-way Slabs

(a) Two-way Slab with Corners Restrained :

Case No.	Type of Panel and Moment Location	Short Span Moment Coefficients α_x for Aspect ratio L_y/L_x 1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	Long Span Mom.Coef. : α_y for all values of L_y/L_x
1	Interior Panel									
	- Support	0.032	0.037	0.043	0.047	0.051	0.053	0.060	0.065	0.032
	- Midspan	0.024	0.028	0.032	0.036	0.039	0.041	0.045	0.049	0.024
2	One Short Edge Discontinuous									
	- Support	0.037	0.043	0.048	0.051	0.055	0.057	0.064	0.068	0.037
	- Midspan	0.028	0.032	0.036	0.039	0.041	0.044	0.048	0.052	0.028
3	One Long Edge Discontinuous									
	- Support	0.037	0.044	0.052	0.057	0.063	0.067	0.077	0.085	0.037
	- Midspan	0.028	0.033	0.039	0.044	0.047	0.051	0.059	0.065	0.028
4	Two Adjacent Edges Discontinuous									
	- Support	0.047	0.053	0.060	0.065	0.071	0.075	0.084	0.091	0.047
	- Midspan	0.035	0.040	0.045	0.049	0.053	0.056	0.063	0.069	0.035
5	Two Short Edges Discontinuous									
	- Support	0.045	0.049	0.052	0.056	0.059	0.060	0.065	0.069	-
	- Midspan	0.035	0.037	0.040	0.043	0.044	0.045	0.049	0.052	0.035
6	Two Long Edges Discontinuous									
	- Support	-	-	-	-	-	-	-	-	0.045
	- Midspan	0.035	0.043	0.051	0.057	0.063	0.068	0.080	0.088	0.035
7	One Long and Two Short Edges Discontinuous									
	- Support	0.057	0.064	0.071	0.076	0.080	0.084	0.091	0.097	-
	- Midspan	0.043	0.048	0.053	0.057	0.060	0.064	0.069	0.073	0.043
8	One Short and Two Long Edges Discontinuous									
	- Support	-	-	-	-	-	-	-	-	0.057
	- Midspan	0.043	0.051	0.059	0.065	0.071	0.076	0.087	0.096	0.043
9	Four Edges Discontinuous									
	- Midspan	0.056	0.064	0.072	0.079	0.085	0.089	0.100	0.107	0.056

Notes :

(1) B.M. for short span $m_x = \alpha_x.wL_x^2$ B.M. for long span $m_y = \alpha_y.wL_x^2$ where α_x and α_y are bending moment coefficients given above.

(2) B.M.at support is -ve i.e.hogging. B.M.at mid-span is +ve i.e.sagging.

(3) No redistribution of moments is allowed when above coefficients are used. The main steel for above moments shall be provided only in the middle strip of width = 3/4 of the span. Steel in edge strips of width = span/8 will be only distribution steel running parallel to the supports.

(4) Torsion steel shall be provided at corners over region $L_x/5$ by $L_x/5$ in two orthogonal meshes(two layers for each mesh).Steel in each layer is given as

3/4 th of steel at middle of short span at corner containing two edges over which the slab is discontinuous,

3/8 th of steel at middle of short span at corner containing only one edge over which the slab is discontinuous.

No torsion steel need be provided at corner containing two edges over both of which the slab is continuous.

(5) For moments at supports where two unequal spans meet or in case where the two adjacent spans are loaded unequally, the average of the two values of of support moments on two sides of the support may be taken for design. The moments at the middle of adjacent spans, in that case, will be increased/decreased appropriately.

(6) When L_y/L_x exceeds 2, the slab will be designed as a one-way slab.

Table B-2 Bending Moment Coefficients For Two-way Slab continued ......

(b) Simply Supported Panel with Corners Free to Lift or Torsion Reinforcement not Provided

*	L_y/L_x	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0
*	α_x	.062	.074	.084	.093	.099	.104	.113	.118
*	α_y	.062	.061	.059	.055	.051	.046	.037	.029

* $m_x = \alpha_x wL_x^2$, $m_y = \alpha_y.wL_x^2$

APPENDIX C

Table C-1

Maximum Spans for Given Depth of Slab Satisfying Serviceability Requirements of Deflection (Not requiring deflection calculation).

(a) One-way Simply Supported Slabs of Spans upto 10 m and Two-way Slabs of Shorter Spans More than 3.5m But Less than 10m And Superimposed Loads of Class More than 3000 N/m²

Steel Grade :-	Fe 250						Fe 415					
Steel:	Depth of slab in mm						Depth of slab in mm					
:D=	100	110	120	130	140	150	100	110	120	130	140	150
p% :d=	80	90	100	109	119	129	80	90	100	110	120	130
	Concrete M15/M20						Concrete M15/M20					
0.12	3.20	3.60	4.00	4.36	4.76	5.16	3.12	3.51	3.90	4.29	4.68	5.07
0.15	3.20	3.60	4.00	4.36	4.76	5.16	2.97	3.33	3.70	4.07	4.44	4.81
0.20	3.20	3.60	4.00	4.36	4.76	5.16	2.68	3.01	3.35	3.68	4.02	4.35
0.30	3.20	3.60	4.00	4.36	4.76	5.16	2.28	2.56	2.85	3.13	3.42	3.70
0.40	3.20	3.60	4.00	4.36	4.76	5.16	2.04	2.30	2.56	2.81	3.07	3.32
0.45	3.20	3.60	4.00	4.36	4.76	5.16	1.96	2.21	2.46	2.70	2.95	3.20
0.50	3.04	3.42	3.80	4.14	4.52	4.90	1.88	2.12	2.36	2.59	2.83	3.07
0.60	2.80	3.15	3.50	3.81	4.16	4.51	1.79	2.01	2.24	2.46	2.70	2.91
0.70	2.64	2.97	3.33	3.60	3.93	4.26	1.71	1.92	2.14	2.35	2.57	2.78
0.72	2.61	2.93	3.26	3.55	3.88	4.20	1.68	1.89	2.10	2.31	2.52	2.73
0.75	2.56	2.88	3.20	3.48	3.80	4.12						
0.80	2.48	2.79	3.10	3.38	3.69	4.00	Fe415 but Concrete grade M20 only.					
0.90	2.36	2.65	2.95	3.21	3.51	3.80	1.65	1.85	2.06	2.26	2.47	2.68
0.96	2.30	2.58	2.87	3.13	3.41	3.70	1.60	1.80	2.00	2.20	2.40	2.60
1.00	2.25	2.54	2.82	3.07	3.35	3.64	1.56	1.75	1.95	2.14	2.34	2.53
1.10	2.17	2.45	2.72	2.96	3.23	3.51						
1.20	2.10	2.89	2.63	2.86	3.13	3.39						
1.30	2.03	2.30	2.56	2.79	3.04	3.30						
1.32	2.03	2.30	2.56	2.79	3.04	3.30						
	Fe250 but Concrete M20 only.											
1.40	2.00	2.25	2.50	2.72	2.97	3.22						
1.50	1.96	2.20	2.45	2.67	2.91	3.16						
1.60	1.92	2.16	2.40	2.61	2.85	3.09						
1.70	1.88	2.11	2.35	2.56	2.79	3.03						
1.76	1.85	2.08	2.32	2.52	2.76	3.00						

Note: (1) For continuous slabs multiply above values by 1.3

(2) For cantilevers multiply the above values by 0.35.

(3) d = effective depth.

Two-way slabs with shorter spans < 3.5m & Imposed Loads of class <= 3000 N/m²

Slab Type	Steel Fe 250, Concrete M15/M20 — Total Depth in mm						Steel Fe415, Concrete M15/M20 — Total Depth in mm.					
	100	110	120	130	140	150	100	110	120	130	140	150
Simply Supported.	3.50	3.85	4.20	4.55	4.90	5.25	2.80	3.08	3.36	3.64	3.92	4.20
Fixed or continuous.	4.00	4.40	4.80	5.20	5.60	6.00	3.20	3.52	3.84	4.16	4.48	4.80

Table-C-2
Ultimate Moment of Resistance of *Slabs* (in kN.m) for Different Depths for Given Diameter & Spacing of Bars

(a) Bars in Outer Layer

Di mm	Pitch mm	Area mm²	Concrete M15-Steel Fe250: Depth of slab in mm 100	110	120	130	140	150	Concrete M15-Steel Fe415: Depth of slab in mm 100	110	120	130	140	150
12	75	1508	-	-	-	-	30.79	34.07	-	-	-	-	-	-
	80	1414	-	-	-	26.27	29.35	32.43	-	-	-	-	-	-
	90	1257	-	-	21.34	24.07	26.81	29.54	-	-	-	-	-	-
	100	1131	-	17.26	19.72	22.18	24.64	27.10	-	-	-	-	-	-
	110	1028	13.83	16.07	18.30	20.54	22.78	25.01	-	-	-	-	-	-
	120	942	12.97	15.02	17.07	19.12	21.16	23.21	-	-	-	-	-	-
	125	904	12.57	14.54	16.50	18.47	20.44	22.40	-	-	-	-	-	33.94
	130	870	12.21	14.10	15.99	17.88	19.77	21.67	-	-	-	-	-	32.96
	140	808	11.52	13.27	15.03	16.79	18.55	20.30	-	-	-	-	28.19	31.11
	150	754	10.89	12.53	14.17	15.81	17.45	19.09	-	-	-	23.99	26.72	29.44
	160	707	10.34	11.87	13.41	14.95	16.49	18.02	-	-	20.28	22.83	25.38	27.94
	170	665	9.82	11.27	12.72	14.16	15.61	17.06	-	-	19.35	21.75	24.15	26.56
	175	646	9.59	10.99	12.40	13.80	15.21	16.61	-	-	18.92	21.25	23.59	25.92
	180	628	9.36	10.73	12.09	13.46	14.82	16.19	-	16.24	18.51	20.78	23.04	25.31
	190	595	8.94	10.23	11.53	12.82	14.12	15.41	-	15.58	17.73	19.88	22.03	24.18
	200	565	8.55	9.78	11.01	12.24	13.47	14.70	12.93	14.97	17.01	19.05	21.09	23.13
	225	502	7.71	8.80	9.90	10.99	12.08	13.17	11.80	13.61	15.43	17.24	19.05	20.86
	250	452	7.03	8.01	8.99	9.98	10.96	11.94	10.85	12.48	14.12	15.75	17.38	19.01
	275	411	6.45	7.34	8.24	9.13	10.03	10.92	10.04	11.52	13.00	14.49	15.97	17.46
	300	377	5.96	6.78	7.60	8.42	9.24	10.06	9.33	10.69	12.06	13.42	14.78	16.14
Eff.Depth in mm			79	89	99	109	119	129	79	89	99	109	119	129
10	75	1048	14.25	16.53	18.81	21.09	23.37	25.65	-	-	-	-	-	-
	80	982	13.59	15.73	17.86	20.00	22.13	24.27	-	-	-	-	-	-
	90	873	12.43	14.33	16.23	18.12	20.02	21.92	-	-	-	-	-	33.36
	100	785	11.43	13.13	14.84	16.55	18.25	19.96	-	-	-	25.02	27.86	30.69
	110	714	10.58	12.13	13.68	15.23	16.79	18.34	-	-	20.69	23.26	25.84	28.42
	120	654	9.83	11.25	12.67	14.10	15.52	16.96	-	-	19.34	21.70	24.06	26.42
	125	628	9.59	10.86	12.23	13.60	14.96	16.33	-	16.47	18.73	21.00	23.27	25.54
	130	604	9.19	10.50	11.81	13.13	14.44	15.76	-	15.98	18.16	20.34	22.52	24.71
	140	561	8.62	9.84	11.06	12.28	13.50	14.72	13.06	15.09	17.11	19.14	21.16	23.19
	150	524	8.12	9.26	10.40	11.54	12.68	13.82	12.39	14.28	16.18	18.07	19.96	21.85
	160	491	7.67	8.74	9.81	10.87	11.94	13.01	11.77	13.55	15.32	17.09	18.86	20.64
	170	461	7.25	8.25	9.26	10.26	11.26	12.26	11.19	12.86	14.52	16.19	17.85	19.51
	175	448	7.07	8.04	9.02	10.00	10.97	11.94	10.94	12.55	14.17	15.79	17.41	19.02
	180	436	6.90	7.85	8.79	9.74	10.69	11.64	10.69	12.27	13.84	15.42	16.99	18.57
	190	413	6.57	7.47	8.36	9.26	10.16	11.06	10.23	11.72	13.21	14.70	16.19	17.68
	200	393	6.28	7.13	8.00	8.84	9.70	10.55	9.81	11.23	12.65	14.07	15.48	16.90
	225	349	5.63	6.39	7.15	7.91	8.67	9.43	8.86	10.12	11.38	12.64	13.90	15.16
	250	314	5.11	5.79	6.47	7.16	7.84	8.52	8.08	9.22	10.35	11.49	12.62	13.75
	275	285	4.66	5.28	5.90	6.52	7.14	7.76	7.42	8.45	9.48	10.51	11.54	12.57
	300	262	4.31	4.88	5.45	6.02	6.59	7.16	6.88	7.83	8.77	9.72	10.67	11.61
Eff.Depth in mm			80	90	100	110	120	130	80	90	100	110	120	130

Note :Values above zig-zag line are in-admissible when 30% redistribution is allowed

A-8 Table C-2

Table C-2 Ultimate Moment of Resistance of Slabs (In kN.m) for Different Depths for Given Diameter and Spacing of Bars *Continued*

(a) Bars in Outer Layer

Di mm	Pitch mm	Area mm²	Concrete M15 - Steel Fe 250: Depth of slab in mm: 100	110	120	130	140	150	Concrete M15 - Steel Fe415: Depth of slab in mm: 100	110	120	130	140	150
8	75	670	10.03	11.49	12.95	14.40	15.86	17.32	-	-	19.71	22.13	24.54	26.96
	80	628	9.50	10.86	12.23	13.60	14.96	16.33	-	16.47	18.73	21.00	23.27	25.54
	90	558	8.58	9.79	11.01	12.22	13.44	14.65	13.01	15.02	17.04	19.05	21.07	23.08
	100	503	7.84	8.93	10.02	11.12	12.21	13.31	12.00	13.82	15.63	17.45	19.27	21.08
	110	457	7.19	8.19	9.18	10.18	11.17	12.16	11.11	12.76	14.41	16.06	17.71	19.36
	120	419	6.65	7.57	8.48	9.39	10.30	11.21	10.35	11.86	13.37	14.89	16.40	17.91
	125	402	6.41	7.28	8.16	9.03	9.91	10.78	10.00	11.45	12.90	14.35	15.80	17.25
	130	387	6.19	7.03	7.87	8.72	9.56	10.40	9.68	11.08	12.48	13.87	15.27	16.67
	140	359	5.78	6.56	7.34	8.12	8.90	9.68	9.08	10.38	11.67	12.97	14.27	15.56
	150	335	5.42	6.15	6.88	7.61	8.34	9.07	8.56	9.76	10.97	12.18	13.39	14.60
	160	314	5.11	5.79	6.47	7.16	7.84	8.52	8.08	9.22	10.35	11.49	12.62	13.75
	170	295	4.82	5.46	6.10	6.74	7.38	8.03	7.65	8.72	9.78	10.85	11.91	12.98
	175	287	4.70	5.32	5.94	6.57	7.19	7.82	7.47	8.50	9.54	10.58	11.61	12.65
	180	279	4.57	5.18	5.79	6.39	7.00	7.61	7.28	8.29	9.30	10.30	11.31	12.32
	190	264	4.34	4.92	5.49	6.06	6.64	7.21	6.93	7.88	8.84	9.79	10.74	11.70
	200	251	4.14	4.68	5.23	5.78	6.32	6.87	6.62	7.53	8.43	9.34	10.25	11.15
	225	223	3.70	4.18	4.67	5.16	5.64	6.13	5.94	6.75	7.55	8.36	9.16	9.97
	250	201	3.35	3.79	4.23	4.66	5.10	5.54	5.40	6.13	6.85	7.58	8.30	9.03
	275	183	3.06	3.46	3.86	4.26	4.65	5.05	4.95	5.61	6.27	6.93	7.59	8.25
	300	168	2.82	3.19	3.55	3.92	4.28	4.65	4.57	5.18	5.78	6.39	7.00	7.60
Eff.depth: in mm			80	90	100	110	120	130	80	90	100	110	120	130
6	75	376	6.03	6.85	7.67	8.48	9.30	10.12	9.45	10.81	12.16	13.52	14.88	16.24
	80	353	5.69	6.46	7.23	7.99	8.76	9.53	8.95	10.23	11.50	12.77	14.05	15.32
	90	314	5.11	5.79	6.47	7.16	7.84	8.52	8.08	9.22	10.35	11.49	12.62	13.75
	100	283	4.63	5.25	5.86	6.48	7.10	7.71	7.37	8.40	9.42	10.44	11.46	12.48
	110	257	4.23	4.79	5.35	5.91	6.47	7.03	6.79	7.69	8.62	9.55	10.48	11.40
	120	236	3.90	4.42	4.93	5.44	5.96	6.47	6.26	7.11	7.96	8.82	9.67	10.52
	125	226	3.75	4.24	4.73	5.22	5.71	6.20	6.02	6.83	7.65	8.47	9.28	10.10
	130	217	3.61	4.08	4.55	5.02	5.49	5.96	5.80	6.58	7.36	8.15	8.93	9.71
	140	202	3.37	3.81	4.25	4.68	5.12	5.56	5.43	6.16	6.89	7.61	8.34	9.07
	150	188	3.14	3.55	3.96	4.37	4.78	5.19	5.08	5.76	6.43	7.11	7.79	8.47
	160	177	2.97	3.35	3.74	4.12	4.51	4.89	4.80	5.44	6.08	6.72	7.36	8.00
	170	166	2.79	3.15	3.51	3.87	4.23	4.59	4.52	5.12	5.72	6.32	6.92	7.52
	175	161	2.71	3.06	3.41	3.76	4.11	4.46	4.39	4.97	5.55	6.14	6.72	7.30
	180	157	2.64	2.98	3.33	3.67	4.01	4.35	4.29	4.86	5.42	5.99	6.56	7.12
	190	149	2.51	2.84	3.16	3.48	3.81	4.13	4.08	4.62	5.16	5.70	6.23	6.77
	200	141	2.38	2.69	2.99	3.30	3.61	3.91	3.87	4.38	4.89	5.40	5.91	6.42
	225	126	2.13	2.41	2.68	2.96	3.23	3.51	3.48	3.94	4.39	4.85	5.30	5.76
	250	113	1.92	2.17	2.41	2.66	2.90	3.15	3.14	3.54	3.95	4.36	4.77	5.18
	275	103	1.75	1.98	2.20	2.43	2.65	2.87	2.87	3.24	3.61	3.98	4.36	4.73
Eff.Depth: in mm.			80	90	100	110	120	130	80	90	100	110	120	130

Note :-Values above zig-zag line are in-admissible when 30% redistribution is allowed and Values below zig-zag line are for steel area < minimum required area therefore to be considered only for extra steel at support.

Table C-2 Ultimate Moment of Resistance of Slabs (In kNm) For Different Depths For Given Diameter & Spacing of Bars (Contd.)

(b) Bars in Inner Layer (With 12 mm Bars in Outer Layer)*

Dia mm	Pitch mm	Area mm²	Concrete M15 – Steel Fe 250: Depth of slab in mm 100	110	120	130	140	150	Concrete M15 – Steel Fe415: Depth of slab in mm 100	110	120	130	140	150
12	75	1508	–	–	–	–	–	30.13	–	–	–	–	–	–
	80	1414	–	Not	–	–	–	28.73	–	–	–	–	–	–
	90	1257	–	Admissible		20.80	23.53	26.26	–	–	–	–	–	–
	100	1131	–	–	16.76	19.22	21.68	24.14	–	–	–	–	–	–
	110	1028	–	–	15.62	17.86	20.10	22.33	–	–	–	–	–	–
	120	942	–	12.56	14.61	16.66	18.71	20.75	– Not admissible			–	–	–
	125	904	–	12.18	14.14	16.11	18.08	20.04	–	–	–	–	–	–
	130	870	9.93	11.83	13.72	15.61	17.50	19.40	–	–	–	–	–	–
	140	808	9.41	11.17	12.92	14.68	16.44	18.20	–	–	–	–	–	–
	150	754	8.93	10.57	12.21	13.85	15.50	17.13	–	–	–	–	–	27.61
	160	707	8.49	10.03	11.57	13.10	14.64	16.18	–	–	–	–	23.45	26.17
	170	665	8.09	9.53	10.98	12.43	13.87	15.32	–	–	–	–	22.32	24.87
	175	646	7.90	9.31	10.71	12.12	13.52	14.93	–	–	–	18.87	21.27	23.67
	180	628	7.72	9.09	10.45	11.82	13.20	14.55	–	–	–	18.46	20.80	23.12
	190	595	7.39	8.68	9.98	11.27	12.56	13.86	–	–	15.15	17.30	19.45	21.60
	200	565	7.08	8.31	9.53	10.76	12.00	13.22	–	–	14.56	16.60	18.64	20.68
	225	502	6.40	7.50	8.60	9.68	10.77	11.86	–	11.44	13.25	15.06	16.88	18.70
	250	452	5.85	6.83	7.81	8.80	9.78	10.76	8.90	10.53	12.16	13.80	15.42	17.05
	275	411	5.38	6.27	7.16	8.06	8.95	9.85	8.25	9.74	11.22	12.71	14.20	15.67
	300	377	4.98	5.80	6.62	7.44	8.26	9.08	7.70	9.06	10.42	11.78	13.14	14.51
Eff.depth mm.			67	77	87	97	107	117	67	77	87	97	107	117
10	75	1048	–	–	16.08	18.36	20.64	22.92	–	–	–	–	–	–
	80	982	–	13.16	15.30	17.44	19.57	21.71	–	–	–	–	–	–
	90	873	10.15	12.05	13.95	15.85	17.74	19.64	– Not admissible			–	–	–
	100	785	9.38	11.08	12.80	14.50	16.21	17.91	–	–	–	–	–	27.30
	110	714	8.71	10.26	11.82	13.37	14.92	16.48	–	–	–	–	22.75	25.33
	120	654	8.12	9.54	10.97	12.40	13.81	15.23	–	–	–	18.87	21.23	23.60
	125	628	7.86	9.22	10.60	11.96	13.32	14.70	–	–	16.01	18.28	20.55	22.82
	130	604	7.61	8.92	10.24	11.55	12.87	14.18	–	–	15.55	17.73	19.91	22.09
	140	561	7.16	8.38	9.60	10.82	12.04	13.26	–	12.66	14.68	16.71	18.73	20.76
	150	524	6.75	7.90	9.03	10.17	11.31	12.45	–	12.01	13.91	15.80	17.70	19.58
	160	491	6.40	7.46	8.52	9.60	10.66	11.73	–	11.42	13.20	14.96	16.74	18.51
	170	461	6.05	7.05	8.05	9.06	10.06	11.06	9.20	10.86	12.52	14.20	15.85	17.52
	180	436	5.76	6.71	7.66	8.60	9.55	10.50	8.81	10.38	11.95	13.53	15.10	16.68
	190	413	5.50	6.40	7.30	8.18	9.08	9.98	8.44	9.93	11.42	12.91	14.40	15.90
	200	393	5.25	6.11	6.96	7.82	8.67	9.53	8.11	9.52	10.94	12.36	13.78	15.20
	225	349	4.72	5.48	6.24	7.00	7.76	8.52	7.35	8.61	9.87	11.13	12.40	13.65
	250	314	4.30	4.97	5.65	6.34	7.02	7.70	6.72	7.86	9.00	10.13	11.26	12.40
	275	285	3.92	4.54	5.16	5.78	6.40	7.02	6.20	7.21	8.24	9.27	10.30	11.33
	300	262	3.63	4.20	4.77	5.34	5.91	6.48	5.75	6.70	7.64	8.58	9.53	10.48
Eff.depth mm			68	78	88	98	108	118	68	78	88	98	108	118

Note : –(1) Values above zig-zag dividing line are not admissible when maximum (30%) redistribution of moments is allowed.

(2) For bar diameter 10mm or less in outer layer, values in Table. (a) for outer layer may be used referring to corresponding effective depth (10mm less).

Table C-2 Ultimate Moment of Resistance of Slabs (In kN.m) for Different Depths for Given Diameter & Spacing of Bars (Contd....)

(b) Bars in Inner Layer (With 12 mm Bars in Outer Layer)

Dia mm	Pitch mm	Area mm²	Concrete M15 – Steel Fe 250: Depth of slab in mm 100	110	120	130	140	150	Concrete M15 – Steel Fe415: Depth of slab in mm 100	110	120	130	140	150
8	75	670	8.43	9.90	11.34	12.80	14.26	15.71	-	-	-	19.46	21.88	24.30
	80	628	8.00	9.36	10.73	12.10	13.46	14.82	-	-	16.24	18.51	20.78	23.04
	90	558	7.25	8.46	9.67	10.90	12.10	13.31	-	12.81	14.82	16.83	18.85	20.86
	100	503	6.63	7.73	8.82	9.91	11.01	12.10	-	11.82	13.64	15.45	17.27	19.08
	110	457	6.10	7.10	8.10	9.08	10.08	11.07	9.30	10.95	12.60	14.25	15.90	17.55
	120	419	5.65	6.56	7.47	8.40	9.30	10.21	8.68	10.20	11.71	13.22	14.74	16.25
	125	402	5.45	6.32	7.20	8.07	8.94	9.82	8.40	9.85	11.30	12.75	14.21	15.66
	130	387	5.26	6.11	6.95	7.80	8.63	9.47	8.15	9.54	10.94	12.34	13.73	15.13
	140	359	4.92	5.70	6.48	7.26	8.04	8.82	7.66	8.95	10.25	11.54	12.84	14.14
	150	335	4.62	5.35	6.08	6.81	7.54	8.26	7.22	8.43	9.64	10.85	12.06	13.27
	160	314	4.35	5.04	5.72	6.40	7.09	7.77	6.84	7.97	9.11	10.24	11.37	12.51
	170	295	4.11	4.75	5.39	6.04	6.68	7.32	6.48	7.54	8.61	9.68	10.74	11.81
	175	287	4.01	4.63	5.26	5.88	6.51	7.13	6.33	7.36	8.40	9.44	10.47	11.51
	180	279	3.90	4.51	5.12	5.73	6.33	6.94	6.17	7.18	8.19	9.20	10.20	11.21
	190	264	3.71	4.28	4.86	5.43	6.01	6.58	5.88	6.83	7.79	8.74	9.69	10.65
	200	251	3.54	4.08	4.63	5.18	5.72	6.27	5.62	6.53	7.44	8.34	9.25	10.15
	225	223	3.17	3.65	4.14	4.62	5.11	5.60	5.06	5.86	6.67	7.47	8.28	9.08
	250	201	2.87	3.31	3.74	4.18	4.62	5.06	4.60	5.33	6.06	6.78	7.51	8.23
	275	183	2.62	3.02	3.42	3.82	4.22	4.62	4.22	4.90	5.55	6.21	6.87	7.53
	300	168	2.42	2.78	3.15	3.52	3.88	4.25	3.90	4.51	5.12	5.72	6.33	6.94
Eff.depth mm:			69	79	89	99	109	119	69	79	89	99	109	119
6	75	376	5.21	6.03	6.85	7.67	8.48	9.30	8.09	9.45	10.81	12.16	13.52	14.88
	80	353	4.92	5.70	6.46	7.23	8.00	8.76	7.68	8.95	10.23	11.50	12.77	14.05
	90	314	4.42	5.11	5.80	6.47	7.16	7.84	6.95	8.08	9.22	10.35	11.50	12.62
	100	283	4.02	4.63	5.25	5.86	6.48	7.10	6.35	7.37	8.40	9.42	10.44	11.46
	110	257	3.67	4.23	4.80	5.35	5.91	6.47	5.84	6.76	7.70	8.62	9.55	10.48
	120	236	3.39	3.90	4.42	4.93	5.44	5.96	5.41	6.26	7.11	7.96	8.82	9.67
	125	226	3.26	3.75	4.24	4.73	5.22	5.71	5.20	6.02	6.83	7.65	8.47	9.28
	130	217	3.13	3.61	4.08	4.55	5.02	5.50	5.01	5.80	6.58	7.36	8.15	8.93
	140	202	2.93	3.37	3.81	4.25	4.68	5.12	4.70	5.43	6.16	6.89	7.61	8.34
	150	188	2.73	3.14	3.55	3.96	4.37	4.78	4.40	5.08	5.76	6.43	7.11	7.80
	160	177	2.58	2.97	3.35	3.74	4.12	4.51	4.16	4.80	5.44	6.08	6.72	7.36
	170	166	2.43	2.80	3.15	3.51	3.87	4.23	3.92	4.52	5.12	5.72	6.32	6.92
	175	161	2.36	2.71	3.06	3.41	3.76	4.11	3.81	4.40	4.97	5.55	6.14	6.72
	180	157	2.30	2.64	2.98	3.33	3.67	4.01	3.72	4.30	4.86	5.42	6.00	6.56
	190	149	2.20	2.51	2.84	3.16	3.48	3.81	3.54	4.08	4.62	5.16	5.70	6.23
	200	141	2.07	2.38	2.70	3.00	3.30	3.61	3.36	3.87	4.38	4.90	5.40	5.91
	225	126	1.86	2.13	2.41	2.68	2.96	3.23	3.03	3.48	3.94	4.40	4.85	5.30
	250	113	1.67	1.92	2.17	2.46	2.66	2.90	2.73	3.14	3.54	3.95	4.36	4.77
Eff.depth mm:			70	80	90	100	110	120	70	80	90	100	110	120

Note:-(1) Values above upper zig-zag dividing line are not admissible when maximum (30%) redistribution of moments is allowed.

(2) Values below lower zig-zag dividing line are for steel area < minimum required area and hence to be considered only for extra support steel.

* (3) For bar diameter 10 mm or less in outer layer values in Table (a) for outer layer bars may be used referring to corresponding effective depth(10mm less)

Table C-3 Distribution Steel (per meter width) for Different Depths of Slabs and Grades of Steel

(a) Grade – Fe250

Depth mm	Area sq.mm	Diam-Pitch mm mm	Diam-Pitch mm mm	Diam-Pitch mm mm	Diam-Pitch mm mm
90	135	6 – 200	–	–	–
100	150	6 – 180	–	–	–
110	165	6 – 160	8 – 300	–	–
120	180	6 – 150	8 – 275	10 – 400	–
130	195	6 – 140	8 – 250	10 – 400	–
140	210	6 – 130	8 – 225	10 – 350	–
150	225	6 – 125	8 – 220	10 – 325	–
160	240	6 – 110	8 – 200	10 – 300	12 – 450
180	270	6 – 100	8 – 180	10 – 280	12 – 400
200	300	–	8 – 160	10 – 260	12 – 375

(b) Grades – Fe415 And Fe500

Depth mm	Area sq.mm	Diam-Pitch mm mm	Diam-Pitch mm mm	Diam-Pitch mm mm	Diam-Pitch mm mm
90	108	6 – 250	–	–	–
100	120	6 – 225	–	–	–
110	132	6 – 200	8 – 375	–	–
120	144	6 – 180	8 – 350	–	–
130	156	6 – 180	8 – 300	–	–
140	168	6 – 160	8 – 300	10 – 450	–
150	180	6 – 150	8 – 275	10 – 400	–
160	192	6 – 140	8 – 250	10 – 400	–
180	216	6 – 130	8 – 225	10 – 350	–
200	240	6 – 110	8 – 200	10 – 300	12 – 450

Note :- (1) Area of distribution steel = 0.15% of gross cross-section for Fe250
= 0.12% of gross cross-section for Fe415 & Fe500
i.e. = 1.5 D for Fe250
= 1.2 D for Fe415 & Fe500
where, D is in mm.

(2) Spacing not to exceed 450 mm or 5 times the effective depth whichever is less.

(3) Depth (D) in the above table refers to total depth of the slab.

Table C-4 Design of One-way Slabs (Residential Buildings)

Concrete M 15, Main Steel Fe 415, Distribution Steel Fe 250
Live load 2 kN/m² , Floor Finish 1 kN/m²

Span m	***Type – A*** (EC = 1) Simply Supported MCOEF = 1/8 — Depth of Slab in mm. 100	110	120	130	140	150	:	***Type – B*** (EC = 2) Continuous at one end, MCOEF = 1/10 — Depth of Slab in mm. 100	110	120	130	140	150
2.50	(8-200)	8-225	8-225	8-250	8-275	8-275	:	8-240	8-270	8-300	8-300	8-300	8-275
2.60	(8-180)	8-200	8-220	8-225	8-250	8-250	:	8-240	8-250	8-275	8-300	8-300	8-275
2.70	(8-175)	8-190	8-200	8-220	8-225	8-240	:	8-220	8-240	8-250	8-275	8-275	8-275
2.80	* –	(8-175)	8-190	8-200	8-210	8-225	:	8-200	8-225	8-240	8-250	8-270	8-275
2.90	–	(8-160	8-175)	8-180	8-200	8-210	:	–	8-200	8-225	8-240	8-250	8-250
3.00	–	(8-150	8-160)	8-175	8-180	8-190	:	–	8-190	8-200	8-220	8-225	8-240
3.10	–	* –	(8-150)	8-160	8-175	8-180	:	–	8-180	8-190	8-200	8-220	8-225
3.20	–	–	(8-140	8-150)	8-160	8-170	:	–	–	8-180	8-190	8-200	8-220
3.30	–	–	(8-130	8-140)	8-150	8-160	:	–	–	8-160	8-180	8-190	8-200
3.40	–	–	* –	(8-130	8-140)	8-150	:	–	–	8-150	8-170	8-180	8-190
3.50	–	–	–	(8-120	8-130)	8-140	:	–	–	8-140	8-160	8-170	8-175
3.60	–	–	–	(8-110	8-120)	8-130	:	–	–	–	8-150	8-160	8-160
3.70	–	–	–	* –	(8-110	8-120)	:	–	–	–	8-140	8-150	8-150
3.80	–	–	–	–	(8-110	8-110)	:	–	–	–	8-130	8-140	8-150
3.90	–	–	–	–	(8-100	8-110)	:	–	–	–	–	8-130	8-140
4.00	–	–	–	–	* –	(8-100)	:	–	–	–	–	8-120	8-130

Notes :– * Values in the parenthesis are applicable only for CONTINUOUS SLABS which are designed for B.M. coefficient = 1/8

Type – C (EC = 3)
Continuous/Fixed at both Ends .... MCOEF = 1/12

Span m.	Depth of Slab in mm 100	110	120	130	140	150	:	Span metres	Depth of Slab in mm 110	120	130	140	150
2.80	8-240	8-275	8-300	8-300	8-300	8-275	:	3.80	–	–	8-160	8-170	8-180
2.90	8-225	8-250	8-275	8-275	8-275	8-275	:	3.90	–	–	8-150	8-160	8-170
3.00	–	8-225	8-250	8-270	8-275	8-275	:	4.00	–	–	–	8-150	8-160
3.10	–	8-210	8-225	8-250	8-250	8-275	:	4.10	–	–	–	8-140	8-150
3.20	–	8-200	8-220	8-225	8-250	8-250	:	4.20	–	–	–	8-130	8-140
3.30	–	8-190	8-200	8-220	8-225	8-240	:	4.30	–	–	–	8-130	8-130
3.40	–	–	8-190	8-200	8-210	8-225	:	4.40	–	–	–	–	8-130
3.50	–	–	8-180	8-190	8-200	8-210	:	4.50	–	–	–	–	8-120
3.60	–	–	8-170	8-180	8-190	8-200	:	4.60	–	–	–	–	8-120
3.70	–	–	–	8-170	8-180	8-190	:	4.70	–	–	–	–	–

Notes :– B. M. = MCOEF x $w_u L^2$

APPENDIX D

Table D-1 Under-reinforced Design – Rectangular Beam – 230 mm wide Ultimate Moment of Resistance (M_{ur}) in kN.m for Different Depths for Given Number – Diameter Combination of Bars in Ascending Order of Area of Steel

No Dia mm	Ast mm²	Cover: mm	Total Depth of the Beam in mm. 300	380	400	450	500	530	600	680	750
			(a) Concrete – M15 Steel – Fe 415								
2-12	226	31	19.7	26.3	27.9	32.0	36.1	–	–	–	–
3-12	339	31	27.9	37.7	40.2	46.3	52.4	56.1	64.7	74.4	83.0
2-16	402	33	31.7	43.3	46.2	53.5	60.8	65.1	75.3	86.9	97.0
4-12	452	31	–	48.1	51.3	59.5	67.7	72.6	84.0	97.0	108.5
5-12	565	31	–	57.3	61.4	71.6	81.8	87.9	102.2	118.5	132.8
3-16	603	33	–	–	64.1	75.0	85.9	92.4	107.7	125.1	140.3
2-20	628	35	–	–	–	77.0	88.3	95.1	111.0	129.1	145.0
* 6-12	678	45	–	–	–	–	91.5	98.9	116.0	135.6	152.7
* 7-12	791	45	–	–	–	–	–	111.5	131.5	154.3	174.3
4-16	804	33	–	–	–	–	–	116.2	136.5	159.7	180.1
* 8-12	904	45	–	–	–	–	–	–	145.8	171.9	194.8
3-20	942	35	–	–	–	–	–	–	–	180.8	204.6
2-25	982	38	–	–	–	–	–	–	–	185.9	210.7
5-16	1005	33	–	–	–	–	–	–	–	190.9	216.3
* 9-12	1017	45	–	–	–	–	–	–	–	188.4	214.1
			(b) Concrete – M20. Steel Fe 415								
2-12	226	31	20.3	26.8	28.4	32.5	36.6	–	–	–	–
3-12	339	31	29.2	39.0	41.4	47.5	53.7	57.2	65.9	75.7	84.3
2-16	402	33	33.5	45.1	48.0	55.3	62.5	66.9	77.0	88.6	98.8
4-12	452	31	37.2	50.3	53.6	61.7	69.9	74.8	86.2	99.3	110.7
5-12	565	31	44.5	60.8	64.9	75.1	85.3	91.4	105.7	122.0	136.3
3-16	603	33	–	63.7	68.1	78.9	89.8	96.4	111.6	129.0	144.3
2-20	628	35	–	65.4	69.9	81.3	92.6	99.4	115.3	133.4	149.3
* 6-12	678	45	–	67.2	72.1	84.3	96.5	103.9	121.0	140.6	157.7
* 7-12	791	45	–	–	–	95.4	109.7	118.3	138.3	161.1	181.1
4-16	804	33	–	–	85.5	100.0	114.5	123.2	143.5	166.8	187.1
* 8-12	904	45	–	–	–	–	122.1	131.8	154.7	180.8	203.6
3-20	942	35	–	–	–	–	129.2	139.4	163.3	190.5	214.3
2-25	982	38	–	–	–	–	132.6	143.2	168.0	196.4	221.2
5-16	1005	33	–	–	–	–	136.5	147.4	172.8	201.9	227.3
* 9-12	1017	45	–	–	–	–	–	@144.6	170.3	199.7	225.4
* 10-12	1131	45	–	–	–	–	–	–	185.0	217.7	246.2
* 6-16	1206	49	–	–	–	–	–	–	192.5	227.4	258.0
** 11-12	1243	58	–	–	–	–	–	–	–	228.8	260.2
4-20	1256	35	–	–	–	–	–	–	–	241.1	272.9
** 12-12	1357	58	–	–	–	–	–	–	–	244.6	278.9
* 7-16	1407	49	–	–	–	–	–	–	–	–	291.6
3-25	1473	38	–	–	–	–	–	–	–	–	308.3

Notes : (1) Dashes indicate that particular Number-Diameter Combination is not admissible from the condition of minimum or maximum steel.
(2) * indicates 2 layers.
(3) ** indicates 3 layers.
(4) Values marked @ are lower than preceeding one due to reduction in eff. depth

Table D-2
Ultimate Moment of Resistance M_{ur} of Singly Reinforced and Doubly Reinforced Rectangular Beams – 150 mm Wide

Grade of Concrete M15 and Grade of Steel Fe415

b mm	D mm	N1-D1+N2-D2 mm mm	NC-DIAMC 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
150	300	2- 8+0- 0	A	101	101	–	1	* 9.16	14.15	14.21	14.21	28.36	28.36
150	300	2-10+0- 0	N	157	157	–	1	*13.66	14.09	17.01	17.01	31.10	31.10
150	300	2-10+1- 8	C	207	157	–	1	*17.33	14.09	18.95	17.01	33.05	31.10
150	300	2-12+0- 0	H	226	226	–	1	*18.54	14.04	19.54	19.54	33.58	33.58
150	300	2-10+1-12	O	270	157	–	1	*21.35	14.04	20.88	16.97	34.92	31.01
150	300	2-12+1- 8	R BARS	276	226	–	1	*21.73	14.04	21.05	19.54	35.10	33.58
150	300	2-12+1-10	2- 8+0- 0	305	226	101	1	25.63	14.04	21.81	19.54	35.85	33.58
150	300	2-12+1-10	2- 8+1-10	305	226	179	1	26.45	14.04	21.81	19.54	35.85	33.58
150	300	2-12+1-10	2-10+0- 0	305	226	157	1	26.27	14.04	21.81	19.54	35.85	33.58
150	300	2-12+1-12	2- 8+0- 0	339	226	101	1	27.95	14.04	22.66	19.54	36.70	33.58
150	300	2-12+1-12	2-10+0- 0	339	226	157	1	28.84	14.04	22.66	19.54	36.70	33.58
150	300	2-12+1-12	2-10+1- 8	339	226	207	1	29.35	14.04	22.66	19.54	36.70	33.58
150	300	2-12+1-12	2-10+1-12	339	226	270	1	29.48	14.04	22.66	19.54	36.70	33.58
150	300	2-12+1-12	2- 8+1-12	339	226	214	1	29.33	14.04	22.66	19.54	36.70	33.58
150	300	2-12+1-12	2-12+0- 0	339	226	226	1	29.32	14.04	22.66	19.54	36.70	33.58
150	300	2-12+2-10	2- 8+0- 0	383	226	101	2	28.82	13.34	22.85	18.92	36.19	32.26
150	300	2-12+2-10	2- 8+1-10	383	226	179	2	30.40	13.34	22.85	18.92	36.19	32.26
150	300	2-12+2-10	2-10+0- 0	383	226	157	2	30.03	13.34	22.85	18.92	36.19	32.26
150	300	2-12+2-10	2-10+1- 8	383	226	207	2	30.80	13.34	22.85	18.92	36.19	32.26
150	300	2-12+2-10	2-10+1-12	383	226	270	2	31.28	13.34	22.85	18.92	36.19	32.26
150	300	2-12+2-10	2-12+0- 0	383	226	226	2	30.93	13.34	22.85	18.92	36.19	32.26
150	300	2-12+2-10	2-12+1- 8	383	226	276	2	31.19	13.34	22.85	18.92	36.19	32.26
150	300	2-12+2-10	2-12+1-10	383	226	305	2	31.48	13.34	22.85	18.92	36.19	32.26
150	300	2-16+0- 0	2- 8+0- 0	402	402	101	1	31.49	13.94	23.91	23.91	37.85	37.85
150	300	2-16+0- 0	2-10+0- 0	402	402	157	1	32.83	13.94	23.91	23.91	37.85	37.85
150	300	2-16+0- 0	2-10+1- 8	402	402	207	1	33.72	13.94	23.91	23.91	37.85	37.85
150	300	2-16+0- 0	2-10+1-12	402	402	270	1	34.33	13.94	23.91	23.91	37.85	37.85
150	300	2-16+0- 0	2-12+0- 0	402	402	226	1	33.90	13.94	23.91	23.91	37.85	37.85
150	300	2-16+0- 0	2-12+1- 8	402	402	276	1	34.37	13.94	23.91	23.91	37.85	37.85
150	300	2-16+0- 0	2-12+1-10	402	402	305	1	34.43	13.94	23.91	23.91	37.85	37.85
150	300	2-16+0- 0	2-12+1-12	402	402	339	1	34.85	13.94	23.91	23.91	37.85	37.85
150	300	2-12+2-12	2- 8+0- 0	452	226	101	2	32.24	13.34	24.15	18.92	37.48	32.26
150	300	2-12+2-12	2-10+0- 0	452	226	157	2	33.98	13.34	24.15	18.92	37.48	32.26
150	300	2-12+2-12	2-10+1-12	452	226	270	2	36.17	13.34	24.15	18.92	37.48	32.26
150	300	2-12+2-12	2- 8+1-12	452	226	214	2	35.24	13.34	24.15	18.92	37.48	32.26
150	300	2-12+2-12	2-12+0- 0	452	226	226	2	35.48	13.34	24.15	18.92	37.48	32.26
150	300	2-12+2-12	2-12+1-10	452	226	305	2	36.55	13.34	24.15	18.92	37.48	32.26
150	300	2-12+2-12	2-12+1-12	452	226	339	2	36.85	13.34	24.15	18.92	37.48	32.26

Note :- Values shown by * are for Singly reinforced sections.

Table D-2 M_{ur} of Singly and Doubly Reinforced Beam 150 mm Wide for M15 & Fe415

continued....

b mm	D mm	N1-D1+N2-D2 mm	NC-DIAMC 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
150	300	3-12+1-12	2- 8+0- 0	452	339	101	2	32.24	13.34	24.15	21.91	37.48	35.25
150	300	3-12+1-12	2-10+0- 0	452	339	157	2	33.98	13.34	24.15	21.91	37.48	35.25
150	300	3-12+1-12	2-10+1-12	452	339	270	2	36.17	13.34	24.15	21.91	37.48	35.25
150	300	3-12+1-12	2-12+0- 0	452	339	226	2	35.48	13.34	24.15	21.91	37.48	35.25
150	300	3-12+1-12	2-12+1-12	452	339	339	2	36.85	13.34	24.15	21.91	37.48	35.25
150	300	2-16+1-12	2-10+0- 0	515	402	157	1	39.13	13.94	25.95	23.91	39.88	37.85
150	300	2-16+1-12	2-10+1-12	515	402	270	1	42.26	13.94	25.95	23.91	39.88	37.85
150	300	2-16+1-12	2- 8+1-12	515	402	214	1	40.89	13.94	25.95	23.91	39.88	37.85
150	300	2-16+1-12	2-12+0- 0	515	402	226	1	41.23	13.94	25.95	23.91	39.88	37.85
150	300	2-16+1-12	2-12+1-12	515	402	339	1	43.41	13.94	25.95	23.91	39.88	37.85
150	375	2-8+0- 0	AN	101	101	-	1	*11.88	18.06	16.37	16.37	34.43	34.43
150	375	2-10+0-0	CH	157	157	-	1	*17.91	18.01	19.69	19.69	37.70	37.70
150	375	2-12+0-0	OR	226	226	-	1	*24.67	17.96	22.74	22.74	40.70	40.70
150	375	2-12+1-10		305	226	-	1	*31.63	17.96	25.49	22.74	43.44	40.70
150	375	2-12+1-12	BARS.	339	226	-	1	*34.43	17.96	26.52	22.74	44.48	40.70
150	375	2-12+2-10	2- 8+0- 0	383	226	101	2	39.19	17.25	27.02	22.19	44.27	39.44
150	375	2-12+2-10	2-10+0- 0	383	226	157	2	40.40	17.25	27.02	22.19	44.27	39.44
150	375	2-12+2-10	2-10+1- 8	383	226	207	2	41.18	17.25	27.02	22.19	44.27	39.44
150	375	2-12+2-10	2-10+1-12	383	226	270	2	41.66	17.25	27.02	22.19	44.27	39.44
150	375	2-12+2-10	2-12+0- 0	383	226	226	2	41.31	17.25	27.02	22.19	44.27	39.44
150	375	2-12+2-10	2-12+1- 8	383	226	276	2	41.57	17.25	27.02	22.19	44.27	39.44
150	375	2-12+2-10	2-12+1-10	383	226	305	2	41.62	17.25	27.02	22.19	44.27	39.44
150	375	2-16+0- 0	2- 8+0- 0	402	402	101	1	42.37	17.85	28.10	28.10	45.95	45.95
150	375	2-16+0- 0	2-10+0- 0	402	402	157	1	43.72	17.85	28.10	28.10	45.95	45.95
150	375	2-16+0- 0	2-10+1-12	402	402	270	1	45.22	17.85	28.10	28.10	45.95	45.95
150	375	2-16+0- 0	2- 8+1-12	402	402	214	1	44.63	17.85	28.10	28.10	45.95	45.95
150	375	2-16+0- 0	2-12+0- 0	402	402	226	1	44.79	17.85	28.10	28.10	45.95	45.95
150	375	2-16+0- 0	2-12+1-10	402	402	305	1	45.34	17.85	28.10	28.10	45.95	45.95
150	375	2-16+0- 0	2-12+1-12	402	402	339	1	45.74	17.85	28.10	28.10	45.95	45.95
150	375	2-12+2-12	2- 8+0- 0	452	226	101	2	44.49	17.25	28.64	22.19	45.89	39.44
150	375	2-12+2-12	2- 8+1-10	452	226	179	2	46.79	17.25	28.64	22.19	45.89	39.44
150	375	2-12+2-12	2-10+0- 0	452	226	157	2	46.22	17.25	28.64	22.19	45.89	39.44
150	375	2-12+2-12	2-10+1- 8	452	226	207	2	47.44	17.25	28.64	22.19	45.89	39.44
150	375	2-12+2-12	2-10+1-12	452	226	270	2	48.42	17.25	28.64	22.19	45.89	39.44
150	375	2-12+2-12	2- 8+1-12	452	226	214	2	47.49	17.25	28.64	22.19	45.89	39.44
150	375	2-12+2-12	2-12+0- 0	452	226	226	2	47.72	17.25	28.64	22.19	45.89	39.44
150	375	2-12+2-12	2-12+1-10	452	226	305	2	48.80	17.25	28.64	22.19	45.89	39.44
150	375	2-12+2-12	2-12+1-12	452	226	339	2	49.10	17.25	28.64	22.19	45.89	39.44
150	375	3-12+1-10	2- 8+1-10	418	339	179	2	43.86	17.25	27.86	25.86	45.11	43.11
150	375	3-12+1-10	2-10+0- 0	418	339	157	2	43.39	17.25	27.86	25.86	45.11	43.11
150	375	3-12+1-10	2-12+1-10	418	339	305	2	45.40	17.25	27.86	25.86	45.11	43.11
150	375	3-12+1-12	2- 8+0- 0	452	339	101	2	44.49	17.25	28.64	25.86	45.89	43.11
150	375	3-12+1-12	2-10+0- 0	452	339	157	2	46.22	17.25	28.64	25.86	45.89	43.11
150	375	3-12+1-12	2-10+1-12	452	339	270	2	48.42	17.25	28.64	25.86	45.89	43.11
150	375	3-12+1-12	2- 8+1-12	452	339	214	2	47.49	17.25	28.64	25.86	45.89	43.11
150	375	3-12+1-12	2-12+0- 0	452	339	226	2	47.72	17.25	28.64	25.86	45.89	43.11

Note :- Values shown by * are for Singly reinforced sections.

Table D-2 M_{ur} of Singly and Doubly Reinforced Beams 150 mm Wide for M15 & Fe415

continued....

b mm	D mm	N1-D1+N2-D2 mm	NC-DIAMC 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
150	375	3-12+1-12	2-12+1-12	452	339	339	2	49.10	17.25	28.64	25.86	45.89	43.11
150	375	2-16+1-12	2-10+0- 0	515	402	157	1	53.08	17.85	30.63	28.10	48.48	45.95
150	375	2-16+1-12	2-10+1-12	515	402	270	1	56.21	17.85	30.63	28.10	48.48	45.95
150	375	2-16+1-12	2- 8+1-12	515	402	214	1	54.83	17.85	30.63	28.10	48.48	45.95
150	375	2-16+1-12	2-12+0- 0	515	402	226	1	55.17	17.85	30.63	28.10	48.48	45.95
150	375	2-16+1-12	2-12+1-12	515	402	339	1	57.37	17.85	30.63	28.10	48.48	45.95
150	375	2-16+1-16	2- 8+0- 0	603	402	101	1	56.30	17.85	32.28	28.10	50.13	45.95
150	375	2-16+1-16	2-10+0- 0	603	402	157	1	59.16	17.85	32.28	28.10	50.13	45.95
150	375	2-16+1-16	2-12+0- 0	603	402	226	1	62.06	17.85	32.28	28.10	50.13	45.95
150	375	2-16+1-16	2-12+1-10	603	402	305	1	64.67	17.85	32.28	28.10	50.13	45.95
150	375	2-16+1-16	2-12+1-12	603	402	339	1	65.55	17.85	32.28	28.10	50.13	45.95
150	375	2-16+1-16	2-12+1-16	603	402	427	1	66.83	17.85	32.28	28.10	50.13	45.95
150	375	2-16+2-12	2-10+0- 0	628	402	157	2	57.07	17.02	31.67	27.24	48.69	44.26
150	375	2-16+2-12	2-12+0- 0	628	402	226	2	60.20	17.02	31.67	27.24	48.69	44.26
150	375	2-16+2-12	2-12+1-12	628	402	339	2	64.09	17.02	31.67	27.24	48.69	44.26
150	450	2-10+0- 0	A	157	157	-	1	*22.17	21.92	22.10	22.10	44.03	44.03
150	450	2-12+0- 0	N	226	226	-	1	*30.79	21.87	25.62	25.62	47.49	47.49
150	450	2-12+1-10	C	305	226	-	1	*39.88	21.87	28.80	25.62	50.67	47.49
150	450	2-12+1-12	H	339	226	-	1	*43.62	21.87	30.01	25.62	51.88	47.49
150	450	2-12+2-10	O	383	226	-	2	*46.28	21.17	30.78	25.12	51.94	46.29
150	450	2-16+0- 0	R BARS	402	402	-	1	*49.72	21.77	31.88	31.88	53.65	53.65
150	450	2-12+2-12	2- 8+0- 0	452	226	101	2	56.74	21.17	32.69	25.12	53.86	46.29
150	450	2-12+2-12	2-10+0- 0	452	226	157	2	58.47	21.17	32.69	25.12	53.86	46.29
150	450	2-12+2-12	2-10+1-12	452	226	270	2	60.67	21.17	32.69	25.12	53.86	46.29
150	450	2-12+2-12	2- 8+1-12	452	226	214	2	59.73	21.17	32.69	25.12	53.86	46.29
150	450	2-12+2-12	2-12+0- 0	452	226	226	2	59.97	21.17	32.69	25.12	53.86	46.29
150	450	2-12+2-12	2-12+1-10	452	226	305	2	61.05	21.17	32.69	25.12	53.86	46.29
150	450	2-12+2-12	2-12+1-12	452	226	339	2	61.35	21.17	32.69	29.40	53.86	50.57
150	450	3-12+1-12	2- 8+0- 0	452	339	101	2	56.74	21.17	32.69	29.40	53.86	50.57
150	450	3-12+1-12	2-10+0- 0	452	339	157	2	58.47	21.17	32.69	29.40	53.86	50.57
150	450	3-12+1-12	2-10+1-12	452	339	270	2	60.67	21.17	32.69	29.40	53.86	50.57
150	450	3-12+1-12	2- 8+1-12	452	339	214	2	59.73	21.17	32.69	29.40	53.86	50.57
150	450	3-12+1-12	2-12+0- 0	452	339	226	2	59.97	21.17	32.69	29.40	53.86	50.57
150	450	3-12+1-12	2-12+1-12	452	339	339	2	61.35	21.17	32.69	29.40	53.86	50.57
150	450	2-16+1-12	2-10+0- 0	515	402	157	1	67.03	21.77	34.87	31.88	56.64	53.65
150	450	2-16+1-12	2-12+0- 0	515	402	226	1	69.12	21.77	34.87	31.88	56.64	53.65
150	450	2-16+1-12	2-12+1-12	515	402	339	1	71.32	21.77	34.87	31.88	56.64	53.65
150	450	2-16+1-16	2- 8+0- 0	603	402	101	1	72.63	21.77	36.83	31.88	58.60	53.65
150	450	2-16+1-16	2-10+0- 0	603	402	157	1	75.49	21.77	36.83	31.88	58.60	53.65
150	450	2-16+1-16	2-12+0- 0	603	402	226	1	78.39	21.77	36.83	31.88	58.60	53.65
150	450	2-16+1-16	2-12+1-10	603	402	305	1	81.00	21.77	36.83	31.88	58.60	53.65
150	450	2-16+1-16	2-12+1-12	603	402	339	1	81.88	21.77	36.83	31.88	58.60	53.65
150	450	2-16+1-16	2-12+1-16	603	402	427	1	83.16	21.77	36.83	31.88	58.60	53.65
150	450	2-16+2-12	2-10+0- 0	628	402	157	2	74.09	20.93	36.39	31.10	57.32	52.04
150	450	2-16+2-12	2-12+0- 0	628	402	226	2	77.21	20.93	36.39	31.10	57.32	52.04
150	450	2-16+2-12	2-12+1-12	628	402	339	2	81.10	20.93	36.39	31.10	57.32	52.04

Note :- Values shown by * are for Singly reinforced sections.

Table D-2 M_{ur} of Singly and Doubly Reinforced Beams 150 mm Wide For M15 & Fe415

continued....

b mm	D mm	N1-D1+N2-D2 mm	NC-DIAMC 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	525	2-10+0- 0	A	157	157	-	1	*26.42	25.84	24.31	24.31	50.15	50.15
150	525	2-10+1- 8	N	207	157	-	1	*34.18	25.84	27.30	24.31	53.14	50.15
150	525	2-12+0- 0	C	226	226	-	1	*36.92	25.79	28.26	28.26	54.05	54.05
150	525	2-12+1-10	H	305	226	-	1	*48.13	25.79	31.85	28.26	57.63	54.05
150	525	2-12+1-12	O	339	226	-	1	*52.81	25.79	33.21	28.26	59.00	54.05
150	525	2-12+2-10	R	383	226	-	2	*56.66	25.08	34.22	27.80	59.30	52.88
150	525	2-16+0- 0		402	402	-	1	*60.61	25.68	35.36	35.36	61.04	61.04
150	525	2-12+2-12	B	452	226	-	2	*64.78	25.08	36.41	27.80	61.50	52.88
150	525	3-12+1-10	A	418	339	-	2	*60.80	25.08	35.35	32.66	60.43	57.74
150	525	3-12+1-12	R	452	339	-	2	*64.78	25.08	36.41	32.66	61.50	57.74
150	525	2-16+1-12	S	515	402	-	1	*73.75	25.68	38.77	35.36	64.45	61.04
150	525	2-16+1-16	2- 8+0- 0	603	402	101	1	88.97	25.68	41.03	35.36	66.71	61.04
150	525	2-16+1-16	2-10+0- 0	603	402	157	1	91.82	25.68	41.03	35.36	66.71	61.04
150	525	2-16+1-16	2-10+1-12	603	402	270	1	96.30	25.68	41.03	35.36	66.71	61.04
150	525	2-16+1-16	2-12+0- 0	603	402	226	1	94.72	25.68	41.03	35.36	66.71	61.04
150	525	2-16+1-16	2-12+1-16	603	402	427	1	99.50	25.68	41.03	35.36	66.71	61.04
150	525	2-16+2-12	2-10+0- 0	628	402	157	2	91.10	24.85	40.74	34.64	65.58	59.49
150	525	2-16+2-12	2-12+0- 0	628	402	226	2	94.23	24.85	40.74	34.64	65.58	59.49
150	525	2-16+2-12	2-12+1-12	628	402	339	2	98.11	24.85	40.74	34.74	65.58	59.49
150	525	2-16+2-12	3-12+1-12	628	402	452	2	100.35	24.85	40.74	34.64	65.58	59.49
150	600	2-10+1-8	A	207	157	-	1	*39.79	29.75	29.65	26.37	59.41	56.12
150	600	2-12+0-0	N	226	226	-	1	*43.04	29.70	30.71	30.71	60.41	60.41
150	600	2-10+1-12	C	270	157	-	1	*50.62	29.70	33.04	26.34	62.74	56.04
150	600	2-12+1-8	H	276	226	-	1	*51.68	29.70	33.35	30.71	63.05	60.41
150	600	2-12+1-10	O	305	226	-	1	*56.39	29.70	34.68	30.71	64.38	60.41
150	600	2-12+1-12	R	339	226	-	1	*62.00	29.70	36.20	30.71	65.90	60.41
150	600	2-12+2-10	BARS.	383	226	-	2	*67.04	29.00	37.42	30.28	66.41	59.28
150	600	2-16+0-0	A	402	402	-	1	*71.50	29.60	38.60	38.60	68.20	68.20
150	600	2-12+1-16	N	427	226	-	1	*75.24	29.60	39.51	30.65	69.11	60.25
150	600	2-12+2-12	C	452	226	-	2	*77.03	29.00	39.87	30.28	68.87	59.28
150	600	3-12+1-10	H	418	339	-	2	*72.11	29.00	38.68	35.67	67.68	64.67
150	600	3-12+1-12	O	452	339	-	2	*77.03	29.00	39.87	35.67	68.87	64.67
150	600	2-16+1-12	R	515	402	-	1	*87.70	29.60	42.41	38.60	72.00	68.20
150	600	3-12+1-16		540	339	-	2	*87.95	28.76	42.39	35.50	71.15	64.26
150	600	2-16+1-16	BARS.	603	402	-	1	*99.12	29.60	44.94	38.60	74.54	68.20
150	600	2-16+2-12	2- 8+0- 0	628	402	101	2	105.07	28.76	44.78	37.93	73.54	66.69
150	600	2-16+2-12	2-10+0- 0	628	402	157	2	108.11	28.76	44.78	37.93	73.54	66.69
150	600	2-16+2-12	2-10+1- 8	628	402	207	2	110.51	28.76	44.78	37.93	73.54	66.69
150	600	2-16+2-12	2-10+1-12	628	402	270	2	112.95	28.76	44.78	37.93	73.54	66.69
150	600	2-16+2-12	2- 8+1-12	628	402	214	2	110.71	28.76	44.78	37.93	73.54	66.69
150	600	2-16+2-12	2-12+0- 0	628	402	226	2	111.24	28.76	44.78	37.93	73.54	66.69
150	600	2-16+2-12	2-12+1-10	628	402	305	2	114.13	28.76	44.78	37.93	73.54	66.69
150	600	2-16+2-12	2-12+1-12	628	402	339	2	115.12	28.76	44.78	37.93	73.54	66.69
150	600	2-16+2-12	3-12+1-12	628	402	452	2	117.37	28.76	44.78	37.93	73.54	66.69

Note :- Values shown by * are for Singly Reinforced Section.

Note : (for Table D-2)

N2-D2	Represent, Number-Diameter of bent up bars.
A_{st}	Total area of Tension steel in mm^2
A_{st1}	Total Area of Tension steel of straight bars going into the support.
A_{sc}	Total Area of Compresion 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}).
V_{uc1}	Shear resisted by concrete corresponding to area of steel (A_{st1}). (i.e. excluding bent-up bars)
$V_{ur.min}$	Shear carried by member with minimum stirrups corresponding to (A_{st}).
$V_{ur.min1}$	Shear carried by member with minimum stirrups corresponding to (A_{st1}).

Table D-3

Moment of Resistance of Flanged Section in kN.m. (for D_f => 100 mm.)

Grade of Concrete M15 And Grade of Steel Fe415

b mm	D mm	N1-# mm	N2-# mm	A_{st} mm^2	d mm	b_f/b_w 4	5	6	7	8	9
150	300	2- 8+	0- 0	101	271.0	9.67	9.70	9.72	9.74	9.75	9.76
150	300	2- 8+	1- 8	151	271.0	14.38	14.45	14.50	14.54	14.57	14.59
150	300	2-10+	0- 0	157	270.0	14.90	14.98	15.04	15.08	15.11	15.13
150	300	2-10+	1- 8	207	270.0	19.50	19.64	19.74	19.80	19.85	19.89
150	300	2-12+	0- 0	226	269.0	21.12	21.29	21.40	21.48	21.54	21.59
150	300	2-10+	1-10	236	270.0	22.04	22.23	22.35	22.44	22.51	22.56
150	300	2-12+	1- 8	276	269.0	25.58	25.83	26.00	26.12	26.21	26.28
150	300	2-12+	1-10	305	269.0	28.05	28.36	28.57	28.71	28.82	28.91
150	300	2-10+	2-10	314	257.5	27.56	27.89	28.11	28.27	28.39	28.48
150	300	2-12+	2- 8	327	255.5	28.36	28.72	28.96	29.12	29.25	29.35
150	300	3-12+	0- 0	339	269.0	31.04	31.42	31.68	31.86	31.99	32.10
150	300	2-16+	0- 0	402	267.0	36.07	36.61	36.97	37.23	37.42	37.57
150	300	3-12+	1-10	418	255.5	35.64	36.22	36.61	36.88	37.09	37.25
150	300	2-12+	2-12	452	255.5	38.32	39.01	39.46	39.79	40.03	40.22
150	300	2-16+	1-10	481	267.0	42.49	43.26	43.77	44.14	44.41	44.63
150	300	3-12+	2-10	496	255.5	41.69	42.51	43.05	43.45	43.74	43.97
150	300	2-16+	1-12	515	267.0	45.25	46.13	46.72	47.14	47.46	47.70
150	300	2-16+	2-10	559	251.0	45.47	46.51	47.21	47.70	48.07	48.36
150	300	3-12+	2-12	565	255.5	46.84	47.91	48.62	49.12	49.50	49.80
150	300	2-16+	2-12	628	251.0	50.37	51.68	52.56	53.18	53.65	54.02
150	300	3-16+	0- 0	603	267.0	52.09	53.30	54.11	54.69	55.12	55.46
150	375	2- 8+	1- 8	151	346.0	18.46	18.54	18.59	18.62	18.65	18.67
150	375	2-10+	0- 0	157	345.0	19.16	19.24	19.29	19.33	19.36	19.38
150	375	2-10+	1- 8	207	345.0	25.11	25.25	25.35	25.42	25.47	25.51
150	375	2-12+	0- 0	226	344.0	27.24	27.41	27.53	27.61	27.67	27.72
150	375	2-10+	1-10	236	345.0	28.43	28.61	28.73	28.82	28.89	28.94
150	375	2-12+	1- 8	276	344.0	33.06	33.32	33.49	33.61	33.70	33.77
150	375	2-12+	1-10	305	344.0	36.30	36.61	36.82	36.96	37.08	37.16
150	375	2-10+	2-10	314	332.5	36.07	36.40	36.62	36.78	36.89	36.98
150	375	2-12+	2- 8	327	330.5	37.21	37.57	37.80	37.97	38.10	38.20
150	375	3-12+	0- 0	339	344.0	40.22	40.61	40.86	41.05	41.18	41.29
150	375	2-12+	2-10	383	330.5	43.29	43.78	44.10	44.34	44.51	44.65
150	375	2-16+	0- 0	402	342.0	46.96	47.50	47.86	48.12	48.31	48.46
150	375	3-12+	1-10	418	330.5	46.95	47.53	47.92	48.20	48.41	48.57

Table D-3: Moment of Resistance of Flanged Sections for M15 & Fe415 (Continued)..

b mm	D mm	N1-# mm	N2-# mm	A_{st} mm²	d mm	b_f/b_w 4	5	6	7	8	9
150	375	2-12+	2-12	452	330.5	50.58	51.26	51.71	52.04	52.28	52.47
150	375	3-12+	1-12	452	330.5	50.58	51.26	51.71	52.04	52.28	52.47
150	375	2-16+	1-10	481	342.0	55.51	56.27	56.79	57.15	57.43	57.64
150	375	3-12+	2-10	496	330.5	55.13	55.95	56.50	56.89	57.18	57.41
150	375	2-16+	1-12	515	342.0	59.20	60.08	60.67	61.09	61.41	61.65
150	375	2-16+	2-10	559	326.0	60.61	61.65	62.35	62.84	63.22	63.51
150	375	3-12+	2-12	565	330.5	62.15	63.22	63.93	64.44	64.82	65.11
150	375	2-16+	2-12	628	326.0	67.38	68.70	69.57	70.20	70.67	71.03
150	450	2- 8+	1- 8	151	421.0	22.54	22.62	22.67	22.71	22.73	22.75
150	450	2-10+	0- 0	157	420.0	23.41	23.49	23.55	23.58	23.61	23.64
150	450	2-10+	1- 8	207	420.0	30.73	30.87	30.96	31.03	31.08	31.12
150	450	2-12+	0- 0	226	419.0	33.37	33.54	33.65	33.73	33.79	33.84
150	450	2-10+	1-10	236	420.0	34.81	34.99	35.11	35.20	35.27	35.32
150	450	2-12+	1- 8	276	419.0	40.55	40.80	40.97	41.10	41.19	41.26
150	450	2-12+	1-10	305	419.0	44.55	44.86	45.07	45.22	45.33	45.41
150	450	2-10+	2-10	314	407.5	44.58	44.91	45.13	45.28	45.40	45.49
150	450	2-12+	2- 8	327	405.5	46.06	46.41	46.65	46.82	46.95	47.04
150	450	3-12+	0- 0	339	419.0	49.41	49.79	50.05	50.23	50.37	50.48
150	450	2-12+	2-10	383	405.5	53.67	54.16	54.48	54.72	54.89	55.03
150	450	2-16+	0- 0	402	417.0	57.85	58.39	58.75	59.00	59.20	59.35
150	450	3-12+	1-10	418	405.5	58.27	58.85	59.24	59.51	59.72	59.88
150	450	2-12+	2-12	452	405.5	62.83	63.51	63.96	64.29	64.53	64.72
150	450	2-16+	1-10	481	417.0	68.52	69.29	69.80	70.17	70.44	70.66
150	450	3-12+	2-10	496	405.5	68.57	69.39	69.94	70.33	70.62	70.85
150	450	2-16+	1-12	515	417.0	73.15	74.04	74.62	75.05	75.36	75.61
150	450	2-16+	2-10	559	401.0	75.76	76.80	77.49	77.99	78.36	78.65
150	450	3-12+	2-12	565	405.5	77.47	78.53	79.24	79.75	80.13	80.42
150	450	2-16+	1-16	603	417.0	84.76	85.97	86.78	87.35	87.79	88.12
150	450	2-16+	2-12	628	401.0	84.40	85.71	86.59	87.21	87.68	88.05
150	525	2-10+	0- 0	157	495.0	27.66	27.74	27.80	27.84	27.87	27.89
150	525	2-10+	1- 8	207	495.0	36.34	36.48	36.58	36.65	36.70	36.74
150	525	2-12+	0- 0	226	494.0	39.49	39.66	39.78	39.86	39.92	39.97
150	525	2-10+	1-10	236	495.0	41.19	41.37	41.49	41.58	41.65	41.70
150	525	2-12+	1- 8	276	494.0	48.04	48.29	48.46	48.58	48.67	48.74
150	525	2-12+	1-10	305	494.0	52.81	53.12	53.32	53.47	53.58	53.66
150	525	2-10+	2-10	314	482.5	53.09	53.41	53.63	53.79	53.91	54.00
150	525	2-12+	2- 8	327	480.5	54.90	55.26	55.50	55.67	55.79	55.89
150	525	3-12+	0- 0	339	494.0	58.60	58.98	59.24	59.42	59.56	59.66
150	525	2-12+	2-10	383	480.5	64.05	64.54	64.86	65.09	65.27	65.41
150	525	2-16+	0- 0	402	492.0	68.74	69.28	69.64	69.89	70.09	70.24
150	525	3-12+	1-10	418	480.5	69.58	70.16	70.55	70.83	71.03	71.20
150	525	2-12+	2-12	452	480.5	75.08	75.76	76.21	76.54	76.78	76.97
150	525	2-16+	1-10	481	492.0	81.54	82.31	82.82	83.19	83.46	83.67
150	525	3-12+	2-10	496	480.5	82.01	82.83	83.38	83.77	84.06	84.29
150	525	2-16+	1-12	515	492.0	87.10	87.99	88.58	89.00	89.31	89.56
150	525	2-16+	2-10	559	476.0	90.90	91.94	92.63	93.13	93.50	93.79
150	525	3-12+	2-12	565	480.5	92.78	93.84	94.55	95.06	95.44	95.74
150	525	2-16+	1-16	603	492.0	101.09	102.30	103.11	103.69	104.12	104.46
150	525	2-16+	2-12	628	476.0	101.41	102.72	103.60	104.23	104.70	105.06

Table D-3 Moment of Resistance of Flanged Sections for M15 & Fe415 continued....

b mm	D mm	N1-# mm	N2-# mm	A_{st} mm²	d mm	b_f/b_w 4	5	6	7	8	9
150	600	2-10+	1- 8	207	570.0	41.96	42.10	42.19	42.26	42.31	42.35
150	600	2-12+	0- 0	226	569.0	45.62	45.79	45.90	45.98	46.04	46.09
150	600	2-10+	1-10	236	570.0	47.57	47.75	47.87	47.96	48.03	48.08
150	600	2-12+	1- 8	276	569.0	55.52	55.78	55.95	56.07	56.16	56.23
150	600	2-12+	1-10	305	569.0	61.06	61.37	61.57	61.72	61.83	61.92
150	600	2-10+	2-10	314	557.5	61.59	61.92	62.14	62.30	62.41	62.51
150	600	2-12+	2- 8	327	555.5	63.75	64.11	64.34	64.51	64.64	64.74
150	600	3-12+	0- 0	339	569.0	67.79	68.17	68.43	68.61	68.75	68.85
150	600	2-12+	2-10	383	555.5	74.43	74.91	75.24	75.47	75.65	75.78
150	600	2-16+	0- 0	402	567.0	79.63	80.17	80.53	80.78	80.97	81.12
150	600	3-12+	1-10	418	[illegible]	80.90	81.48	81.86	82.14	82.35	82.51
150	600	2-12+	2-12	452	555.5	87.33	88.01	88.46	88.79	89.03	89.22
150	600	2-16+	1-10	481	567.0	94.55	95.32	95.83	96.20	96.48	96.69
150	600	3-12+	2-10	496	555.5	95.45	96.27	96.82	97.21	97.50	97.73
150	600	2-16+	1-12	515	567.0	101.05	101.94	102.53	102.95	103.26	103.51
150	600	2-16+	2-10	559	551.0	106.04	107.08	107.78	108.27	108.64	108.93
150	600	3-12+	2-12	565	555.5	108.09	109.16	109.87	110.37	110.75	111.05
150	600	2-16+	1-16	603	567.0	117.42	118.64	119.44	120.02	120.45	120.79
150	600	2-16+	2-12	628	551.0	118.42	119.74	120.62	121.24	121.71	122.08

Table D-4

Percentage of Steel Required for Given Moment of Resistance Factor (R_u)

Singly Reinforced Sections

Moment of Resistance factor	Concrete M15			M20		
	Grade of steel			Grade of Steel		
R_u (N/mm²)	Fe250	Fe415	Fe500	Fe250	Fe415	Fe500
	Required percentage of Steel (P_t %)					
0.30	0.140	0.086	0.073	0.140	0.086	0.073
0.35	0.165	0.100	0.084	0.164	0.100	0.084
0.40	0.190	0.114	0.096	0.188	0.114	0.096
0.45	0.215	0.129	0.108	0.212	0.128	0.108
0.50	0.240	0.144	0.120	0.236	0.143	0.120
0.55	0.265	0.159	0.132	0.261	0.158	0.132
0.60	0.290	0.174	0.145	0.286	0.173	0.144
0.65	0.316	0.189	0.158	0.311	0.188	0.156
0.70	0.342	0.205	0.171	0.336	0.203	0.168
0.75	0.368	0.221	0.184	0.361	0.218	0.180
0.80	0.394	0.237	0.197	0.386	0.233	0.193
0.85	0.421	0.253	0.210	0.412	0.248	0.206
0.90	0.448	0.269	0.223	0.438	0.263	0.219
0.95	0.475	0.285	0.237	0.464	0.279	0.232
1.00	0.502	0.302	0.251	0.490	0.295	0.245
1.05	0.530	0.319	0.265	0.516	0.311	0.258
1.10	0.558	0.336	0.279	0.542	0.327	0.271

Table D-4:% Steel Required for Given Moment of Resistance Factor continued....

Moment of Resistance factor	Concrete M 15			M 20		
	Grade of steel			Grade of Steel		
R_u (N/mm²)	Fe250	Fe415	Fe500	Fe250	Fe415	Fe500
	Required percentage of Steel (p_t %)					
1.15	0.586	0.353	0.293	0.569	0.343	0.284
1.20	0.615	0.370	0.307	0.596	0.359	0.297
1.25	0.644	0.388	0.322	0.623	0.375	0.311
1.30	0.673	0.406	0.337	0.650	0.392	0.325
1.35	0.703	0.424	0.352	0.678	0.409	0.339
1.40	0.733	0.442	0.367	0.706	0.426	0.353
1.45	0.764	0.460	0.382	0.734	0.443	0.367
1.50	0.795	0.479	0.398	0.762	0.460	0.381
1.55	0.827	0.498	0.414	0.791	0.477	0.395
1.60	0.859	0.517	0.430	0.820	0.494	0.409
1.65	0.891	0.537	0.446	0.849	0.511	0.424
1.70	0.924	0.557	0.462	0.878	0.529	0.439
1.75	0.958	0.577	0.479	0.908	0.547	0.454
1.80	0.992	0.598	0.496	0.938	0.565	0.469
1.85	1.027	0.619	0.514	0.968	0.583	0.484
1.90	1.062	0.640	0.532	0.999	0.601	0.499
1.95	1.098	0.661	0.551	1.030	0.620	0.514
2.00	1.135	0.683	0.570	1.061	0.639	0.530
2.05	1.172	0.706	-	1.092	0.658	0.546
2.07	-	0.720	-	-	-	-
2.10	1.210	-	-	1.124	0.677	0.562
2.15	1.249	-	-	1.156	0.696	0.578
2.20	1.289	-	-	1.189	0.716	0.594
2.23	1.320	-	-	-	-	-
2.25	-	-	-	1.222	0.736	0.610
2.30	-	-	-	1.255	0.756	0.627
2.35	-	-	-	1.289	0.776	0.644
2.40	Not permissible		-	1.323	0.797	0.661
2.45	-	-	-	1.358	0.818	0.679
2.50	-	-	-	1.394	0.839	0.697
2.55	-	-	-	1.431	0.860	0.715
2.60	-	-	-	1.468	0.882	0.733
2.65	-	-	-	1.505	0.904	0.752
2.67	-	-	-	-	-	0.760
2.70	-	-	-	1.543	0.927	-
2.75	-	-	-	1.582	0.950	-
2.76	-	-	-	-	0.960	-
2.80	-	-	-	1.621	-	-
2.85	-	-	-	1.661	-	-
2.90	-	-	-	1.702	-	-
2.95	-	-	-	1.743	-	-
2.97	-	-	-	1.760	-	-

Table D-5

Percentage of Steel Required for Given Moment of Resistance Factor

Doubly Reinforced Sections -- Concrete M15 and Steel Fe415

R_u N/mm²:	d'/d = 0.05 p_t	p_c	d'/d = 0.10 p_t	p_c	d'/d = 0.15 p_t	p_c	d'/d = 0.20 p_t	p_c
2.08	0.719	0.003	0.720	0.003	0.720	0.003	0.720	0.003
2.10	0.725	0.009	0.726	0.009	0.726	0.010	0.727	0.011
2.20	0.754	0.039	0.757	0.041	0.759	0.045	0.761	0.050
2.30	0.784	0.069	0.787	0.073	0.791	0.080	0.796	0.089
2.40	0.813	0.099	0.818	0.106	0.824	0.115	0.831	0.127
2.50	0.842	0.129	0.849	0.138	0.857	0.150	0.865	0.166
2.60	0.871	0.160	0.880	0.170	0.889	0.185	0.900	0.205
2.70	0.900	0.190	0.910	0.202	0.922	0.220	0.935	0.244
2.80	0.929	0.220	0.941	0.234	0.954	0.255	0.969	0.282
2.90	0.959	0.250	0.972	0.267	0.987	0.290	1.004	0.321
3.00	0.988	0.280	1.003	0.299	1.020	0.325	1.039	0.360
3.10	1.017	0.311	1.034	0.331	1.052	0.360	1.073	0.399
3.20	1.046	0.341	1.064	0.363	1.085	0.395	1.108	0.438
3.30	1.075	0.371	1.095	0.395	1.117	0.430	1.142	0.476
3.40	1.104	0.401	1.126	0.427	1.150	0.465	1.177	0.515
3.50	1.134	0.432	1.157	0.460	1.183	0.500	1.212	0.554
3.60	1.163	0.462	1.188	0.492	1.215	0.535	1.246	0.593
3.70	1.192	0.492	1.218	0.524	1.248	0.571	1.281	0.631
3.80	1.221	0.522	1.249	0.556	1.280	0.606	1.316	0.670
3.90	1.250	0.552	1.280	0.588	1.313	0.641	1.350	0.709
4.00	1.279	0.583	1.311	0.621	1.346	0.676	1.385	0.748
4.10	1.309	0.613	1.342	0.653	1.378	0.711	1.420	0.787
4.20	1.338	0.643	1.372	0.685	1.411	0.745	1.454	0.825
4.30	1.367	0.673	1.403	0.717	1.443	0.781	1.489	0.864
4.40	1.396	0.703	1.434	0.749	1.476	0.816	1.524	0.903
4.50	1.425	0.734	1.465	0.781	1.509	0.851	1.558	0.942
4.60	1.455	0.764	1.495	0.814	1.541	0.886	1.593	0.980
4.70	1.484	0.794	1.526	0.846	1.574	0.921	1.627	1.019
4.80	1.513	0.824	1.557	0.878	1.606	0.956	1.662	1.058
4.90	1.542	0.855	1.588	0.910	1.639	0.991	1.697	1.097
5.00	1.571	0.885	1.619	0.942	1.672	1.026	1.731	1.136
5.10	1.600	0.915	1.649	0.975	1.704	1.061	1.766	1.174
5.20	1.630	0.945	1.680	1.007	1.737	1.096	1.801	1.213
5.30	1.659	0.975	1.711	1.039	1.769	1.131	1.835	1.252
5.40	1.688	1.006	1.742	1.071	1.802	1.166	1.870	1.291
5.50	1.717	1.036	1.773	1.103	1.835	1.201	1.905	1.329
5.60	1.746	1.066	1.803	1.136	1.867	1.236	1.939	1.368
5.70	1.775	1.096	1.834	1.168	1.900	1.271	1.974	1.407
5.80	1.805	1.126	1.865	1.200	1.932	1.306	2.008	1.446
5.90	1.834	1.157	1.896	1.232	1.965	1.341	2.043	1.485
6.00	1.863	1.187	1.927	1.264	1.998	1.376	2.078	1.523
6.10	1.892	1.217	1.957	1.296	2.030	1.411	2.112	1.562
6.20	1.921	1.247	1.988	1.329	2.063	1.446	2.147	1.601
6.30	1.950	1.278	2.019	1.361	2.095	1.481	2.182	1.640

Table D-6 (a)
Number-Diameter Combination of Bars in Ascending Order of Area of Steel

Area mm²	No.	Diam. mm	Area mm²	No.	Diam. mm	Area mm²	No.	Diam. mm
226	* 2	12	1272	** 5	18	2660	** 7	22
339	* 3	12	1357	*** 12	12	2799	*** 11	18
402	* 2	16	1407	** 7	16	2826	*** 9	20
452	* 4	12	1473	* 3	25	2945	** 6	25
508	* 2	18	1520	* 4	22	3040	** 8	22
565	* 5	12	1526	** 6	18	3053	** 12	18
603	* 3	16	1570	** 5	20	3078	** 5	28
628	* 2	20	1608	** 8	16	3141	*** 10	20
678	** 6	12	1781	** 7	18	3420	*** 9	22
760	** 2	22	1809	** 9	16	3437	** 7	25
763	* 3	18	1847	* 3	28	3455	*** 11	20
791	** 7	12	1884	** 6	20	3694	** 6	28
804	* 4	16	1900	** 5	22	3769	*** 12	20
904	** 8	12	1963	* 4	25	3800	*** 10	22
942	* 3	20	2010	** 10	16	3927	** 8	25
982	* 2	25	2035	** 8	18	4180	*** 11	22
1005	* 5	16	2198	** 7	20	4310	*** 7	28
1017	* 4	18	2211	*** 11	16	4418	*** 9	25
1017	** 9	12	2280	** 6	22	4560	*** 12	22
1131	** 10	12	2290	*** 9	18	4909	*** 10	25
1140	* 3	22	2412	*** 12	16	4926	*** 8	28
1206	** 6	16	2454	** 5	25	5400	*** 11	25
1231	* 2	28	2463	** 4	28	5541	*** 9	28
1243	*** 11	12	2512	** 8	20	5890	*** 12	25
1256	* 4	20	2544	*** 10	18			

Note:- * Bars in 1 layer , *** Bars in 3 layers for b = 230 mm
** Bars in 2 layers for b = 230 mm. Maximum 3 layers considered.

Table. D-6(b)
Maximum Number of Bars in One Layer for Different Widths (b in mm.)

WIDTH mm	12	16	18	20	22	25	28	32
	Diameter of Bar in mm							
150	3	3	2	2	2	2	2	1
200	4	4	4	3	3	3	3	2
230	5	5	4	4	4	4	3	3
250	6	5	5	5	4	4	4	3
300	6	6	6	6	5	5	4	4
350	6	6	6	6	6	6	5	4
380	6	6	6	6	6	6	6	5
400	6	6	6	6	6	6	6	5

Note :- Maximum of 6 bars are assumed in one layer.

Table D-6(c)
Effective Cover in mm for Bars of Different Diameter, in Layer

Diameter mm	Number of Layers 1	2	3
12	31	45	58
16	33	49	65
18	34	52	70
20	35	55	75
22	36	58	80
25	38	63	88
28	42	70	98
32	48	80	112
	1	2	3

Line-1 ,Line-2 and Line-3 for 50mm, 75 mm,and 100 cover resp.

Table D-7
Minimum Shear Reinforcement (Stirrups) in Beams for Different Widths

Diam mm	Area of 2 legs mm²	150	230	250	300	350	380	400	450
		Width of Beam in mm							
Steel : Fe 250		Maximum spacing of stirrups in mm.							
6	56	235	150	140	115	100	–	–	–
Steel : Fe 415									
6	56	390	250	230	190	165	150	145	125
8	100	450	450	415	345	295	270	260	230
10	157	450	450	450	450	450	425	405	360
12	226	450	450	450	450	450	450	450	450

Note :- Maximum spacing is limited to 0.75 x eff.depth or 450mm whichever is less

Table D-8
Shear Strength Of 2-legged Vertical Stirrups - Values of V_{us}/d In kN/m Or N/mm

Pitch s mm	Fe 250 Diameter in mm. 6	8	10	12	Fe 415 Diameter in mm 6	8	10	12
75	164	291	455	656	272	483	756	1089
80	154	273	427	615	255	454	709	1021
90	137	243	380	546	227	403	630	907
100	123	218	341	492	204	363	567	817
110	112	199	310	447	185	330	515	742
120	102	182	285	410	170	302	472	681
130	95	168	263	378	157	279	436	629
140	88	156	244	351	146	259	405	583
150	82	146	228	328	136	242	378	544
160	77	136	213	307	127	227	354	510
180	68	121	190	273	113	202	315	454
200	61	109	171	246	102	181	284	408
250	49	87	137	197	82	145	227	327
300	41	73	114	164	68	121	189	272
350	35	62	98	141	58	104	162	233
400	31	55	85	123	51	91	142	204
450	27	48	76	109	45	81	126	181

Table D-9 Ultimate Shear Resistance of Minimum Stirrups $V_{usv.min}$ = 0.35bd in kN for Different Width & Depth of Rectangular Beams. Concrete : M15

DEPTH in mm Total	Effective	150	230	250	300	350	380	400	450
	:	Width of Beam in mm.							
300	250	13.12	20.12	21.87	26.25	30.62	33.25	35.00	39.37
350	300	15.75	24.15	26.25	31.50	36.75	39.90	42.00	47.25
400	350	18.37	28.17	30.62	36.75	42.87	46.55	49.00	55.12
450	400	21.00	32.20	35.00	42.00	49.00	53.20	56.00	63.00
500	450	23.62	36.22	39.37	47.25	55.12	59.85	63.00	70.87
550	500	26.25	40.25	43.75	52.50	61.25	66.50	70.00	78.75
600	550	28.87	44.27	48.12	57.75	67.37	73.15	77.00	86.62
650	600	31.50	48.30	52.50	63.00	73.50	79.80	84.00	94.50
700	650	34.12	52.32	56.87	68.25	79.62	86.45	91.00	102.37
750	700	36.75	56.35	61.25	73.50	85.75	93.10	98.00	110.25

APPENDIX - E

Table E-1 Load Carrying Capacity of Short Columns (Minimum eccentricity 20mm)

Concrete M15 Steel Fe 415

Note:- For b=150mm, Reduction factor = 0.7 & for b=225mm, Reduction factor = 0.9 (Vide Table 9.3.2 of Authors' Book[3.1])

b = 150 , Reduction factor = 0.7

b x D mm mm	P_{uc} kN	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} mm²	452	678	804	904	1030	1206	1256	1432	1608	1834	2010
		P_{us} kN	86	129	153	172	196	230	239	273	306	349	383
150x225	141		227	270	294	313	337	371	380	414	447	490	524
150x300	189		275	318	342	361	385	419	428	462	495	538	572
150x375	236		322	365	389	408	432	466	475	509	542	585	619
150x450	283		-	412	436	455	479	513	522	556	589	632	666
150x525	330		-	459	483	502	526	560	569	603	636	679	713
150x600	378		-	-	531	550	574	608	617	651	684	727	761

b = 225 mm Reduction factor = 0.9

b x D mm mm	P_{uc} kN	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} mm²	452	678	804	904	1030	1206	1256	1432	1608	1834	2010
		P_{us} kN	110	166	197	221	252	295	307	350	393	449	492
225x225	273		383	439	470	494	525	568	580	623	666	722	765
225x300	364		-	530	561	585	616	659	671	714	757	813	856
225x375	455		-	621	652	676	707	750	762	805	848	904	947
225x450	546		-	-	-	767	798	841	853	896	939	995	1038
225x525	637		-	-	-	-	889	932	944	987	1030	1086	1129
225x600	729		-	-	-	-	-	1024	1034	1079	1122	1178	1221
225x675	820		-	-	-	-	-	-	1127	1170	1213	1269	1312

Table E-2 Allowable Combinations of Ultimate Axial Load P_u (in kN) and Ultimate Bending Moment M_u (in kN.m) on Rectangular Columns – 230 mm wide.
Concrete M 15 Steel Fe 415

Depth D mm	Steel No.-Diam. mm	Steel 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 = $k_u = x_u/D$											
230	4 - 12	452	103	19	141	20	194	19	254	16	306	14	352	12
	6 - 12	678	97	24	141	25	205	23	281	20	344	17	400	14
	4 - 16	804	92	27	140	27	215	25	299	21	369	18	428	14
	8 - 12	904	91	30	140	30	217	28	308	23	383	19	447	16
	6 - 16	1206	81	35	139	36	237	32	348	27	489	22	514	18
	4 - 20	1256	77	35	139	36	245	32	359	27	451	22	528	18
	4 - 22	1520	66	40	138	41	263	36	395	30	501	24	587	20
	8 - 16	1608	70	44	137	45	258	40	398	33	509	27	600	22
	6 - 20	1884	58	49	136	49	281	43	439	35	563	29	663	23
	4 - 25	1964	48	49	136	49	296	43	458	35	585	28	687	23
250	6 - 12	678	110	28	153	29	215	27	293	24	360	20	418	16
	4 - 16	804	105	31	153	32	223	30	310	25	383	21	446	17
	8 - 12	904	105	34	152	35	225	33	319	28	397	23	464	19
	6 - 16	1206	96	42	151	42	242	38	357	32	451	26	529	22
	4 - 20	1256	92	42	151	42	249	38	367	32	463	26	543	21
	4 - 22	1520	84	48	150	48	265	43	401	36	510	29	600	24
	8 - 16	1608	86	52	150	52	261	47	403	39	518	32	613	26
	6 - 20	1884	76	57	149	59	280	51	442	42	510	34	675	28
	4 - 25	1964	70	57	149	58	294	51	460	42	591	34	698	28
	6 - 22	2280	64	66	148	67	305	59	493	48	641	39	761	31
300	6 - 12	678	137	39	134	40	242	39	328	35	401	29	466	23
	4 - 16	804	134	43	184	44	248	42	343	36	423	31	493	26
	8 - 12	904	134	47	183	48	248	46	350	39	436	33	510	27
	6 - 16	1206	127	57	182	58	260	55	384	46	485	38	571	31
	4 - 20	1256	125	58	182	59	266	55	392	46	497	38	584	31
	4 - 22	1520	119	66	181	67	273	62	422	52	540	43	638	35
	8 - 16	1608	119	71	181	72	273	68	424	56	548	46	650	31
	6 - 20	1884	113	79	180	80	287	74	458	61	596	50	708	40
	4 - 25	1964	109	80	180	80	298	74	474	61	615	50	730	40
	6 - 22	2280	105	92	179	92	304	85	503	70	661	57	790	46
	8 - 20	2512	101	101	178	101	308	94	524	76	695	62	833	50
350	6 - 12	678	165	51	215	52	276	51	364	45	445	38	517	31
	4 - 16	804	162	56	215	57	279	56	378	48	465	41	542	33
	8 - 12	904	162	61	214	62	281	60	384	52	477	44	569	36
	6 - 16	1206	156	73	213	75	289	72	414	61	523	51	617	42
	4 - 20	1256	155	75	213	76	291	73	422	62	534	51	629	42
	4 - 22	1520	150	85	212	87	298	83	449	69	575	57	681	47
	8 - 16	1608	151	91	212	92	198	88	451	74	582	61	693	50
	6 - 20	1884	145	102	211	103	306	98	482	81	627	67	748	54
	4 - 25	1964	141	103	211	104	311	99	496	82	644	67	768	54
	6 - 22	2280	138	118	210	119	317	113	522	93	688	76	826	61
	8 - 20	2512	135	129	209	130	322	123	541	101	720	83	867	67
	6 - 25	2946	126	145	207	145	336	137	591	112	793	91	957	73
	8 - 22	3040	126	150	207	151	336	143	594	116	801	95	970	76

Table E-2 Allowable Combinations of Ultimate Axial load P_u (in kN) and Ultimate Bending Moment M_u (in kN.m) on Rectangular Columns - 230 Wide

Concrete M 15 Steel Fe 415

Dep D mm	Steel No.Diam. mm	Area mm²	Neutral Axis Factor = $k_u = x_u/D$ 1.0 P_u	1.0 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
230	4 - 12	452	396	9.0	411	7.8	423	6.9	433	6.1	441	5.4	454	4.4
	6 - 12	678	450	10.7	468	9.4	482	8.3	494	7.4	504	6.6	519	5.5
	4 - 16	804	483	11.3	502	9.9	517	8.8	530	7.8	540	7.1	557	5.8
	8 - 12	904	505	12.4	525	10.9	541	9.7	554	8.7	566	7.8	584	6.5
	6 - 16	1206	582	14.2	604	12.5	623	11.2	638	10.0	652	9.1	673	7.5
	4 - 20	1256	598	14.1	621	12.4	639	11.0	655	9.9	668	8.9	690	7.4
	4 - 22	1520	665	15.3	691	13.7	712	12.2	729	11.0	744	10.0	768	8.3
	8 - 16	1608	680	17.1	707	15.2	729	13.6	747	12.2	763	11.1	789	9.3
	6 - 20	1884	753	18.3	782	16.3	806	14.5	827	13.1	844	11.9	872	9.9
	4 - 25	1964	780	17.9	810	15.8	834	14.2	855	12.8	872	11.6	900	9.7
250	6 - 12	678	471	12.8	490	11.2	505	9.9	518	8.8	528	7:9	545	6.5
	4 - 16	804	503	13.6	523	11.9	540	10.5	553	9.4	564	8.5	582	7.0
	8 - 12	904	525	14.8	546	13.0	562	11.6	578	10.4	590	9.4	609	7.8
	6 - 16	1206	599	17.1	624	15.1	643	13.4	660	12.1	674	10.9	696	9.1
	4 - 20	1256	615	17.0	639	15.0	659	13.3	676	12.0	690	10.8	713	9.0
	4 - 22	1520	681	18.8	708	16.6	730	14.9	749	13.4	765	12.1	790	10.1
	8 - 16	1608	695	20.6	724	18.2	747	16.3	767	14.7	784	13.4	811	11.2
	6 - 20	1884	767	22.2	798	19.7	823	17.6	845	15.9	863	14.4	893	12.1
	4 - 25	1964	793	21.8	824	19.3	850	17.3	872	15.6	890	14.2	920	11.9
	6 - 22	2280	865	25.0	900	22.2	929	19.9	954	18.0	975	16.4	1008	13.8
300	6 - 12	678	526	18.4	548	16.1	565	14.2	580	12.7	592	11.4	611	9.4
	4 - 16	804	557	19.6	580	17.2	598	15.3	614	13.7	626	12.3	647	10.2
	8 - 12	904	577	21.3	602	18.8	621	16.7	638	15.0	651	13.5	673	11.2
	6 - 16	1206	648	24.7	676	21.9	698	19.5	717	17.5	733	15.9	758	13.3
	4 - 20	1256	662	24.8	690	21.9	713	19.5	732	17.6	748	15.9	774	13.3
	4 - 22	1520	725	27.6	756	24.5	781	21.9	802	19.7	820	17.9	849	15.0
	8 - 16	1608	740	29.8	772	26.5	798	23.7	821	21.4	839	19.5	870	16.3
	6 - 20	1884	807	32.4	842	28.8	871	25.9	895	23.4	916	21.3	949	17.9
	4 - 25	1964	831	32.3	867	28.7	896	25.7	920	23.2	941	21.1	975	17.7
	6 - 22	2280	901	36.7	940	32.7	973	29.4	1000	26.6	1023	24.3	1061	20.5
	8 - 20	2512	951	40.1	994	35.8	1029	32.2	1058	29.2	1083	26.6	1124	22.5
350	6 - 12	678	584	24.6	608	21.5	627	19.0	643	17.0	657	15.3	678	12.6
	4 - 16	804	613	26.4	638	23.1	659	20.5	676	18.3	691	16.5	713	13.7
	8 - 12	904	633	28.5	660	25.1	682	22.3	700	20.0	715	18.1	739	15.0
	6 - 16	1206	701	33.1	731	29.3	756	26.1	777	23.5	795	21.3	822	17.8
	4 - 20	1256	714	33.3	745	29.4	771	26.3	792	23.6	809	21.4	838	17.9
	4 - 22	1520	774	37.2	808	33.0	836	28.5	860	26.6	873	24.2	911	20.3
	8 - 16	1608	789	39.9	825	35.4	854	31.7	878	28.7	899	26.1	932	21.9
	6 - 20	1884	853	43.6	892	38.7	924	34.8	950	31.4	973	28.6	1009	24.1
	4 - 25	1964	876	43.6	915	38.8	948	34.8	975	31.5	998	28.6	1034	24.1
	6 - 22	2280	944	49.4	987	44.1	1022	39.6	1052	35.9	1078	32.7	1119	27.6
	8 - 20	2512	993	53.8	1039	48.0	1077	43.3	1109	39.2	1137	35.8	1181	30.3
	6 - 25	2946	1096	59.0	1147	52.8	1189	47.6	1225	43.2	1255	39.4	1304	33.4
	8 - 22	3040	1113	61.6	1165	55.1	1208	49.7	1245	45.2	1276	41.3	1327	35.0

Table E-2 Allowable Combinations of Ultimate Axial Load P_u (in kN) and Ultimate Bending Moment M_u (in kN.m) on Rectangular Columns – 230mm Wide

Concrete M 15 Steel – Fe 415

Depth D mm	Steel No.-Diam. mm	Steel 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											
380	4 - 16	804	179	64	233	66	300	64	400	56	491	47	512	39
	8 - 12	904	178	69	233	71	301	69	406	60	503	51	519	48
	6 - 16	1206	173	84	232	85	308	82	434	71	548	59	646	48
	4 - 20	1256	172	85	232	87	311	84	442	71	558	60	658	49
	4 - 22	1520	168	97	231	100	317	95	467	80	597	67	708	54
	8 - 16	1608	168	103	231	105	317	101	469	86	605	71	720	58
	6 - 20	1884	163	116	230	117	324	112	497	94	648	78	774	63
	4 - 25	1964	160	117	229	119	329	113	511	95	664	78	794	63
	6 - 22	2280	157	134	228	135	334	129	535	107	707	88	850	71
	8 - 20	2512	155	146	228	147	338	140	553	117	738	95	898	77
	6 - 25	2946	146	164	226	165	351	157	601	129	808	105	978	85
	8 - 22	3040	146	171	226	171	351	163	604	135	817	109	991	88
400	* 8 - 12	904	151	63	246	65	336	64	447	57	548	49	635	40
	*10 - 12	1130	149	75	245	77	340	75	465	66	579	56	676	46
	* 6 - 16	1206	119	69	246	71	359	70	493	61	612	52	711	43
	4 - 20	1256	183	92	244	94	324	91	455	78	574	65	677	53
	4 - 22	1520	179	105	244	107	330	103	480	88	613	73	727	59
	* 8 - 16	1608	114	91	243	92	367	90	526	77	668	65	784	53
	6 - 20	1884	175	125	242	127	337	122	509	103	662	85	792	69
	4 - 25	1964	172	127	242	129	341	123	522	104	679	85	811	69
	*10 - 16	2010	109	112	243	114	375	110	560	93	723	78	857	63
	6 - 22	2280	169	145	241	146	346	140	546	117	720	96	867	78
	8 - 20	2512	167	158	240	160	350	152	563	127	751	104	907	84
	6 - 25	2946	159	178	239	178	362	170	609	141	819	115	993	93
	8 - 22	3040	159	184	238	185	362	177	611	146	828	119	1006	96
450	8 - 12	904	177	76	277	79	372	78	486	70	594	60	688	49
	10 - 12	1130	175	90	276	93	376	91	503	81	624	69	727	56
	6 - 16	1206	145	83	277	86	395	85	531	75	657	64	763	53
	8 - 16	1608	141	108	275	112	402	108	562	94	711	79	834	65
	6 - 20	1884	100	111	275	113	429	110	613	96	778	80	911	66
	10 - 16	2010	136	133	274	136	409	131	593	114	764	95	905	77
	6 - 22	2280	74	126	274	129	449	124	661	107	848	90	998	73
	8 - 20	2512	93	149	273	151	440	146	662	125	862	104	1023	84
	6 - 25	2946	29	152	273	154	488	149	743	127	968	105	1145	85
	8 - 22	3040	65	172	272	174	463	168	722	143	952	118	1134	95
	10 - 20	3140	86	189	271	189	451	182	712	154	947	127	1135	103
	10 - 22	3800	56	219	269	220	477	211	783	178	1055	146	1270	118
	8 - 25	3928	18	211	269	213	502	204	823	171	1103	140	1321	113
500	10 - 12	1130	201	106	307	110	412	108	542	97	669	82	780	68
	6 - 16	1206	171	99	308	102	431	100	571	90	703	77	816	64
	8 - 16	1608	167	127	306	130	437	127	600	113	755	95	885	78
	6 - 20	1884	126	130	306	133	464	130	650	114	821	96	962	79
	10 - 16	2010	163	156	305	159	443	154	628	135	806	113	954	92
	6 - 22	2280	100	148	305	151	484	146	696	128	891	107	1047	88
	8 - 20	2512	120	174	304	177	474	171	696	148	903	124	1071	101
	6 - 25	2946	56	177	304	181	517	174	775	150	1008	125	1192	102
	8 - 22	3040	93	201	303	204	496	196	753	169	990	140	1180	114
	10 - 20	3140	114	218	302	221	484	213	742	283	985	151	1180	123
	10 - 22	3800	85	254	300	256	509	246	810	210	1090	173	1313	140
	8 - 25	3928	46	246	301	248	534	238	850	203	1138	167	1365	135

**Table E-2 Allowable Combinations of Ultimate Axial Load [illegible] Bending Moment M_u (in kN.m) on Rectangular Col[illegible]
Concrete M 15 Steel Fe - 415**

Depth D mm*	Steel No.-Diam. mm	Area mm²	1.0 P_u	1.0 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 = $k_u = x_u/D$											
380	4 - 16	804	647	30.7	674	26.9	696	23.9	714	21.3	730	19.2	759	15.9
	8 - 12	904	667	33.1	695	29.1	719	25.9	738	23.2	754	21.0	779	17.4
	6 - 16	1206	734	38.5	766	34.0	792	30.3	814	27.3	833	24.7	862	20.7
	4 - 20	1256	747	38.8	779	34.2	806	30.6	828	27.5	847	24.9	877	20.8
	4 - 22	1520	806	43.3	841	38.4	871	34.3	895	31.0	916	28.1	949	23.6
	8 - 16	1608	820	46.2	858	41.0	888	36.8	914	33.2	936	30.2	970	25.4
	6 - 20	1884	883	50.6	924	45.0	957	40.4	985	36.5	1009	33.2	1047	28.0
	4 - 25	1964	905	50.8	946	45.2	980	40.5	1009	36.7	1033	33.4	1071	28.1
	6 - 22	2280	972	57.4	1017	51.2	1054	46.0	1086	41.7	1112	38.0	1155	32.1
	8 - 20	2512	1020	62.4	1068	55.7	1108	50.2	1142	45.5	1171	41.5	1217	35.2
	6 - 25	2946	1121	68.7	1174	61.4	1219	55.3	1256	50.2	1287	45.9	1339	38.9
	8 - 22	3040	1138	71.5	1192	64.0	1238	57.8	1276	52.5	1309	48.0	1361	40.7
400	* 8 - 12	904	721	31.4	748	27.5	770	24.3	788	21.7	802	19.5	826	16.1
	*10 - 12	1130	769	36.3	800	31.9	823	28.3	843	25.4	860	23.0	886	19.1
	* 6 - 16	1206	812	33.7	840	29.5	863	26.2	882	23.4	897	21.1	922	17.5
	4 - 20	1256	769	42.5	803	37.6	830	33.5	853	30.2	873	27.3	903	22.8
	4 - 22	1520	827	47.5	864	42.1	894	37.7	920	34.0	941	30.8	975	25.9
	* 8 - 16	1608	897	42.1	931	37.2	958	33.2	981	29.9	1000	27.1	1030	22.6
	6 - 20	1884	904	55.4	946	49.3	980	44.2	1009	40.0	1033	36.4	1072	36.7
	4 - 25	1964	925	55.7	968	49.6	1003	44.5	1032	40.2	1057	36.6	1090	30.8
	*10 - 16	2010	983	50.6	1022	44.9	1053	40.2	1080	36.3	1102	33.0	1138	27.8
	6 - 22	2280	991	62.9	1038	56.1	1076	50.4	1109	45.7	1136	41.7	1180	35.2
	8 - 20	2512	1039	68.3	1089	61.0	1130	54.9	1165	49.8	1194	45.5	1241	38.5
	6 - 25	2946	1139	75.2	1194	67.3	1239	60.6	1277	55.1	1310	50.3	1362	42.6
	8 - 22	3040	1150	78.3	1212	70.1	1258	62.2	1297	57.4	1331	52.5	1388	44.5
450	8 - 12	904	781	38.8	810	33.9	834	30.0	853	26.7	869	24.0	894	19.8
	10 - 12	1130	828	44.6	860	39.2	886	34.8	908	31.2	926	28.2	954	23.4
	6 - 16	1206	870	41.6	901	36.4	926	32.3	946	28.9	963	26.0	990	21.5
	8 - 16	1608	954	51.8	990	45.7	1020	40.7	1044	36.6	1064	33.2	1097	27.7
	6 - 20	1884	1046	52.3	1083	46.2	1113	41.2	1137	37.1	1158	33.6	1191	28.1
	10 - 16	2010	1038	62.0	1079	55.0	1113	49.2	1142	44.4	1165	40.4	1203	33.9
	6 - 22	2280	1149	58.5	1189	55.8	1222	46.3	1249	41.7	1272	37.9	1308	31.8
	8 - 20	2512	1178	67.9	1223	60.3	1260	54.1	1290	49.0	1316	44.6	1358	37.5
	8 - 25	2946	1324	68.5	1369	60.9	1406	54.6	1438	49.4	1464	45.0	1506	37.9
	8 - 22	3040	1310	77.1	1360	68.7	1401	61.8	1435	56.0	1464	51.0	1511	43.1
	10 - 20	3140	1310	83.5	1363	74.5	1407	67.1	1444	60.8	1475	55.5	1525	47.0
	10 - 22	3800	1470	95.7	1530	85.6	1579	77.2	1621	70.2	1656	64.2	1713	54.4
	8 - 25	3928	1532	92.2	1590	82.4	1639	74.3	1679	67.5	1713	61.6	1769	52.3
500	10 - 12	1130	887	53.5	981	47.0	950	41.7	973	37.3	992	33.7	1022	28.0
	6 - 16	1206	929	50.0	963	43.8	990	38.8	1012	34.7	1030	31.2	1059	25.8
	8 - 16	1608	1012	62.0	1051	54.7	1082	48.7	1108	43.8	1130	39.6	1164	33.1
	6 - 20	1884	1103	62.7	1143	55.3	1175	49.3	1201	44.4	1223	40.2	1258	33.5
	10 - 16	2010	1094	73.9	1138	65.5	1175	58.6	1205	52.9	1230	48.1	1270	40.4
	6 - 20	2280	1206	70.0	1248	61.9	1283	55.4	1312	49.9	1336	45.3	1375	37.9
	8 - 20	2512	1233	81.0	1281	72.0	1320	64.5	1352	58.3	1380	53.1	1424	44.7
	6 - 25	2946	1378	81.9	1426	72.8	1466	65.3	1499	59.0	1527	53.7	1572	45.2
	8 - 22	3040	1363	91.9	1416	81.8	1459	73.6	1496	66.6	1526	60.7	1576	51.3
	10 - 20	3140	1363	99.0	1419	88.6	1465	79.7	1504	72.3	1537	66.0	1589	55.8
	10 - 22	3800	1521	113.8	1583	101.7	1635	91.8	1679	83.4	1716	76.2	1776	64.6
	8 - 25	3928	1582	109.8	1644	98.1	1694	88.5	1737	80.3	1773	73.4	1832	62.2

Table E-2 Allowable Combinations of Ultimate Axail Load P_u (in kN) and Ultimate Bending Moment M_u (in kN-m) on Rectangular Columns - 230mm Wide

Concrete M15 Steel Fe 415

Depth D mm	Steel No.Diam. 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
530	10 - 12	1130	216	116	326	120	433	118	565	107	697	91	812	75
	6 - 16	1206	186	108	326	112	453	110	595	100	731	85	847	70
	8 - 16	1608	183	139	325	142	458	139	622	124	782	105	916	86
	6 - 20	1884	142	142	325	145	485	142	672	126	848	106	993	87
	10 - 16	2010	179	170	324	173	464	168	650	148	833	124	985	102
	6 - 22	2280	116	161	324	164	505	160	718	141	917	118	1078	97
	8 - 20	2512	136	189	323	192	495	187	717	163	928	136	1101	111
	6 - 25	2946	73	193	323	197	538	190	796	165	1033	138	1222	112
	8 - 22	3040	109	218	322	221	517	214	773	185	1015	154	1209	125
	10 - 20	3140	131	237	321	240	504	231	761	200	1009	166	1208	135
	10 - 22	3800	102	275	319	278	528	268	827	230	1112	189	1340	154
	8 - 25	3928	63	267	319	269	553	259	868	223	1161	183	1392	149
550	10 - 12	1130	227	123	339	127	448	126	582	113	716	97	833	80
	6 - 16	1206	196	115	339	119	467	117	611	106	750	91	869	75
	8 - 16	1608	193	147	338	151	472	148	638	132	800	111	927	92
	6 - 20	1884	152	150	337	154	500	150	688	134	866	113	1013	93
	10 - 16	2010	190	179	336	182	478	178	665	157	850	132	1005	108
	6 - 22	2280	127	170	336	174	519	169	733	149	935	125	1098	103
	8 - 20	2512	147	200	335	203	509	197	731	173	946	144	1121	118
	6 - 25	2946	83	204	335	207	551	201	810	176	1050	146	1241	119
	8 - 22	3040	120	230	334	233	530	226	786	197	1031	163	1228	133
	10 - 20	3140	142	249	333	252	518	244	776	212	1025	176	1228	146
	10 - 22	3800	113	290	331	293	542	282	840	244	1128	201	1359	163
	8 - 25	3928	74	281	332	284	567	274	880	236	1176	194	1411	158
	10 - 25	4910	65	358	328	360	582	346	950	296	1302	242	1580	196
600	10 - 12	1130	252	141	370	146	484	144	622	131	763	112	886	93
	6 - 16	1206	221	131	370	136	503	135	651	123	797	106	923	87
	8 - 16	1608	218	167	369	172	508	169	677	152	846	129	990	106
	6 - 20	1884	178	171	368	175	535	172	726	154	912	131	1066	107
	10 - 16	2010	215	203	367	207	514	203	703	180	895	152	1057	124
	6 - 22	2280	153	193	367	198	554	193	771	172	979	145	1150	119
	8 - 20	2512	173	226	366	230	544	224	768	198	989	166	1171	136
	6 - 25	2946	110	231	366	236	586	229	846	202	1093	168	1291	138
	8 - 22	3040	147	260	365	264	565	256	821	225	1074	187	1273	153
	10 - 20	3140	168	281	364	285	552	277	809	243	1067	201	1277	164
	10 - 22	3800	141	327	363	330	575	320	872	278	1168	230	1406	187
	8 - 25	3928	102	317	363	320	600	310	912	270	1216	223	1458	181
	10 - 25	4910	94	403	359	405	614	391	979	338	1339	277	1624	225
650	6 - 16	1206	246	149	401	155	540	154	693	141	845	121	977	100
	6 - 16	1608	243	188	400	194	544	191	717	173	893	147	1043	121
	6 - 20	1884	203	192	399	198	571	195	766	175	958	149	1119	123
	10 - 16	2010	240	227	398	233	549	228	742	205	941	172	1109	142
	6 - 22	2280	178	217	398	222	590	218	809	195	1025	165	1202	135
	8 - 20	2512	199	253	397	258	579	252	805	225	1034	188	1223	154
	6 - 25	2946	136	259	397	264	621	258	883	228	1138	191	1342	157
	8 - 22	3040	172	291	396	295	600	288	858	255	1117	212	1328	177
	10 - 20	3140	194	314	395	319	587	310	845	274	1110	228	1326	186
	10 - 22	3800	166	364	393	368	609	359	906	314	1209	260	1454	212
	8 - 25	3928	128	354	393	358	634	347	946	304	1257	252	1506	205
	10 - 25	4910	120	448	391	451	647	437	1010	380	1377	313	1670	254

Neutral Axis Factor = $k_u = x_u/D$

Table E-2 Allowable Combinations of Ultimate Axial Load P_u (in kN) and Ultimate Bending Moment M_u (in kN.m) on Rectangular Columns – 230mm Wide
Concrete M 15 Steel Fe 415

Depth D mm	Steel No.-Diam. mm	Area mm²	1.0 P_u	1.0 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											
530	10 - 12	1130	922	59.1	959	51.9	988	46.0	1012	41.2	1032	37.2	1064	30.8
	6 - 16	1206	965	55.4	1000	48.5	1028	42.9	1051	38.3	1070	34.5	1100	28.5
	8 - 18	1608	1047	68.4	1087	60.3	1120	53.7	1147	48.2	1170	43.7	1205	36.4
	6 - 20	1884	1138	69.2	1179	61.1	1212	54.4	1240	48.9	1263	44.3	1299	37.0
	10 - 16	2010	1129	81.3	1174	72.1	1212	64.5	1243	58.2	1269	52.8	1310	44.4
	6 - 22	2280	1240	77.2	1284	68.3	1320	61.0	1350	55.0	1375	49.9	1415	41.8
	8 - 20	2512	1267	89.2	1316	79.2	1357	71.0	1390	64.1	1418	58.3	1464	49.1
	6 - 25	2946	1411	90.3	1461	80.2	1502	71.9	1536	65.0	1565	59.1	1611	49.8
	8 - 22	3040	1396	101.1	1450	90.0	1495	80.9	1533	73.2	1564	66.7	1615	56.4
	10 - 20	3140	1395	109.1	1453	97.3	1501	87.5	1540	79.4	1574	72.4	1628	61.3
	10 - 22	3800	1552	124.9	1617	111.7	1670	100.7	1715	91.5	1753	83.6	1814	70.9
	8 - 25	3928	1614	120.7	1677	108.8	1729	97.2	1773	88.2	1810	80.6	1870	68.3
550	10 - 12	1130	946	63.0	984	55.2	1014	49.0	1038	43.8	1059	39.5	1091	32.8
	6 - 16	1206	989	59.0	1025	51.7	1054	45.7	1078	40.8	1097	36.7	1128	30.3
	8 - 16	1608	1070	72.7	1112	64.1	1145	57.1	1173	51.3	1196	46.4	1233	38.7
	6 - 20	1884	1162	73.7	1204	65.0	1238	57.8	1266	52.0	1289	47.1	1326	39.3
	10 - 16	2010	1152	86.4	1199	76.5	1237	68.5	1268	61.7	1295	56.1	1337	47.1
	6 - 22	2280	1263	82.1	1308	72.6	1345	64.9	1376	58.4	1402	53.0	1442	44.4
	8 - 20	2512	1289	94.7	1340	84.1	1381	75.4	1415	68.1	1444	61.9	1490	52.1
	6 - 25	2946	1434	96.0	1485	85.2	1527	76.4	1562	69.0	1591	62.8	1638	52.8
	8 - 22	3040	1418	107.0	1474	95.5	1519	85.8	1557	77.7	1590	70.8	1641	59.8
	10 - 20	3140	1417	115.0	1476	103.0	1525	92.8	1565	84.2	1600	76.8	1655	64.9
	10 - 22	3800	1573	132.0	1639	118.0	1693	106.0	1739	97.0	1778	88.6	1840	75.2
	8 - 25	3928	1635	128.0	1699	114.0	1752	103.0	1797	93.6	1835	85.5	1896	72.5
	10 - 25	4910	1836	160.0	1914	143.0	1978	130.0	2032	118.0	2079	108.0	2153	92.1
600	10 - 12	1130	1006	73.0	1049	64.0	1078	56.7	1104	50.7	1126	45.7	1160	37.8
	6 - 16	1206	1049	68.6	1088	60.0	1118	53.0	1144	47.3	1165	42.5	1197	35.1
	8 - 16	1608	1130	84.0	1173	74.0	1209	65.0	1238	59.1	1263	53.5	1301	44.6
	6 - 20	1884	1221	85.2	1265	75.1	1301	66.8	1331	60.0	1356	54.3	1395	45.3
	10 - 16	2010	1210	99.5	1259	88.1	1300	78.7	1333	71.0	1361	64.4	1405	54.0
	6 - 22	2280	1321	94.7	1369	83.7	1408	74.8	1440	67.3	1468	61.0	1510	51.1
	8 - 20	2512	1347	109.0	1400	96.6	1443	86.6	1499	78.2	1510	71.1	1558	59.8
	6 - 25	2946	1490	110.0	1544	98.1	1588	87.9	1626	79.4	1656	72.2	1705	60.7
	8 - 22	3040	1474	123.0	1532	109.0	1580	98.5	1620	89.1	1654	81.2	1708	68.5
	10 - 20	3140	1473	132.0	1535	118.0	1585	106.0	1628	96.4	1663	87.9	1721	74.3
	10 - 22	3800	1627	151.0	1696	135.0	1753	122.0	1800	111.0	1841	101.0	1906	86.0
	8 - 25	3928	1689	147.0	1756	131.0	1812	118.0	1858	107.0	1898	98.0	1961	83.0
	10 - 25	4910	1888	183.0	1968	164.0	2035	148.0	2091	135.0	2039	123.0	2217	105.0
650	6 - 16	1006	1110	78.7	1151	68.7	1183	60.7	1210	54.1	1232	48.7	1267	40.1
	8 - 16	1608	1189	95.9	1236	84.4	1273	75.0	1304	67.3	1330	60.9	1370	50.6
	6 - 20	1884	1280	97.3	1327	85.6	1365	76.2	1396	68.4	1422	61.8	1464	51.5
	10 - 16	2010	1269	113.0	1321	100.0	1363	89.4	1398	80.5	1427	73.1	1474	61.2
	6 - 22	2280	1380	108.0	1430	95.4	1471	85.1	1505	76.6	1534	69.4	1579	58.0
	8 - 20	2512	1405	123.0	1460	109.0	1506	98.3	1544	88.7	1575	80.6	1626	67.7
	6 - 25	2946	1548	125.0	1605	111.0	1651	99.8	1689	90.1	1721	81.9	1773	68.9
	8 - 22	3040	1531	139.0	1592	124.0	1642	111.0	1684	101.0	1719	91.9	1775	77.5
	10 - 20	3140	1529	150.0	1594	133.0	1647	120.0	1691	109.0	1728	99.4	1788	84.0
	10 - 22	3800	1682	171.0	1754	153.0	1813	138.0	1862	125.0	1904	114.0	1972	97.0
	8 - 25	3928	1744	166.0	1814	148.0	1872	133.0	1920	121.0	1961	110.0	2027	93.8
	10 - 25	4910	1943	207.0	2024	185.0	2094	167.0	2152	152.0	2201	139.0	2181	118.0

Table E-2 Allowable Combinations of Ultimate Axial Load P_u (in kN) and Ultimate Bending Moment M_u (in kN.m) on Rectangular Columns – 230mm Wide

Concrete M 15 Steel – Fe 415

Depth D mm	Steel No. Diam. mm	Steel 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
680	8 – 16	1608	259	201	418	207	566	205	741	186	921	158	1075	130
	6 – 20	1884	218	205	418	211	593	209	790	189	986	160	1151	132
	10 – 16	2010	256	243	417	248	571	244	765	220	968	185	1140	152
	6 – 22	2280	193	232	417	238	611	234	833	210	1053	177	1233	146
	8 – 20	2512	214	270	416	275	601	270	828	241	1061	202	1254	166
	6 – 25	2946	151	276	416	282	643	275	906	245	1164	206	1373	168
	8 – 22	3040	187	310	415	315	621	307	880	273	1143	228	1359	186
	10 – 20	3140	209	334	414	339	608	331	867	293	1136	244	1357	200
	10 – 22	3800	182	387	412	392	630	381	927	336	1234	278	1484	227
	8 – 25	3928	143	376	412	380	655	370	967	326	1283	270	1536	220
	10 – 25	4910	136	476	409	479	667	465	1029	406	1401	334	1699	272
700	*12 – 12	1356	283	188	431	194	583	192	742	175	902	150	1046	123
	* 8 – 16	1606	238	184	430	190	622	187	800	170	971	145	1122	119
	6 – 20	1884	228	214	430	221	607	218	806	198	1005	168	1172	139
	*10 – 16	2010	235	226	429	233	627	227	824	205	1018	173	1187	142
	6 – 22	2280	203	242	429	248	626	244	849	220	1071	186	1255	153
	*12 – 16	2412	232	269	428	275	631	268	848	240	1065	202	1252	165
	8 – 20	2512	224	281	428	287	615	281	844	252	1079	212	1275	174
	6 – 25	2946	161	288	428	294	657	287	895	256	1182	215	1394	176
	8 – 22	3040	198	322	427	328	635	320	921	285	1161	238	1379	195
	10 – 20	3140	219	348	426	353	622	345	882	306	1153	255	1377	209
	10 – 22	3800	192	403	425	407	644	397	941	351	1252	290	1504	237
	8 – 25	3928	154	391	425	396	669	385	982	340	1300	282	1556	230
	10 – 25	4910	146	494	422	498	681	484	1042	424	1418	349	1718	284
750	8 – 16	1608	264	204	461	212	659	208	842	190	1019	163	1176	134
	10 – 16	2010	261	251	460	258	664	253	864	229	1065	194	1241	159
	12 – 16	2412	258	297	458	304	668	297	887	267	1111	225	1305	184
	8 – 20	2512	202	265	458	272	716	265	950	238	1175	201	1367	165
	8 – 22	3040	165	301	457	307	749	298	1014	266	1266	223	1480	182
	10 – 20	3140	197	338	456	344	723	334	987	297	1247	249	1469	203
	12 – 20	3768	193	410	454	416	730	403	1023	357	1320	296	1571	241
	10 – 22	3800	160	388	454	394	757	381	1059	337	1355	280	1603	228
	8 – 25	3928	104	360	454	366	805	353	1122	312	1420	260	1669	211
	12 – 22	4560	154	475	452	480	766	464	1103	409	1443	337	1726	274
	10 – 25	4910	96	472	451	477	816	459	1181	403	1535	333	1829	270
	12 – 25	5892	89	584	448	588	827	566	1239	495	1651	406	1989	329
800	8 – 16	1608	290	226	492	235	696	231	883	212	1067	182	1230	150
	10 – 16	2010	287	276	491	284	700	279	905	253	1122	215	1294	177
	12 – 16	2412	284	326	489	334	705	326	927	295	1158	249	1358	204
	8 – 20	2512	228	292	489	300	752	292	991	264	1221	223	1421	183
	8 – 22	3040	192	330	488	338	786	328	1054	294	1312	247	1532	202
	10 – 20	3140	224	370	487	377	759	366	1026	328	1293	275	1521	224
	12 – 20	3768	219	447	485	454	766	441	1061	393	1365	326	1622	266
	10 – 22	3800	186	424	485	431	794	417	1097	371	1399	309	1654	252
	8 – 25	3928	131	394	485	401	841	387	1161	344	1465	287	1721	233
	12 – 22	4560	181	518	483	524	802	507	1139	449	1487	371	1777	302
	10 – 25	4910	124	515	482	521	852	502	1217	443	1579	366	1879	297
	12 – 25	5892	117	636	479	641	863	617	1273	543	1692	446	2037	362

Table E-2 ΦA P_u-M_u Values for 230mm wide

Table E-2 Allowable Combinations of Ultimate Axial Load P_u (in kN) and Ultimate Bending Moment M_u (in kN.m) on Rectangular Columns 230 mm wide
Concrete M-15 Steel - Fe 415

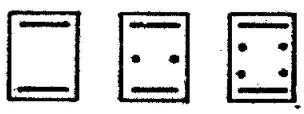
Depth D mm	Steel No.-Diam. mm	Area mm²	1.0 P_u	1.0 M_u	1.05 P_u	1.05 M_u	1.10 P_u	1.10 M_u	1.15 P_u	1.15 M_u	1.20 P_u	1.20 M_u	1.30 P_u	1.30 M_u
			Neutral Axis Factor = $k_u = x_u/D$											
680	8 - 16	1608	1225	103.0	1273	90.8	1312	80.7	1344	72.4	1370	65.4	1412	54.4
	6 - 20	1884	1316	104.0	1364	92.2	1404	82.0	1436	82.0	1436	66.5	1505	55.3
	10 - 16	2010	1304	121.0	1358	107.0	1401	96.0	1437	86.4	1467	78.4	1515	65.6
	6 - 22	2280	1415	116.0	1467	102.0	1510	91.5	1545	82.3	1574	74.5	1620	62.3
	8 - 20	2512	1440	133.0	1497	117.0	1544	105.0	1582	95.1	1615	86.4	1667	72.6
	6 - 25	2946	1583	135.0	1641	119.0	1688	107.0	1728	96.7	1761	87.9	1813	73.9
	8 - 22	3040	1566	150.0	1628	133.0	1680	119.0	1722	108.0	1758	98.5	1816	83.1
	10 - 20	3140	1564	161.0	1630	143.0	1684	129.0	1729	116.0	1767	106.0	1828	89.9
	10 - 22	3800	1716	184.0	1789	164.0	1850	148.0	1900	134.0	1943	122.0	2012	103.0
	8 - 25	3928	1778	178.0	1850	159.0	1909	143.0	1958	130.0	2000	118.0	2067	100.0
	10 - 25	4910	1973	221.0	2058	198.0	2129	179.0	2180	163.0	2239	149.0	2321	127.0
700	*12 - 12	1356	1192	96.1	1239	84.2	1274	74.6	1304	66.7	1329	60.1	1367	49.8
	* 8 - 16	1606	1285	92.0	1330	80.5	1366	71.2	1395	63.6	1420	57.3	1457	47.4
	6 - 20	1884	1340	110.0	1389	96.7	1429	85.9	1462	77.1	1490	69.6	1533	58.0
	*10 - 16	2010	1364	111.0	1414	97.8	1455	87.0	1489	78.2	1517	70.7	1560	59.1
	6 - 22	2280	1439	121.0	1492	107.0	1535	95.8	1571	86.1	1601	78.0	1647	65.2
	*12 - 16	2412	1443	130.0	1499	115.0	1544	103.0	1582	92.7	1614	84.2	1663	70.7
	8 - 20	2512	1463	139.0	1522	123.0	1569	110.0	1608	99.5	1642	90.4	1694	75.9
	6 - 25	2946	1606	141.0	1665	125.0	1714	112.0	1754	101.0	1787	92.0	1841	77.2
	8 - 22	3040	1589	157.0	1653	139.0	1705	125.0	1748	113.0	1785	103.0	1843	86.2
	10 - 20	3140	1587	168.0	1654	150.0	1709	134.0	1755	122.0	1793	111.0	1855	93.9
	10 - 22	3800	1739	192.0	1813	171.0	1874	154.0	1925	140.0	1969	128.0	2038	108.0
	8 - 25	3928	1801	186.0	1873	166.0	1933	149.0	1983	135.0	2026	124.0	2094	105.0
	10 - 25	4910	1995	231.0	2081	207.0	2153	187.0	2213	170.0	2264	155.0	2347	132.0
750	8 - 16	1608	1346	103.0	1393	90.0	1431	79.9	1462	71.3	1487	64.2	1527	53.1
	10 - 16	2010	1424	124.0	1477	109.0	1520	97.2	1555	87.3	1584	78.9	1629	65.8
	12 - 16	2412	1502	145.0	1561	128.0	1608	114.0	1647	103.0	1680	93.7	1732	78.6
	8 - 20	2512	1579	127.0	1633	112.0	1677	100.0	1713	90.0	1744	81.5	1790	68.2
	8 - 22	3040	1716	141.0	1774	125.0	1821	111.0	1861	100.0	1894	91.5	1945	76.9
	10 - 20	3140	1702	159.0	1765	141.0	1816	126.0	1859	114.0	1895	104.0	1951	87.9
	12 - 20	3768	1824	191.0	1896	170.0	1955	153.0	2004	139.0	2046	127.0	2112	107.0
	10 - 22	3800	1864	180.0	1933	160.0	1990	144.0	2037	130.0	2077	118.0	2139	100.0
	8 - 25	3928	1945	164.0	2011	146.0	2064	131.0	2109	118.0	2146	108.0	2204	91.3
	12 - 22	4560	2013	218.0	2092	195.0	2158	176.0	2213	160.0	2260	146.0	2334	124.0
	10 - 25	4910	2138	214.0	2217	191.0	2282	172.0	2337	156.0	2383	143.0	2456	121.0
	12 - 25	5892	2330	263.0	2424	236.0	2501	213.0	2565	194.0	2621	178.0	2708	152.0
800	8 - 16	1608	1407	115.0	1457	100.0	1496	89.0	1529	79.0	1555	71.0	1597	59.0
	10 - 16	2010	1485	138.0	1540	121.0	1585	108.0	1621	96.0	1651	87.0	1699	73.0
	12 - 16	2412	1562	160.0	1623	141.0	1672	126.0	1713	114.0	1748	103.0	1801	86.0
	8 - 20	2512	1639	141.0	1696	124.0	1742	111.0	1779	99.0	1811	90.0	1860	75.0
	8 - 22	3040	1775	157.0	1836	138.0	1855	123.0	1926	111.0	1961	101.0	2014	85.0
	10 - 20	3140	1761	176.0	1826	156.0	1880	140.0	1924	126.0	1961	115.0	2020	96.0
	12 - 20	3768	1882	211.0	1957	188.0	2018	169.0	2069	153.0	2112	140.0	2180	118.0
	10 - 22	3800	1923	199.0	1994	176.0	2053	158.0	2109	143.0	2143	130.0	2207	110.0
	8 - 25	3928	2004	182.0	2072	161.0	2128	145.0	2174	131.0	2212	119.0	2273	100.0
	12 - 22	4560	2070	241.0	2153	215.0	2220	193.0	2277	175.0	2325	160.0	2401	136.0
	10 - 25	4910	2195	236.0	2277	210.0	2345	189.0	2401	172.0	2448	157.0	2523	133.0
	12 - 25	5892	2386	290.0	2482	259.0	2561	234.0	2628	213.0	2685	195.0	2774	166.0

Table E-2 Allowable Combinations of Ultimate Axial Load P_u (in kN) and Ultimate Bending Moment M_u (in kN-m) on Rectangular Columns -- 230 mm Wide
Concrete M 15 Steel - Fe 415

Depth D mm	Steel No.-Diam. 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
840	8 - 16	1608	310	244	517	253	726	250	917	229	1105	198	1274	163
	10 - 16	2010	307	296	516	306	730	300	938	274	1150	233	1338	192
	12 - 16	2412	304	349	514	359	734	351	959	318	1196	269	1401	220
	8 - 20	2512	249	313	514	322	782	315	1023	285	1259	242	1464	198
	8 - 22	3040	213	354	513	363	815	352	1086	317	1349	267	1575	218
	10 - 20	3140	245	396	512	404	789	393	1057	353	1330	296	1564	242
	12 - 20	3768	240	478	510	486	795	472	1091	422	1401	351	1664	287
	10 - 22	3800	208	453	510	461	823	447	1128	399	1436	333	1696	271
	8 - 25	3928	152	422	510	430	870	415	1192	370	1502	309	1762	252
	12 - 22	4560	202	553	508	560	831	542	1169	482	1522	399	1817	325
	10 - 25	4910	145	550	507	557	881	537	1247	476	1614	394	1920	320
	12 - 25	5892	138	677	503	683	892	659	1301	582	1726	479	2077	388
900	10 - 16	2010	338	328	553	340	774	334	988	305	1208	261	1404	215
	12 - 16	2412	335	386	551	396	778	388	1008	354	1252	299	1466	246
	8 - 20	2512	280	347	551	358	826	350	1073	318	1316	270	1528	222
	8 - 22	3040	244	391	550	401	859	391	1134	353	1406	298	1638	244
	10 - 20	3140	276	436	549	446	833	435	1105	392	1386	330	1627	270
	12 - 20	3768	271	525	547	534	839	520	1138	467	1456	390	1726	318
	10 - 22	3800	239	498	547	508	867	493	1175	442	1491	370	1759	302
	8 - 25	3928	184	465	547	474	915	459	1239	411	1557	344	1826	280
	12 - 22	4560	234	606	545	614	875	596	1215	532	1576	442	1879	360
	10 - 25	4910	178	603	544	611	925	591	1292	526	1667	436	1981	355
	12 - 25	5892	171	741	541	748	936	723	1345	641	1778	529	2137	429

For arrangement of bars, see figure shown below.

I II III



Arrangement	Depth in mm	Nos. of Bars
I	230, 250, 300, 350, 380, 400	4, 6, 8
II	* 400, 450, 500, 530, 550, 600, 650, 680, 700	6, 8, 10
III	* 700, 750, 800, 840, 900	8, 10, 12

* Only for bar diameter 12 mm & 16 mm

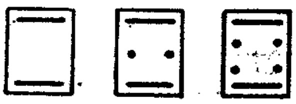
Table E-2: Allowable Combinations of Ultimate Axial Load P_u (in kN) and Ultimate Bending Moment M_u (in kN.m) on Rectangular Columns - 230mm Wide

Concrete M 15 Steel - Fe 415

Depth D mm	Steel No.Diam. mm	Area mm²	Neutral Axis Factor = $k_u = x_u/D$ 1.0 P_u	1.0 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
840	8 - 16	1608	1456	125.0	1507	109.0	1548	96.0	1582	86.0	1610	77.0	1653	64.0
	10 - 16	2010	1533	149.0	1590	131.0	1636	116.0	1674	104.0	1706	94.0	1755	78.0
	12 - 16	2412	1610	173.0	1673	153.0	1724	136.0	1766	123.0	1801	111.0	1856	93.0
	8 - 20	2512	1688	153.0	1746	135.0	1794	120.0	1832	107.0	1865	97.0	1915	81.0
	8 - 22	3040	1823	169.0	1886	149.0	1937	133.0	1979	120.0	2014	109.0	2069	91.0
	10 - 20	3140	1808	190.0	1876	168.0	1931	151.0	1977	136.0	2015	123.0	2075	104.0
	12 - 20	3768	1929	227.0	2006	202.0	2069	181.0	2121	164.0	2165	150.0	2234	127.0
	10 - 22	3800	1970	214.0	2043	190.0	2104	170.0	2154	154.0	2196	140.0	2262	118.0
	8 - 25	3928	2051	196.0	2121	174.0	2179	156.0	2226	141.0	2266	128.0	2327	108.0
	12 - 22	4560	2116	258.0	2201	231.0	2271	208.0	2329	188.0	2378	172.0	2456	146.0
	10 - 25	4910	2241	254.0	2326	226.0	2395	203.0	2452	184.0	2501	169.0	2578	143.0
	12 - 25	5892	2431	311.0	2530	278.0	2611	251.0	2679	228.0	2736	210.0	2828	178.0
900	10 - 16	2000	1606	167.0	1666	146.0	1714	130.0	1754	116.0	1787	105.0	1838	87.0
	12 - 16	2412	1683	193.0	1748	170.0	1802	152.0	1846	136.0	1882	123.0	1940	103.0
	8 - 20	2512	1760	171.0	1822	150.0	1871	134.0	1912	120.0	1946	108.0	1999	90.0
	8 - 22	3040	1895	189.0	1961	166.0	2014	148.0	2058	133.0	2095	121.0	2152	101.0
	10 - 20	3140	1880	211.0	1951	187.0	2008	167.0	2056	151.0	2096	137.0	2158	115.0
	12 - 20	3768	2000	252.0	2080	224.0	2145	211.0	2199	182.0	2245	166.0	2317	140.0
	10 - 22	3800	2041	338.0	2118	211.0	2180	189.0	2332	171.0	2276	155.0	2345	131.0
	8 - 25	3928	2123	218.0	2196	194.0	2255	173.0	2304	156.0	2346	142.0	2410	120.0
	12 - 22	4560	2186	286.0	2274	255.0	2346	230.0	2406	208.0	2457	190.0	2537	161.0
	10 - 25	4910	2311	281.0	2399	250.0	2470	225.0	2530	204.0	2580	186.0	2659	158.0
	12 - 25	5892	2500	344.0	2601	307.0	2685	277.0	2755	252.0	2814	231.0	2909	196.0

For arrangement of bars, see figure shown below.

I II III



Arrangement	Depth in mm	Nos. of Bars
I	230, 250, 300, 350, 380, 400	4, 6, 8
II	* 400, 450, 500, 530, 550, 600, 650, 680, 700	6, 8, 10
III	* 700, 750, 800, 840, 900	8, 10, 12

* Only for bar diameter 12mm & 16mm

TABLE E-3(a) CHARTS FOR COMPRESSION WITH BENDING — RECTANGULAR SECTIONS

Chart – 1 : Interaction Diagram for Combined Bending and Compression Rectangular Section-Equal Reinforcement on Opposite Sides

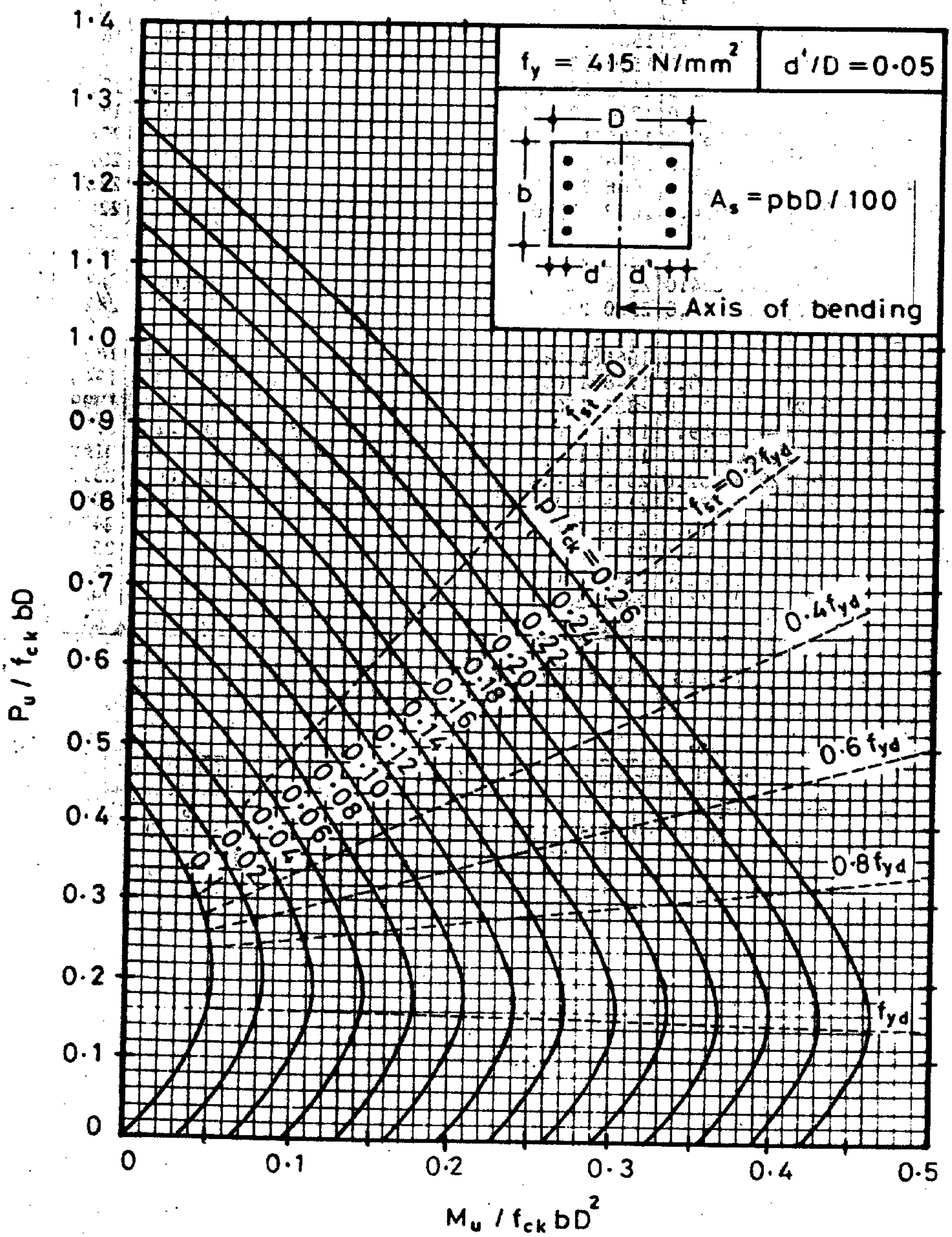


Chart - 2 : Interaction Diagram for Combined Bending and Compression Rectangular Section-Equal Reinforcement on Opposite Sides

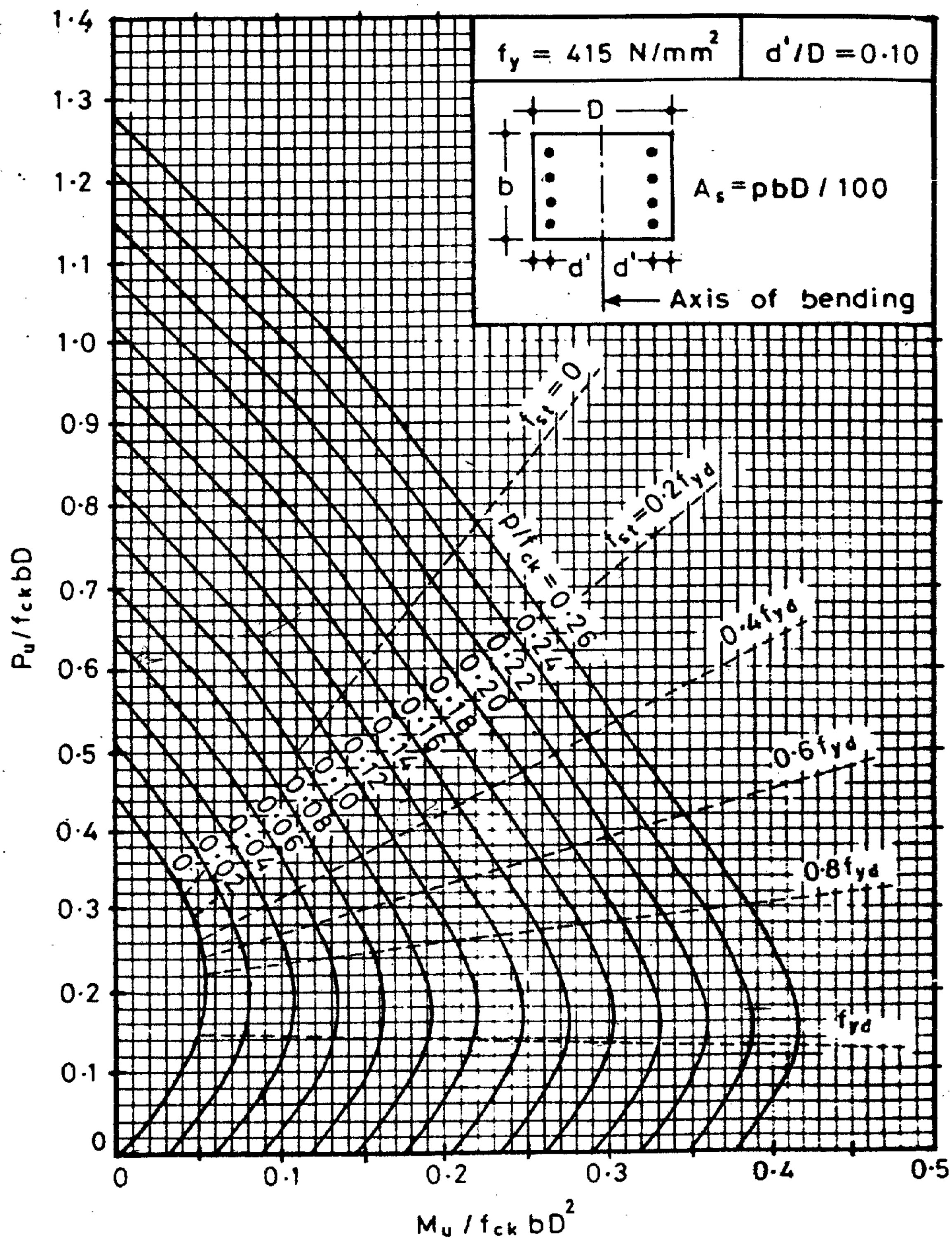


Chart – 3 : Interaction Diagram for Combined Bending and Compression Rectangular Section-Equal Reinforcement on Opposite Sides

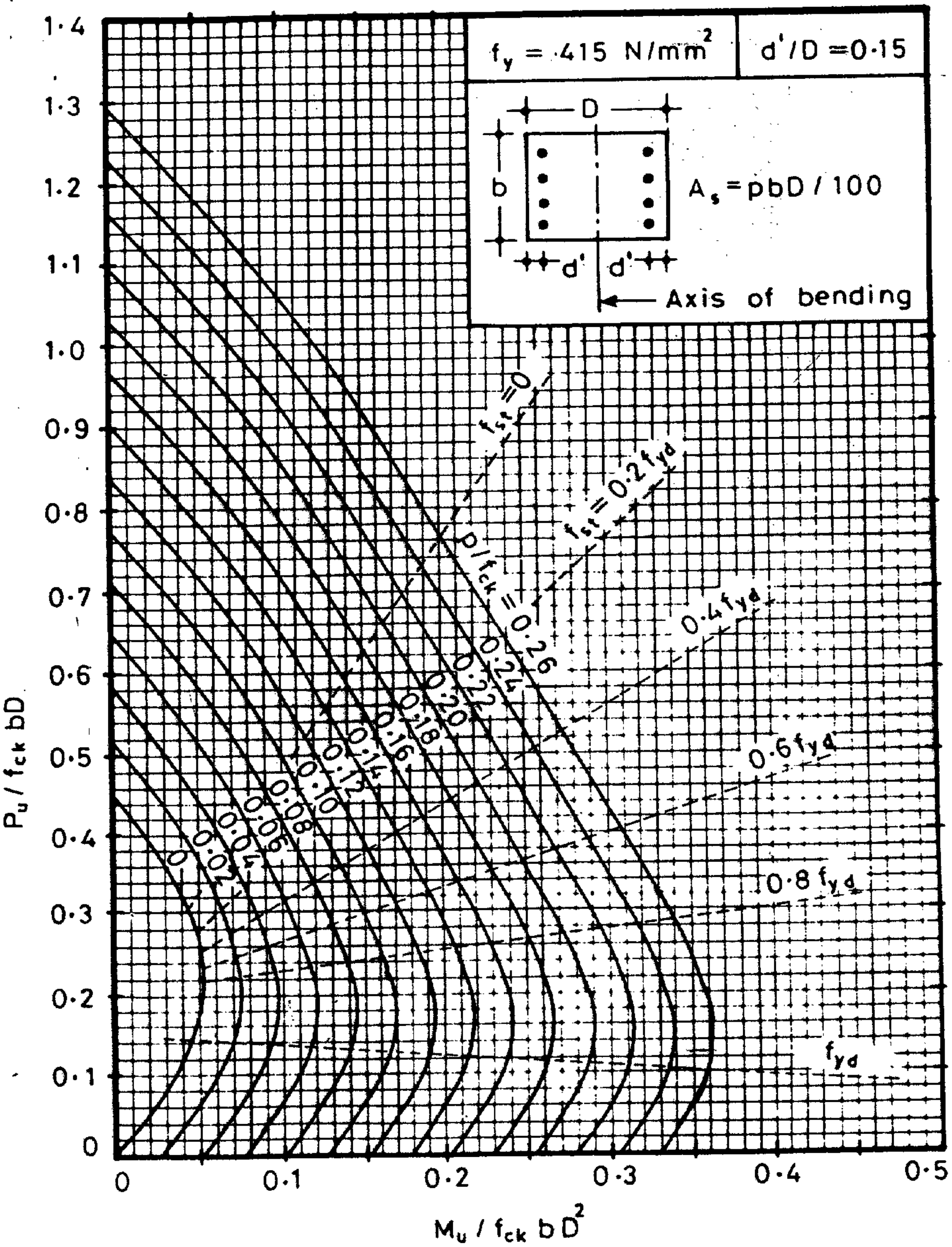


Chart - 4 : Interaction Diagram for Combined Bending and Compression Rectangular Section-Equal Reinforcement on Opposite Sides

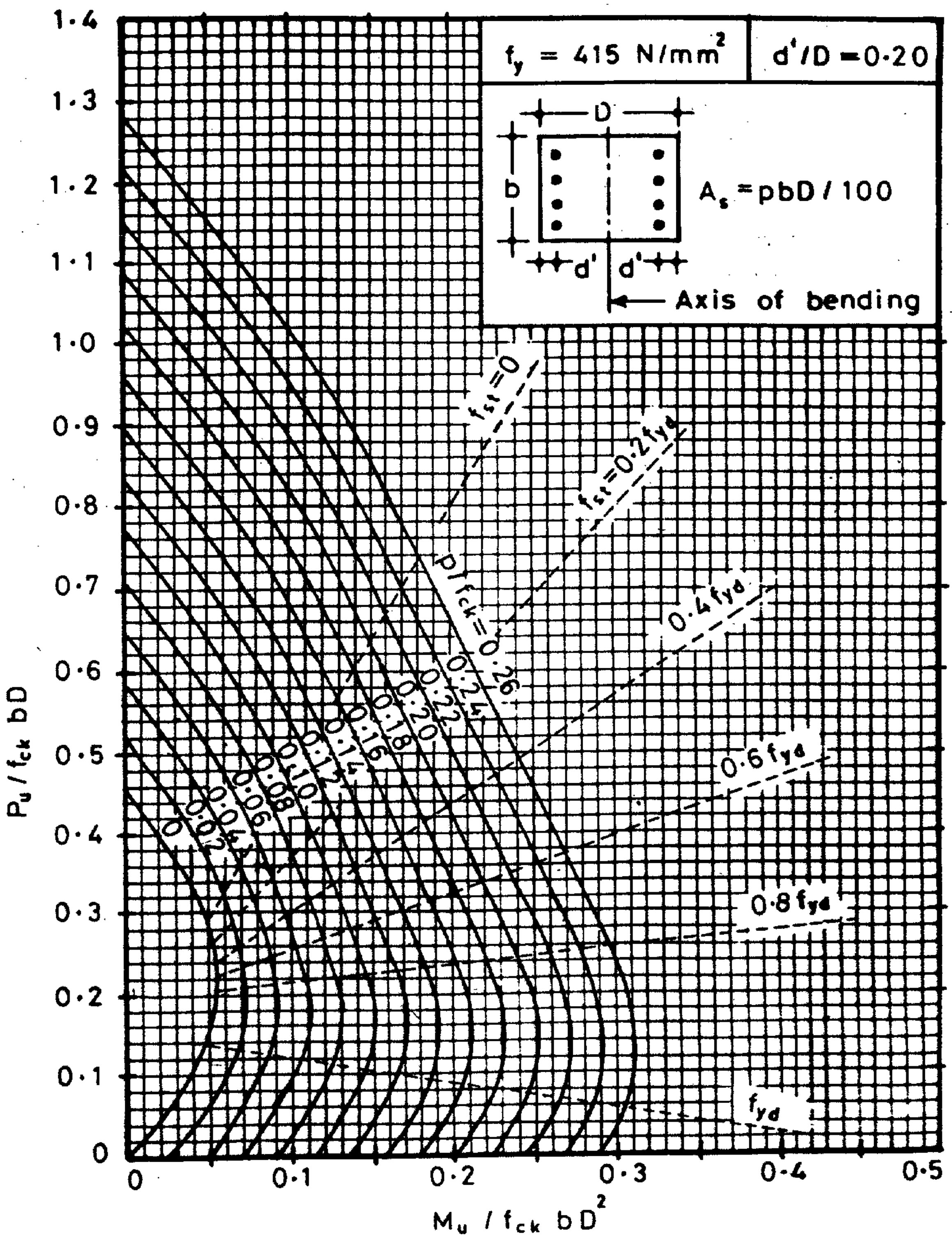


Chart – 5 : Interaction Diagram for Combined Bending and Compression Rectangular Section-Equal Reinforcement on All Sides

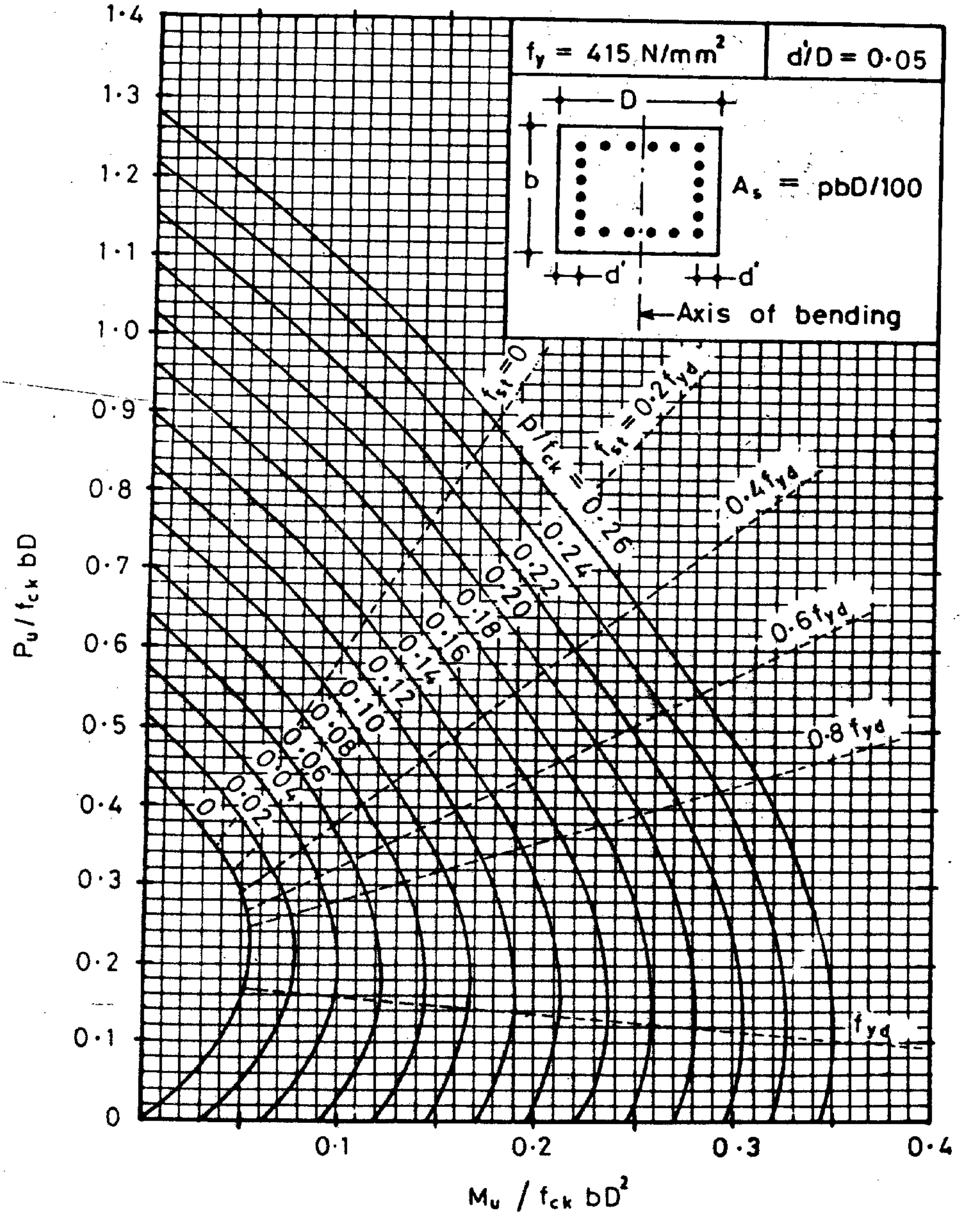


Chart 6 : Interaction Diagram for Combined Bending and Compression Rectangular Section - Equal Reinforcement on All Sides.

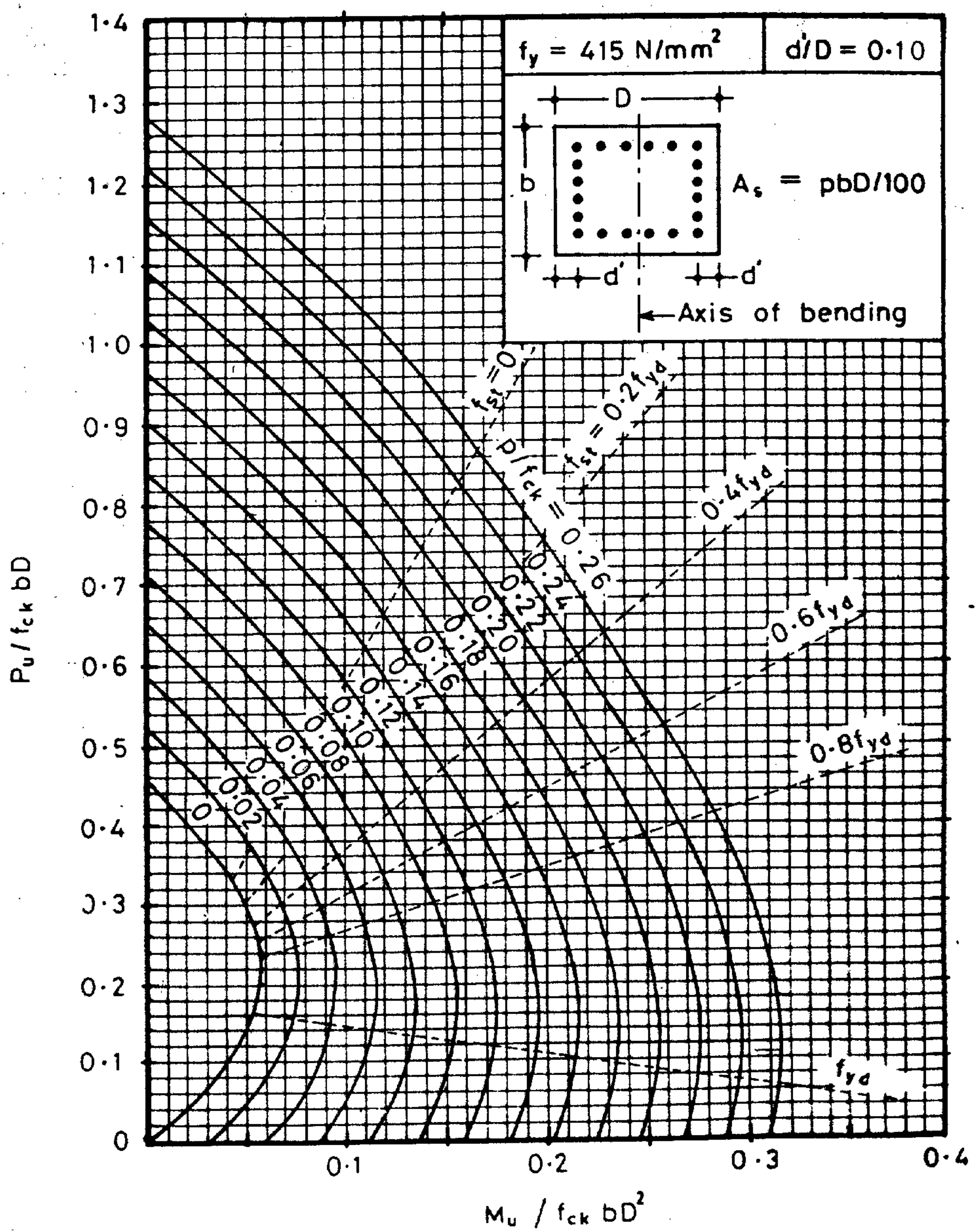


Chart – 7 : Interaction Diagram for Combined Bending and Compression Rectangular Section – Equal Reinforcement on All Sides

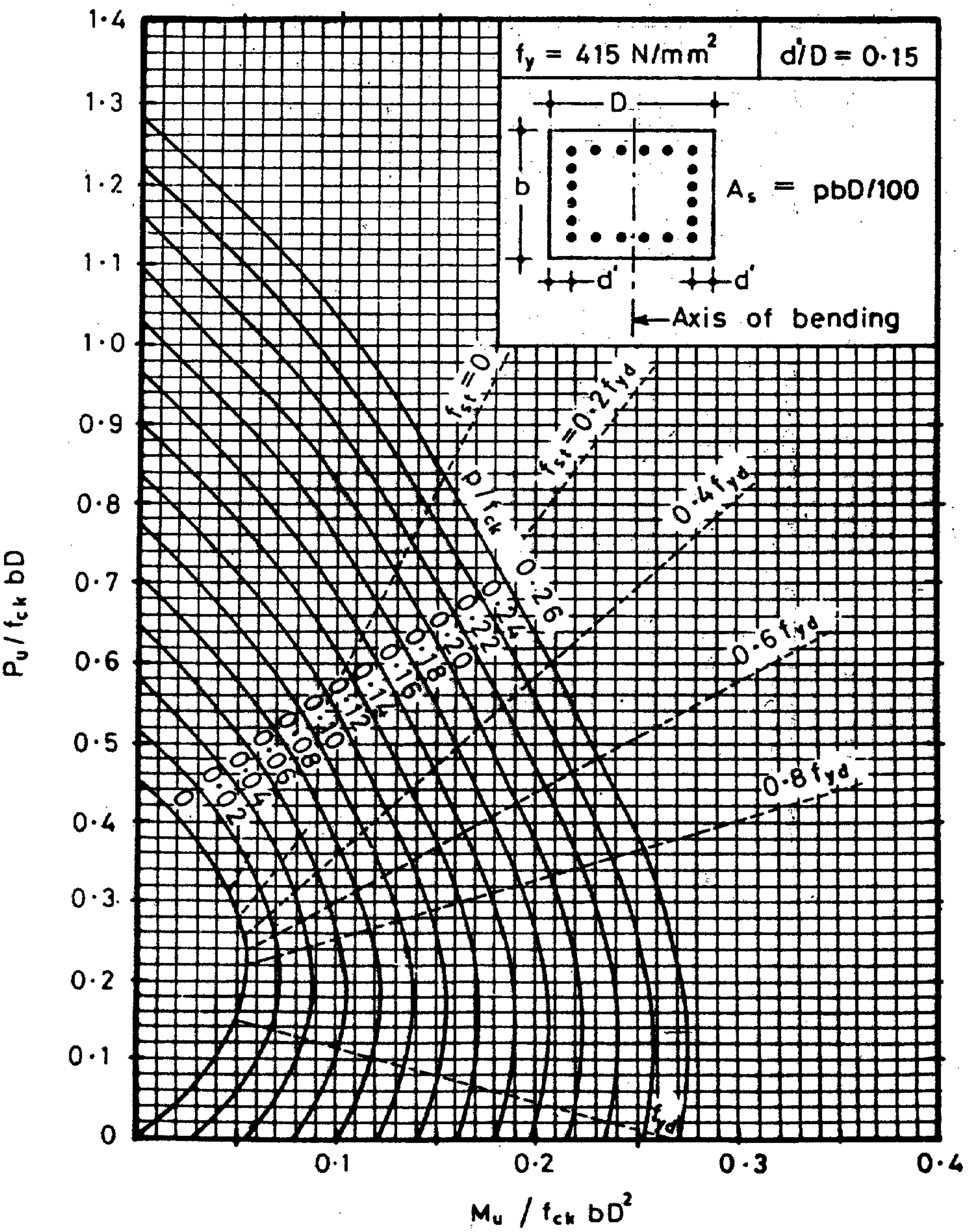


Chart – 8 : Interaction Diagram for Combined Bending and Compression Rectangular Section – Equal Reinforcement on All Sides.

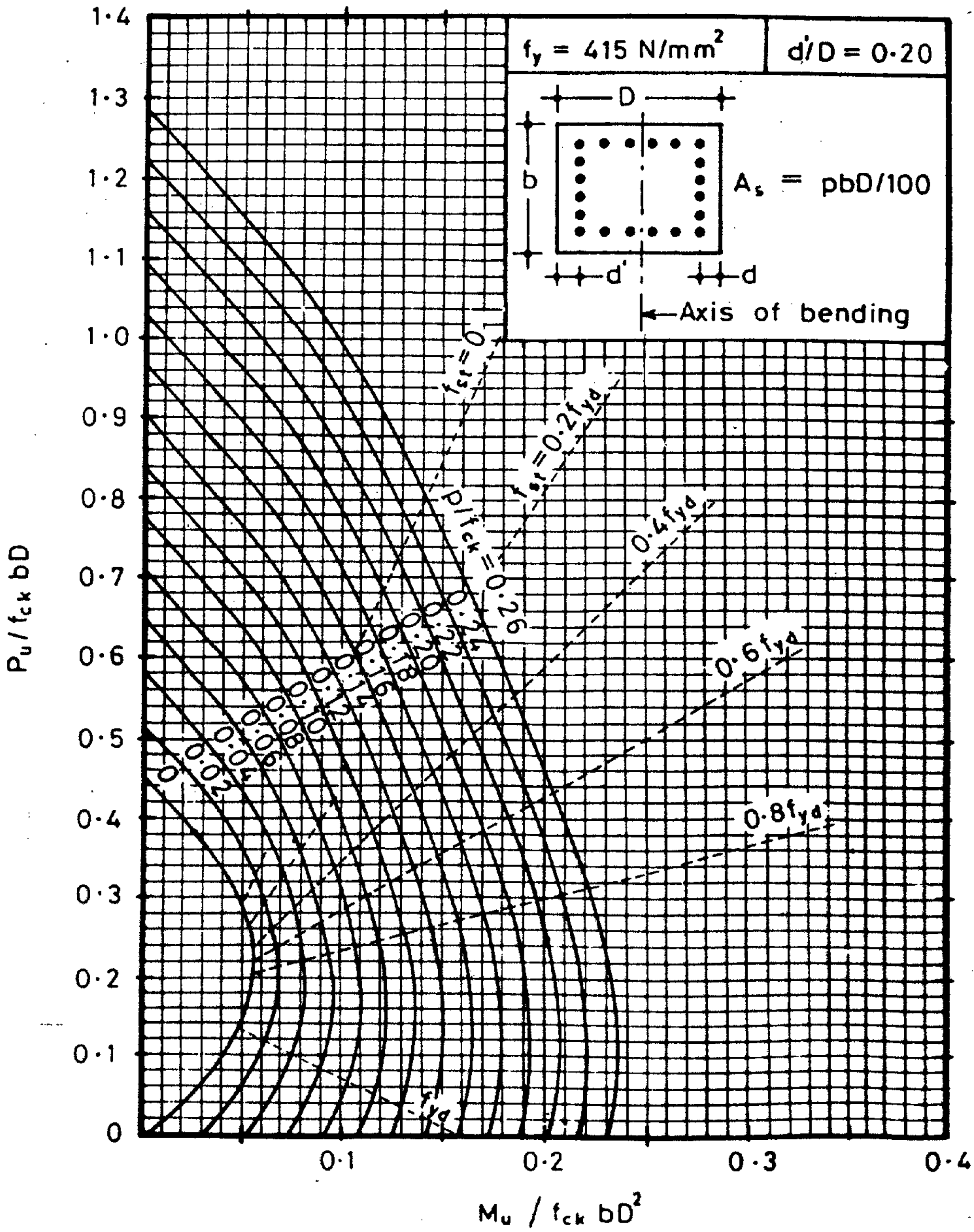


TABLE E-3(b) CHART FOR VALUES OF P_{uz} FOR COMPRESSION MEMBERS

Chart – 9 : Values of P_{uz} for Compression Members.

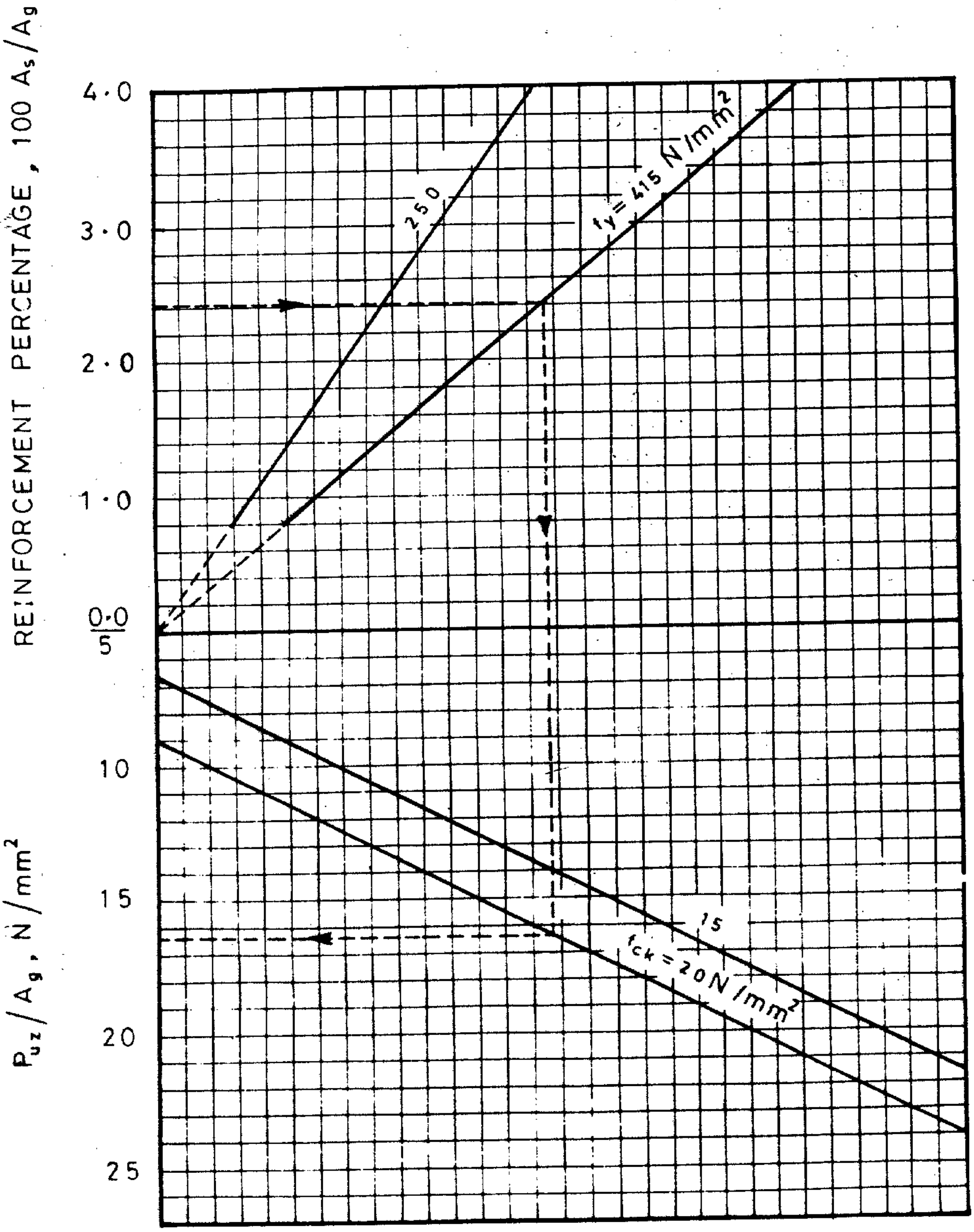


TABLE E-4 CHART FOR BI-AXIAL BENDING IN COMPRESSION MEMBERS

Chart - 10 : Bi-axial Bending for Compression Members

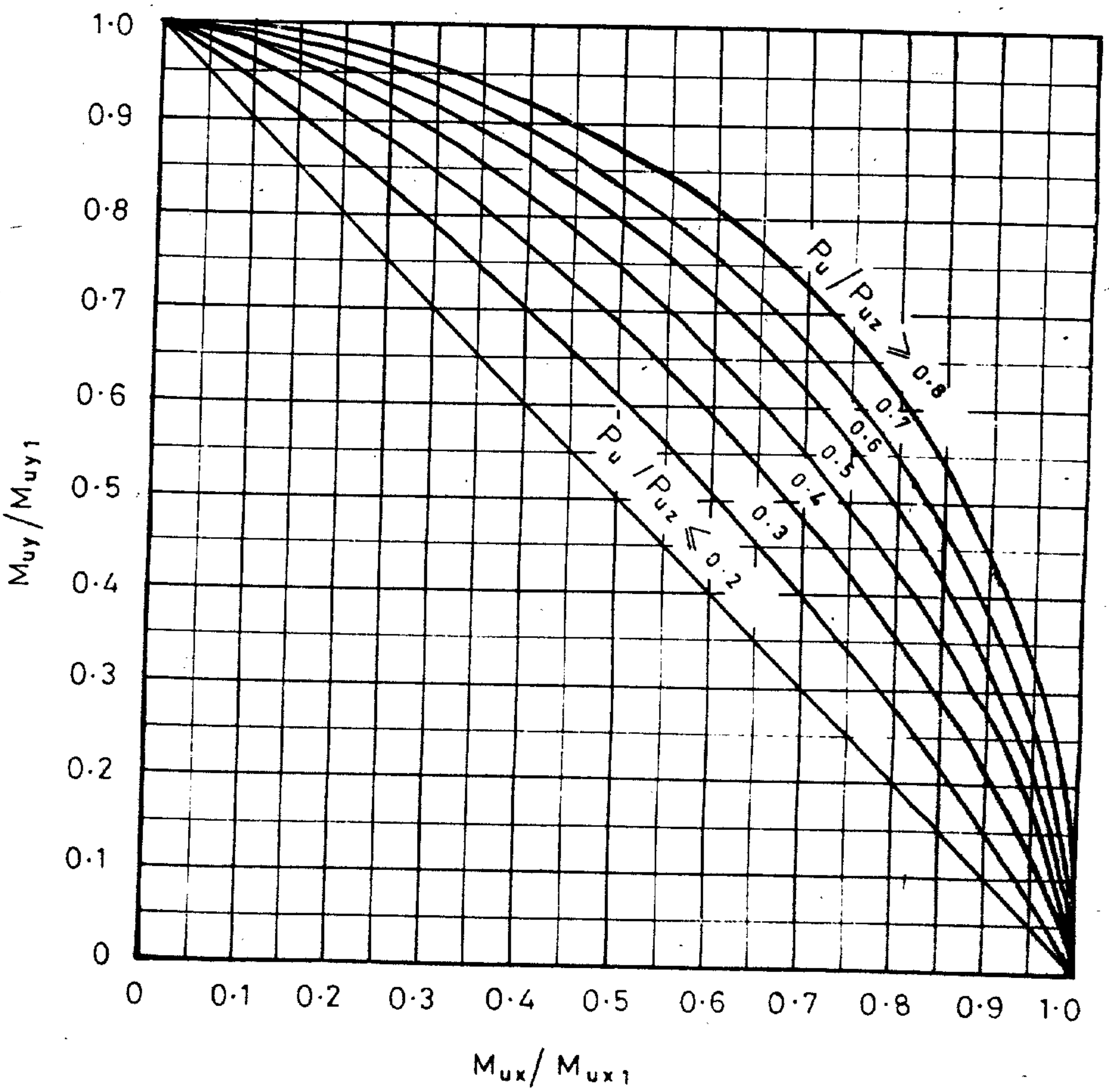


Table E-4 Values of α_n for different values of P_u/P_{uz}

P_u/P_{uz}	<=0.2	0.3	0.4	0.5	0.6	0.7	>=0.8
α_n	1.000	1.167	1.333	1.500	1.667	1.833	2.000

Table E-5 Transverse Reinforcement – Maximum Pitch of Lateral Ties.

Diameter of Main Bar-mm	Diameter of Lateral Tie in mm 5	6	8	10	Remarks
12	190	–	–	–	or 'b' whichever is less
16	240	250	–	–	or 'b' whichever is less
20	240	280	–	–	or 'b' whichever is less
22	–	280	350	–	or 'b' whichever is less
25	–	–	380	400	or 'b' whichever is less
28	–	–	380	440	or 'b' whichever is less

Table E-6 Values of k_1, k_2 for calculation of P_{ub} for Slender Columns.

For Rectangular Section : $P_{ub} = (k_1 + k_2\, p/f_{ck})\, f_{ck} bD$

Values of k_1

Section	d'/D 0.05	0.10	0.15	0.2
Rectangular	0.219	0.207	0.196	0.184

Values of k_2

Section	f_y N/mm²	d'/D 0.05	0.10	0.15	0.2
Rectangular					
-Equal Reinforcement on 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

APPENDIX - F

Table F-1 Properties of Round Bars

Diam mm	Area mm^2	Perimeter mm	Weight kg/m	Length per tonne m	Diam mm	Area mm^2	Perimeter mm	Weight kg/m	Length per tonne 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	1.578	633	32	804.2	100.5	6.313	159
18	254.4	56.5	1.998	500	36	1017.9	113.1	7.990	125

Note:-(1) Basic Weight = 0.00785 $kg/mm^2/m$
= $0.006165(F^2)$ or $(F^2/162)$kg/m where, F is in mm.
(2) The maximum length of bars available ex. stock is 13 m.

Table F-2 Area of Steel for Combinations of Bar Diameter and Number.

No. of Bars	5	6	8	10	12	16	18	20	22	25	28	32
	DIAMETERS OF BARS in mm — 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	3817	4712	5702	7363	9236	12063
16	314	452	804	1256	1809	3217	4071	5026	6082	7854	9852	12868

Table F-3 Area of Steel for Combinations of Bar Diameter and Spacing

Spacing 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

APPENDIX - G

Table G-1 IMPOSED FLOOR LOADS FOR DIFFERENT OCCUPANCIES

Sr. NO.	OCCUPANCY CLASSIFICATION	UNIFORMLY DISTRIBUTED LOAD (UDL) kN/m^2	CONCENTRATED LOAD kN
(1)	(2)	(3)	(4)
i)	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	same as (a) dwelling houses Sr. No. (4)	
	c) Hotels, hostels, boarding houses, lodging houses, dormitories, residential clubs :		
	1) Living rooms, bed rooms& 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 rooms2.5	2.7	
	7) Rooms for indoor games	3.0	1.8
	8) Baths and toilets	2.0	–
	9) Corriodors, passages, staircases 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

TABLE G-1 : IMPOSED FLOOR LOADS FOR DIFFERENT OCCUPANCIES - *Continued*

Sr. NO.	OCCUPANCY CLASSIFICATION	UNIFORMLY DISTRIBUTED LOAD (UDL) kN/m²	CONCENTRATED LOAD kN
(1)	(2)	(3)	(4)
	Residential Buildings continued :		
	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
	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
ii)	EDUCATIONAL BUILDINGS		
	a) Class rooms and lecture rooms (not used for assembly purposes)	3.0	2.7
	b) Dining rooms, cafeterias& 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/m² for a minimum height of 2.2 m + 2.0 kN/m² 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
	m) 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
	n) 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
iii)	INSTITUTIONAL BUILDINGS		
	a) Bed rooms, wards, dressing rooms, dormitories and lounges	2.0	1.8
	b) Kitchens, laundries and laboratories	3.0	4.5

Table G-1 IMPOSED FLLOR LOADS FOR DIFFERENT OCCUPANCIES - Contd

Sr. NO.	OCCUPANCY CLASSIFICATION	UNIFORMLY DISTRIBUTED LOAD (UDL) kN/m²	CONCENTRATED LOAD kN
(1)	(2)	(3)	(4)
	Institutional Buildings Continued :		
	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
	g) Corridors, passages, lobbies and staircases including fire escapes- as per the floor serviced 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 theygive access but with a minimum of 4.0	1.5 per metre run concentrated at the outer edge
iv)	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 rooms5.0	–	
	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	–
	j) Corridors passages, staircases including fire escapes	4.0	4.5
	k) Balconies	same as (iii) Institutional Buildings Sr. No. (j)	
	m) Boiler rooms and plant rooms including weight of machinery	7.5	4.5
	n) 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
v)	BUSINESS AND OFFICE BUILDINGS (see also 3.1.2		
	a) Rooms for general use with separate storage	2.5	2.7
	b) Rooms without separate storage	4.0	4.5

TABLE G-1 IMPOSED FLOOR LOADS FOR DIFFERENT OCCUPANCIES - Contd.

Sr. NO.	OCCUPANCY CLASSIFICATION	UNIFORMLY DISTRIBUTED LOAD (UDL) kN/m^2	CONCENTRATED LOAD kN
(1)	(2)	(3)	(4)
	Business and Office Buildings Continued :		
	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
	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
	j) Corridors, passages, lobbies and staircases including fire escapes – as per the floor serviced (excluding stores) but not less than	4.0	4.5
	k) Bath and toilet rooms	2.0	–
	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
	n) Stationay Stores4.0 for each metre of storage height	9.0	
	p) Boiler rooms and plant rooms – to be calculated but not less than	5.0	6.7
	q) Libraries	see Sl No. (ii)	
vi)	MERCANTILE BUILDINGS		
	a) Retail shops	4.0	3.6
	b) Wholesale shops – to be calculated but not less than	6.0	4.5
	c) Office rooms	2.5	2.7
	d) Dining rooms, restaurants & 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
	j) Corridors, passages, staircases subject to loads greater than from crowds, such as wheeled vehicles, trolleys and the like.	5.0	4.5
	k) Balconies	Same as (v) Business and Office Buildings Sr. No. (m)	
vii)	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

TABLE G-1 IMPOSED FLOOR LOADS FOR DIFFERENT OCCUPANCIES - Continued.

Sr. NO.	OCCUPANCY CLASSIFICATION	UNIFORMLY DISTRIBUTED LOAD (UDL) kN/m^2	CONCENTRATED LOAD kN
(1)	(2)	(3)	(4)
	Industrial Buildings Contunued :		
	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 straircases 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	–
viii)	STORAGE BUILDINGS**		
	a) Storage rooms (other than cold storage) warehouses – to be calculated based on the bulk density of materials stored but not less than	2.4 kN/m^2 per each metre of storage height with a minimum of 7.5 kN/m^2.	7.0
	b) Cold storage– to be calculated but not less thanmetre	5.0 kN/m^2 per each of storage height with a minimum of 15kN/m^2.	9.0
	c) Corridors, passage and staircases including fire escapes – 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, 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 purposes 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 "ligth", "medium" and "heavy" are introduced in order to allow for more economical desinges but the terms have no special meaning in themselves other than the imposed load for which the relevant floor is designed. 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, workshops, store rooms, etc. the actual load coming from the use of such equipment shall be ascertained and design should cater to such loads.

Note : 1) The concentrated loads specified may be asumed 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.