

## International Journal of Innovative Research in Science, Engineering and Technology

(A High Impact Factor, Monthly, Peer Reviewed Journal)

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Vol. 6, Issue 10, October 2017

# Designing and Development of the Multi Storied Residential Building

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**ABSTRACT:** Structural planning and design is an art and science of designing with economy and elegance, serviceable and durable structure. The entire process of structural planning and designing requires not only imagination and conceptual thinking but also sound knowledge of science of structural engineering besides knowledge of practical aspects, such as relevant design codes and byelaws backed up by example experience.

The process of design commence with planning of structural primarily to meet the defined as he is not aware of various implications involved in the process of planning and design. The functional requirements and aspects of aesthetics are locked into normally be the architect while the aspect of the safety, serviceability, durability and economy of the structure are attended by structural designer.

For this purpose a site is selected in which the building has three floors including a ground floor, it consists of all the rooms required for a residential house like bedroom, toilet, living/dining, kitchen and store room.

The design of the constructing depends upon the minimum necessities as prescribed inside the Indian Standard Codes. The minimal necessities bearing on the structural protection of homes are being protected by means of manner of laying down minimum design masses which should be assumed for lifeless hundreds, imposed masses, and other external loads, the structure would be required to bear. Strict conformity to loading standards recommended on this code, it is hoped, will not most effective make sure the structural safety of the homes which might be being designed..

### I. INTRODUCTION

The design of the constructing depends upon the minimum necessities as prescribed inside the Indian Standard Codes. The minimal necessities bearing on the structural protection of homes are being protected by means of manner of laying down minimum design masses which should be assumed for lifeless hundreds, imposed masses, and other external loads, the structure would be required to bear. Strict conformity to loading standards recommended on this code, it is hoped, will not most effective make sure the structural safety of the homes which might be being designed. To carry out an correct analysis a structural engineer must decide such information as structural loads, geometry, help conditions, and materials properties. The consequences of such an analysis commonly include support reactions, stresses and displacements. This information is then in comparison to criteria that indicate the situations of failure. Advanced structural analysis may additionally take a look at dynamic reaction, balance and non-linear conduct. The aim of design is the achievement of an appropriate chance that systems being designed will carry out satisfactorily for the duration of their intended existence. With the right degree of safety, they have to maintain all the hundreds and deformations of everyday construction and use and have ok durability and adequate resistance to the results of seismic and wind. Structure and structural factors shall generally be designed via Limit State Method. Account must be taken of universal theories, test and enjoy and the want to design for sturdiness. Design, which includes design for durability, production and use in service need to be considered as an entire. The realization of design targets requires compliance with really described requirements for substances, production, workmanship and also upkeep and use of structure in service.

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## II. LITERATURE REVIEW

### Method of flexibility coefficients

The method of analysis is comprises reducing the hyper static structure to a determinate structure form by removing the redundant support (or) introducing adequate cuts (or) hinges.

Limitations: It is not applicable for degree of redundancy > 3

### Slope displacement method

It is advantageous when kinematic indeterminacy < static indeterminacy. This procedure was first formulated by axle bender in 1914 based on the applications of compatibility and equilibrium conditions. The method derives its name from the fact that support slopes and displacements are explicitly computed. Set up simultaneous equations is formed the solution of these parameters and the joint moment in each element or computed from these values.

Limitations:

A solution of simultaneous equations makes methods tedious for manual computations. This method is not recommended for frames larger than two bays and two storeys. This method involves distributing the known fixed end moments of the structural member to adjacent members at the joints in order satisfy the conditions of compatibility.

Limitations of Hardy cross method:

It presents some difficulties when applied to rigid frame especially when the frame is susceptible to side sway. The method cannot be applied to structures with intermediate hinges.

### Kane's method

This method over comes some of the disadvantages of Hardy cross method. Kane's approach is similar to H.C.M to that extent it also involves repeated distribution of moments at successive joints in frames and continuous beams. However there is a major difference in distribution process of two methods. H.C.M distributes only the total joint moment at any stage of iteration. The most significant feature of Kane's method is that process of iteration is self-corrective. Any error at any stage of iterations corrected in subsequent steps consequently skipping a few steps error at any stage of iteration is corrected in subsequent consequently skipping a few steps of iterations either by oversight or by intention does not lead to error in final end moments.

Advantages:

It is used for side way of frames.

Limitations:

The rotational of columns of any storey should be functioning single rotation value of same storey. The beams of storey should not undergo rotation when the column undergoes translation. That is the column should be parallel. Frames with intermediate hinges cannot be analysis.

### Approximate method

Approximate analysis of hyper static structure provides a simple means of obtaining a quick solution for preliminary design. It makes some simplifying assumptions regarding Structural behavior so to obtain a rapid solution to complex structures. The usual process comprises reducing the given indeterminate configuration to determine structural system by introducing adequate no of hinges. It is possible to sketch the deflected profile of the structure for the given loading and hence by locate the point of inflection. Since each point of inflection corresponds to the location of zero moment in the structures. The inflection points can be visualized as hinges for the purpose of analysis. The solution of structures is simpler once the inflection points are located. The loading cases are arising in multistoried frames namely horizontal and vertical loading. The analysis carried out separately for these two cases.

Horizontal cases:

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The behavior of a structure subjected to horizontal forces depends upon its heights to width ratio among their factor. It is necessary to differentiate between low rise and high rise frames in this case.

Low rise structures: Height < width

It is characterized predominately by shear deformation. High-rise buildings

Height > width

It is dominated by bending action

## Matrix analysis of frames

The individual elements of frames are oriented in different directions unlike those of continuous beams so their analysis is more complex. Nevertheless the rudimentary flexibility and stiffness methods are applied to frames stiffness method is more useful because its adaptability to computer programming stiffness method is used when degree of redundancy is greater than degree of freedom. However stiffness method is used degree of freedom is greater than degree of redundancy especially for computers..

## III. DESIGN METHODOLOGY

### DESIGN OF SLABS

Depending up on the ratio of longer span to short span ( $l_y/l_x$ ) the slabs are classified in to:

- a. One way slab
- b. Two way slab

#### 1 One way slab

Slabs which are supported on all four edges and the ratio of longer span to the shorter span ( $l_y/l_x$ ) are greater than 2 are called as one way slabs. One way slabs bends in one direction .i.e. along the shorter span and hence span and hence it needs main reinforcement in one direction only (along the shorter span) to resist one way bending However minimum reinforcement known as distribution steel is provided along the longer span above the main reinforcement to distribute the load uniformly and to resist temperature and shrinkage stresses.

#### 2 Two way slab

When the slabs are supported on all the four edges and the ratio of longer span to the shorter span ( $l_y/l_x$ ) is less than or equal to 2, the slabs are likely to bend along the two spans and such slabs are called as two way slabs. The load is transferred in both the direction to the four supporting edges and hence main reinforcement has to be designed in both directions to resist two way bending.

General Design Requirements for slabs as per IS 456:2000

**A. Effective Span:** The effective span of a simply supported slab shall be taken as clear span plus effective depth of the slab or center to center distance between the supports whichever is less.

The effective span of a cantilever slab shall be taken as its length to the face of the support plus half the effective depth except where it forms the end of a continuous slab where the length to the center of support shall be taken.

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**B. Limiting Stiffness:** The stiffness of slabs is governed by the span to depth ratio. As per Clause 23.2 of IS 456 for spans not exceeding 10m, the span to depth ratio (Basic values) should not exceed the limits given below

Cantilever – 7

Simply supported – 20

Continuous – 26

Depending upon the type of steel and percentage of steel, the above values have to be modified as per Fig .4 of IS-456  
For two way slabs, the shorter span be used for calculating the span to effective depth ratio

**C. Minimum Reinforcement:** The reinforcement in either direction of span shall not be less than 0.15% of gross cross sectional area if mild steel is use. However, this value is reduced to 0.12% where high strength deformed bars or welded wire fabrics are used. (Clause 26.5.2.2 of IS -456)

**D. Maximum Reinforcement:** the diameter of the bars shall not exceed one eighth of the total thickness of slab (clause 26.5.2.2 of IS-456)

**E. Spacing of Main Reinforcement:** The spacing of main reinforcement in slabs shall not be more than three times the effective depth of solid slab or 300 mm whichever is less (clause 26.3.3 of IS-456)

### Slab: A1-A2-B1-B2

#### Continuous Two way Slab (5.29X3.96)

Thickness of slab = 170 mm

Live load = 2KN/m<sup>2</sup>

Floor finish = 1 KN/m<sup>2</sup>

Self weight of slab = 25×0.170 = 4.25 KN/m<sup>2</sup>

Total load = 2+1+4.25= 7.25 KN/m

Factored load = 7.25×1.5= 10.875 KN/m

Dimension of the slab  $l_x = 4.11m, l_y = 5.44m$

Ratio of longer span to shorter span ( $l_y/l_x$ ) = 5.44/4.11 = 1.32

Two adjacent edges are discontinuous:

Bending moment coefficients (Clauses D-1.1 and 24.41)

Table:.1 Bending moment coefficients

Type of panel and Moments Considered	Short Span Coefficients: ( $\alpha_x$ )	Long Span Coefficients: ( $\alpha_y$ )
Positive Moment at mid span	0.050	0.035
Negative Moment at continuous edge	0.066	0.047

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Theoretical value of length to depth ratio =  $l_x/d = 4.11 \times 10^3 / 150 = 27.4 < 49.4$   
Hence it is safe.

**Slab: A1-A2-B1-B2(TERRACE)**  
**Continuous Two way Slab (5.29X3.96)**  
**Thickness of slab = 170 mm**

Live load = 1.5KN/m<sup>2</sup>  
Floor finish = 1 KN/m<sup>2</sup>  
Self weight of slab = 25×0.170 = 4.25 KN/m<sup>2</sup>  
Total load = 1.5+1+4.25= 6.75 KN/m  
Factored load = 6.75×1.5= 10.125 KN/m  
Dimension of the slab  $l_x = 4.11m, l_y = 5.44m$   
Ratio of longer span to shorter span ( $l_y/l_x$ ) = 5.44/4.11 = 1.32  
Two adjacent edges are discontinuous:  
Bending moment coefficients (Clauses D-1.1 and 24.41)

Table:.2 Bending moment coefficients

Type of panel and Moments Considered	Short Span Coefficients: ( $\alpha_x$ )	Long Span Coefficients: ( $\alpha_y$ )
Positive Moment at mid span	0.050	0.035
Negative Moment at continuous edge	0.066	0.047

Theoretical value of length to depth ratio =  $l_x/d = 4.11 \times 10^3 / 150 = 27.4 < 52$   
Hence it is safe.

**Slab: A2-A3-B2-B3**  
**Continuous Two way Slab (5.29X4.80)**  
**Thickness of slab = 160 mm**

Live load = 2KN/m<sup>2</sup>  
Floor finish = 1 KN/m<sup>2</sup>  
Self weight of slab = 25×0.160 = 4KN/m<sup>2</sup>  
Total load = 2+1+4= 7 KN/m  
Factored load = 7×1.5 = 10.5 KN/m  
Dimension of the slab  $l_x = 4.94m, l_y = 5.43m$   
Ratio of longer span to shorter span ( $l_y/l_x$ ) = 5.43/4.94 = 1.1  
Two adjacent edges are discontinuous:  
Bending moment coefficients (Clauses D-1.1 and 24.41)

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Table:3 Bending moment coefficients

Type of panel and Moments Considered	Short Span Coefficients: ( $\alpha_x$ )	Long Span Coefficients: ( $\alpha_y$ )
Positive Moment at mid span	0.040	0.035
Negative Moment at continuous edge	0.063	0.047

Theoretical value of length to depth ratio =  $l_x/d = 4.94 \times 10^3 / 140 = 35.28 < 52$

Hence it is safe.

### Slab: A2-A3-B2-B3(TERRACE)

#### Continuous Two way Slab (5.29X4.80)

Thickness of slab = 160 mm

Live load = 1.5 KN/m<sup>2</sup>

Floor finish = 1 KN/m<sup>2</sup>

Self weight of slab =  $25 \times 0.160 = 4 \text{ KN/m}^2$

Total load =  $1.5 + 1 + 4 = 6.5 \text{ KN/m}$

Factored load =  $6.5 \times 1.5 = 9.75 \text{ KN/m}$

Dimension of the slab  $l_x = 4.94 \text{ m}$ ,  $l_y = 5.43 \text{ m}$

Ratio of longer span to shorter span ( $l_y/l_x$ ) =  $5.43/4.94 = 1.1$

Two adjacent edges are discontinuous:

Bending moment coefficients (Clauses D-1.1 and 24.41)

Table:4 Bending moment coefficients

Type of panel and Moments Considered	Short Span Coefficients: ( $\alpha_x$ )	Long Span Coefficients: ( $\alpha_y$ )
Positive Moment at mid span	0.040	0.035
Negative Moment at continuous edge	0.063	0.047

Theoretical value of length to depth ratio =  $l_x/d = 4.94 \times 10^3 / 140 = 35.28 < 52$

Hence it is safe.

### Slab: B1-B2-C1-C2

#### Continuous Two way Slab (4.99X3.96)

Thickness of slab = 170 mm

Live load = 2 KN/m<sup>2</sup>

Floor finish = 1 KN/m<sup>2</sup>

Self weight of slab =  $25 \times 0.170 = 4.25 \text{ KN/m}^2$

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Total load =  $2+1+4= 7.25$  KN/m  
 Factored load =  $7.25 \times 1.5= 10.87$  KN/m  
 Dimension of the slab  $l_x = 4.11\text{m}$  ,  $l_y = 5.14\text{m}$   
 Ratio of longer span to shorter span ( $l_y/l_x$ ) =  $5.14/4.11 = 1.25$   
 One long edge discontinuous:  
 Bending moment coefficients (Clauses D-1.1 and 24.41)

Table:5 Bending moment coefficients

Type of panel and Moments Considered	Short Span Coefficients: ( $\alpha_x$ )	Long Span Coefficients: ( $\alpha_y$ )
Positive Moment at mid span	0.0415	0.028
Negative Moment at continuous edge	0.0545	0.037

Theoretical value of length to depth ratio =  $l_x / d = 4.11 \times 10^3 / 150 = 27.4 < 52$   
 Hence it is safe.

**Slab: B1-B2-C1-C2(TERRACE)**  
**Continuous Two way Slab (4.99X3.96)**  
**Thickness of slab = 170 mm**

Live load =  $1.5\text{KN/m}^2$   
 Floor finish =  $1 \text{ KN/m}^2$   
 Self weight of slab =  $25 \times 0.170 = 4.25\text{KN/m}^2$   
 Total load =  $1.5+1+4= 6.75$  KN/m  
 Factored load =  $6.75 \times 1.5= 10.125$  KN/m  
 Dimension of the slab  $l_x = 4.11\text{m}$  ,  $l_y = 5.14\text{m}$   
 Ratio of longer span to shorter span ( $l_y/l_x$ ) =  $5.14/4.11 = 1.25$   
 One long edge discontinuous:  
 Bending moment coefficients (Clauses D-1.1 and 24.41)

Table:6 Bending moment coefficients

Type of panel and Moments Considered	Short Span Coefficients: ( $\alpha_x$ )	Long Span Coefficients: ( $\alpha_y$ )
Positive Moment at mid span	0.0415	0.028
Negative Moment at continuous edge	0.0545	0.037

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Theoretical value of length to depth ratio =  $l_x / d = 4.11 \times 10^3 / 150 = 27.4 < 52$   
Hence it is safe.

**Slab: B2-B3-C2-C3**

**Continuous Two way Slab (4.99X4.80)**

**Thickness of slab = 200 mm**

Live load = 2KN/m<sup>2</sup>

Floor finish = 1 KN/m<sup>2</sup>

Self weight of slab = 25×0.20 = 5KN/m<sup>2</sup>

Total load = 2+1+5= 8 KN/m

Factored load = 8×1.5= 12 KN/m

Dimension of the slab  $l_x = 4.98\text{m}, l_y = 5.17\text{m}$

Ratio of longer span to shorter span ( $l_y/l_x$ ) = 5.17/4.98 = 1.03

One long edge discontinuous:

Bending moment coefficients (Clauses D-1.1 and 24.41)

Table:7 Bending moment coefficients

Type of panel and Moments Considered	Short Span Coefficients: ( $\alpha_x$ )	Long Span Coefficients: ( $\alpha_y$ )
Positive Moment at mid span	0.0295	0.028
Negative Moment at continuous edge	0.039	0.037

Theoretical value of length to depth ratio =  $l_x / d = 4.98 \times 10^3 / 180 = 27.6 < 52$   
Hence it is safe.

**Slab: B2-B3-C2-C3(Terrace)**

**Continuous Two way Slab (4.99X4.80)**

**Thickness of slab = 200 mm**

Live load = 1.5KN/m<sup>2</sup>

Floor finish = 1 KN/m<sup>2</sup>

Self weight of slab = 25×0.20 = 5KN/m<sup>2</sup>

Total load = 1.5+1+5= 7.5 KN/m

Factored load = 7.5×1.5= 11.25 KN/m

Dimension of the slab  $l_x = 4.98\text{ m}, l_y = 5.17\text{m}$

Ratio of longer span to shorter span ( $l_y/l_x$ ) = 5.17/4.98 = 1.03

One long edge discontinuous:

Bending moment coefficients (Clauses D-1.1 and 24.41)

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Table:8 Bending moment coefficients

Type of panel and Moments Considered	Short Span Coefficients: ( $\alpha_x$ )	Long Span Coefficients: ( $\alpha_y$ )
Positive Moment at mid span	0.0295	0.028
Negative Moment at continuous edge	0.039	0.037

Theoretical value of length to depth ratio =  $l_x/d = 4.98 \times 10^3 / 180 = 27.6 < 52$

Hence it is safe.

### Slab: C1-C2-D1-D2

#### Continuous Two way Slab (3.35X3.96)

Thickness of slab = 120 mm

Live load = 2KN/m<sup>2</sup>  
 Floor finish = 1 KN/m<sup>2</sup>  
 Self weight of slab = 25×0.12 = 3KN/m<sup>2</sup>  
 Total load = 2+1+3= 6 KN/m  
 Factored load = 6×1.5= 9 KN/m  
 Dimension of the slab  $l_x = 3.45$  m,  $l_y = 4.06$  m  
 Ratio of longer span to shorter span ( $l_y/l_x$ ) = 4.06/3.45 = 1.176  
 Two edges are discontinuous:  
 Bending moment coefficients (Clauses D-1.1 and 24.41)

Table:9 Bending moment coefficients

Type of panel and Moments Considered	Short Span Coefficients: ( $\alpha_x$ )	Long Span Coefficients: ( $\alpha_y$ )
Positive Moment at mid span	0.043	0.035
Negative Moment at continuous edge	0.057	0.047

Effective length to depth ratio =  $1.9 \times 32 = 60.8$

Theoretical value of length to depth ratio =  $l_x/d = 3.45 \times 10^3 / 100 = 34.5 < 60.8$

Hence it is safe.

### Slab: C1-C2-D1-D2(Terrace)

#### Continuous Two way Slab (3.35X3.96)

Thickness of slab = 120 mm

Live load = 1.5 KN/m<sup>2</sup>  
 Floor finish = 1 KN/m<sup>2</sup>

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Self weight of slab =  $25 \times 0.12 = 3 \text{ KN/m}^2$   
 Total load =  $1.5 + 1 + 3 = 5.5 \text{ KN/m}$   
 Factored load =  $5.5 \times 1.5 = 8.25 \text{ KN/m}$   
 Dimension of the slab  $l_x = 3.45 \text{ m}, l_y = 4.06 \text{ m}$   
 Ratio of longer span to shorter span ( $l_y/l_x$ ) =  $4.06/3.45 = 1.176$   
 Two edges are discontinuous:  
 Bending moment coefficients (Clauses D-1.1 and 24.41)

Table:10 Bending moment coefficients

Type of panel and Moments Considered	Short Span Coefficients: ( $\alpha_x$ )	Long Span Coefficients: ( $\alpha_y$ )
Positive Moment at mid span	0.043	0.035
Negative Moment at continuous edge	0.057	0.047

Effective length to depth ratio =  $1.9 \times 32 = 60.8$   
 Theoretical value of length to depth ratio =  $l_x/d = 3.45 \times 10^3/100 = 34.5 < 60.8$   
 Hence it is safe.

**Slab: C2-C3-D2-D3**

**Continuous Two way Slab (3.35X4.80)**

**Thickness of slab = 120 mm**

Live load =  $2 \text{ KN/m}^2$   
 Floor finish =  $1 \text{ KN/m}^2$   
 Self weight of slab =  $25 \times 0.12 = 3 \text{ KN/m}^2$   
 Total load =  $2 + 1 + 3 = 6 \text{ KN/m}$   
 Factored load =  $6 \times 1.5 = 9 \text{ KN/m}$   
 Dimension of the slab  $l_x = 3.45 \text{ m}, l_y = 4.90 \text{ m}$   
 Ratio of longer span to shorter span ( $l_y/l_x$ ) =  $4.90/3.45 = 1.420$   
 Two edges are discontinuous:  
 Bending moment coefficients (Clauses D-1.1 and 24.41)

Table:11 Bending moment coefficients

Type of panel and Moments Considered	Short Span Coefficients: ( $\alpha_x$ )	Long Span Coefficients: ( $\alpha_y$ )
Positive Moment at mid span	0.053	0.035
Negative Moment at continuous edge	0.071	0.047

Effective length to depth ratio =  $2 \times 32 = 64$   
 Theoretical value of length to depth ratio =  $l_x/d = 3.45 \times 10^3/100 = 34.5 < 64$

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Hence it is safe.

**Slab: C2-C3-D2-D3(Terrace)**

**Continuous Two way Slab (3.35X4.80)**

**Thickness of slab = 120 mm**

Live load = 1.5KN/m<sup>2</sup>

Floor finish = 1 KN/m<sup>2</sup>

Self weight of slab = 25×0.12 = 3KN/m<sup>2</sup>

Total load = 1.5+1+3= 5.5 KN/m

Factored load = 5.5×1.5= 8.25 KN/m

Dimension of the slab  $l_x = 3.45 \text{ m}, l_y = 4.90 \text{ m}$

Ratio of longer span to shorter span ( $l_y/l_x$ ) = 4.90/3.45 = 1.420

Two edges are discontinuous:

Bending moment coefficients (Clauses D-1.1 and 24.41)

Table:12 Bending moment coefficients

Type of panel and Moments Considered	Short Span Coefficients: ( $\alpha_x$ )	Long Span Coefficients: ( $\alpha_y$ )
Positive Moment at mid span	0.053	0.035
Negative Moment at continuous edge	0.071	0.047

Effective length to depth ratio =  $2 \times 32 = 64$

Theoretical value of length to depth ratio =  $l_x / d = 3.45 \times 103 / 100 = 34.5 < 64$

Hence it is safe.

## DESIGN OF BEAMS

### (1) Continuous Beam (AK, DR)

$$M_{AB} = -wl^2/12 = -33.14 \times 5.52^2/12 = -84.14 \text{ kN-m}$$

$$M_{BA} = wl^2/12 = 33.14 \times 5.52^2/12 = 84.14 \text{ kN-m}$$

$$M_{BC} = -wl^2/12 = -34.059 \times 5.22^2/12 = -77.33 \text{ kN-m}$$

$$M_{CB} = wl^2/12 = 34.059 \times 5.22^2/12 = 77.33 \text{ kN-m}$$

$$M_{CD} = -wl^2/12 = -35.31 \times 3.58^2/12 = -37.71 \text{ kN-m}$$

$$M_{DC} = wl^2/12 = 35.31 \times 3.58^2/12 = 37.7 \text{ kN-m}$$

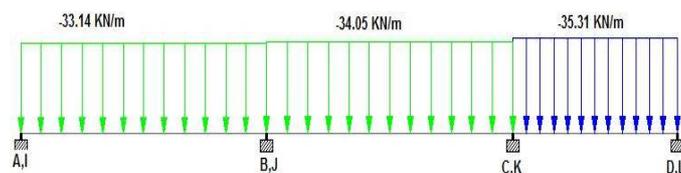


Fig : Loading diagram for floor

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Table 14: Distribution factors

Joints	Member	Relative stiffness	Total	Distribution factor
<b>B</b>	BA	I/5.52	$I/5.52 + I/5.22 = 0.37 I$	$1/0.5I \times I/4.7 = 0.48$
	BC	I/5.22		$1/0.5I \times I/3.45 = 0.51$
<b>C</b>	CB	I/5.22	$I/5.22 + I/3.58 = 0.47I$	$1/0.47I \times I/5.22 = 0.40$
	CD	I/3.58		$1/0.47I \times I/3.58 = 0.59$

### DESIGN OF COLUMN

Table 15: Column reactions

Grid / column	Load from terrace(A)KN	Load from Typical floor(B) KN	Weight of the column (C)KN	Total column reaction (A+B+C)KN
A1	115.09	237.78x3	$(0.23 \times 0.4 \times 13 \times 20) = 23.92$	852.35
A2	186.16	344.08x3	$(0.23 \times 0.4 \times 13 \times 20) = 23.92$	1242.32
A3	99.105	198.99x3	$(0.23 \times 0.4 \times 13 \times 20) = 23.92$	719.99
B1	199.35	409.28x3	$(0.23 \times 0.5 \times 13 \times 20) = 29.90$	1457.09
B2	317.05	630.13x3	$(0.23 \times 0.5 \times 13 \times 20) = 29.90$	2237.34
B3	172.3	349.18x3	$(0.23 \times 0.4 \times 13 \times 20) = 23.92$	1243.76
C1	132.9	420.45x3	$(0.23 \times 0.5 \times 13 \times 20) = 29.90$	1424.15
C2	297.9	653.79x3	$(0.23 \times 0.5 \times 13 \times 20) = 29.90$	2289.17
C3	290.64	300.99x3	$(0.23 \times 0.4 \times 13 \times 20) = 23.92$	1217.53
D1	115.09	273.16x3	$(0.23 \times 0.4 \times 13 \times 20) = 23.92$	958.49

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D2	263.67	287.40x3	(0.23x0.5x13x20) =29.90	1155.77
D3	86.32	185.00x3	(0.23x0.4x13x20) =23.92	665.24

## DESIGN CALCULATIONS

### 1. C1=900 KN

Size of the column = 230x400mm

Concrete mix = M 20

Characteristic strength of reinforcement = 415 N/mm<sup>2</sup>

$e_{min,x} = 3000/500 + 230/30 = 13.66$  mm

$e_{min,y} = 3000/500 + 400/30 = 19.33$  mm

$M_{ux} = P_u \cdot e_x$  KN-m = 900x13.66=12.30 KN-m

$M_{uy} = P_u \cdot e_y$  KN-m = 900x19.33=17.39 KN-m.

Assume the reinforcement Percentage = 1.5

$P/f_{ck} = 0.075$

Uniaxial moment capacity of the section about x-x axis :

$(d' + dia)/D = (40 + 12)/230 = 0.22$

$P_u / (f_{ck} b d) = 900 \times 10^3 / (20 \times 230 \times 400) = 0.489$

Referring chart 46

$M_u / (f_{ck} b D^2) = 0.06$

$M_{ux1} = 0.06 \times 20 \times 230 \times 400^2 = 44.16$  KN-m

Uniaxial moment capacity of the section about y-y axis :

$(d' + dia)/D = (40 + 12)/400 = 0.13$

$P_u / (f_{ck} b d) = 900 \times 10^3 / (20 \times 230 \times 400) = 0.489$

Referring chart 45

$M_u / (f_{ck} b D^2) = 0.07$

$M_{uy1} = 0.07 \times 20 \times 230 \times 400^2 = 51.52$  KN-m

Calculating of  $P_{uz}$  ;

$P = 1.5$  ,  $f_y = 415$  N/mm<sup>2</sup> and  $f_{ck} = 20$  N/mm<sup>2</sup>

Referring chart 63 corresponding above values

$P_{uz} / A_g = 13$  N/mm<sup>2</sup>

$P_{uz} = 13 \times 230 \times 400 = 1196$  KN

$P_u / P_{uz} = 0.752$

$M_{ux} / M_{ux1} = 0.278$

$M_{uy} / M_{uy1} = 0.33$

Referring to chart 64 the permissible value of  $M_{ux} / M_{ux1}$  Corresponding to the above values  $P_u / P_{uz}$  and  $M_{uy} / M_{uy1}$  is equal to 0.93

$0.93 > 0.27$

Hence it is safe.

$A_{st} = 230 \times 400 \times 1.5 / 100 = 1380$  mm<sup>2</sup>

Using 16 mm dia bars

No of bars =  $1380 / 200.96 = 6.86 = 7$  bars

Provide 7, 16mm dia bars.

Provide lateral ties of 6 mm dia 230 mm c/c

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## 2.C2=1300 KN

Procedure same as above, Provide 2.5 % of steel it is safe

Ast required =  $230 \times 400 \times 2.5 / 100 = 2300 \text{ mm}^2$

Provide 18 mm dia bars 10 in number

Provide Lateral ties 6 mm @ 280 mm c/c

## 3.C3=2400 KN

Procedure same as above, Provide 3 % of steel it is safe

Ast required =  $230 \times 500 \times 3 / 100 = 3450 \text{ mm}^2$

Provide 20mm dia bars 10 in number

Provide Lateral ties of 6 mm dia 230 mm c/c

## DESIGN OF FOOTING

### DESIGN CALCULATIONS

#### F<sub>1</sub> = 900KN

Factored load	= 900KN
Size of column	= 230mm × 400mm
S.B.C of Soil	= 350 KN/m <sup>2</sup>
Un-factored	= 900/1.5 = 600 KN

#### 1. Size of the footing:-

Load from the column	= 600 KN
Self weight of footing	= 10% of the column load = 600/10 = 60
Total load of footing	= 600 + 60 = 660 KN
Area of the footing	= Total load/SBC of soil
	= 660/350 = 1.8m <sup>2</sup>

Size of the square footing

$$B = \sqrt{1.8} = 1.37$$

Adopt 1.4m × 1.4m square footing.

#### 2. Upward soil pressure:-

Factor load = P <sub>u</sub>	= 900KN
Soil pressure at ultimate load	
q <sub>u</sub> = P <sub>u</sub> /area of footing	
q <sub>u</sub> = 900/(1.4 × 1.4)	= 459.18 KN/m <sup>2</sup>
q <sub>u</sub> = 0.459 N/mm <sup>2</sup>	

#### 3. Depth of Footing from Bending Moment consideration:

The critical section for B.M will be at the face of the column as shown in fig below

$$M_u = q_u \left[ \frac{B(B-b)^2}{8} \right] = 0.459 \left[ \frac{1400(1400-230)^2}{8} \right]$$

$$M_u = 109.95 \times 10^6 \text{ N-mm}$$

$$M_u = 0.138 f_{ck} B d^2$$

$$109.95 \times 10^6 = 0.138 \times 20 \times 1400 \times d^2$$

$$d = 168.68 \text{ mm}$$

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Depth required to resist shear in footings is much higher than required for bending  
Let us assume the effective depth as twice the depth required from bending consideration.

Provide effective depth =  $d = 400\text{mm}$   
Overall depth  $D = 450\text{mm}$

#### 4. Reinforcement:-

$$M_u = 0.87 f_y A_{st} d \left[ 1 - \frac{f_y A_{st}}{f_{ck} B d} \right]$$

$$109.95 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400 \left[ 1 - \frac{415 \times A_{st}}{20 \times 1400 \times 400} \right]$$

$$109.95 \times 10^6 = 144420 \times A_{st} - 5.34 A_{st}^2$$

Area of steel required,  $A_{st} = 784.05 \text{ mm}^2$

Use 12mm diameter bars

$$a_{st} = \frac{\pi}{4} d^2 = 113.04 \text{ mm}^2$$

$$\text{Spacing: } S = \frac{a_{st} \times B}{A_{st}} = \frac{113.04 \times 1400}{784.05} = 201.84 \text{ mm}$$

Provide spacing =  $S = 200 \text{ mm}$

Provide Area of steel

$$A_{st} = \frac{a_{st} \times B}{S} = \frac{113.04 \times 1400}{200}$$

$$A_{st} = 791.28 \text{ mm}^2$$

#### 5. Check for one way shear:-

The critical section for one way shear is at a distance “d” from the face of the column as shown in the fig

Factored shear force

$V_u =$  Soil presser from the shaded area

$$V_u = q_u B \left[ \frac{(B-b)}{2} - d \right]$$

$$= 0.459 \times 1400 \left[ \frac{(1400-230)}{2} - 400 \right]$$

$$V_u = 118.8 \times 10^3 \text{ N}$$

Shear stress

$$\tau_v = \frac{V_u}{Bd} = \frac{118800}{1400 \times 400}$$

$$\tau_v = 0.21 \text{ N/mm}^2$$

Percentage of steel

$$P_t = \frac{A_{st} \times 100}{Bd} = \frac{100 \times 791.28}{1400 \times 400}$$

$$P_t = 0.14 \%$$

From table 19, IS 456:2000

$$\tau_c > \tau_v$$

So it is safe in one way shear

#### 6. Check for two way shear :-

The critical section for two way shear is at a distance  $\frac{d}{2}$  from the face of the column as shown in the fig

$$\text{Perimeter of the critical section (p)} = 2[(230+300) + (400+300)]$$

$$= 2460 \text{ mm}$$

$$\text{Area of critical section (A)} = p \times d = 2460 \times 400 = 768000.00 \text{ mm}^2$$

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Two way shear  $V_{u2} = q_u \times \text{area of shaded portion}$   
 $V_{u2} = 0.459[(1400 \times 1400) - (630 \times 800)] = 668.30 \text{ KN}$

Two way shear stress  $\tau_{v2} = \frac{V_{u2}}{A} = \frac{668304}{984000}$

$\tau_{v2} = 0.679 \text{ N/mm}^2$

Permissible punching shear

$\tau_p = 0.25\sqrt{f_{ck}} = 0.25\sqrt{20}$

$\tau_p = 1.11 \text{ N/mm}^2$

$\tau_{v2} < \tau_p$

Hence, it is safe with respect to two way shear.

**F<sub>2</sub>=1300KN**

Factored load	=1300KN
Size of column	=230mm×400mm
S.B.C of Soil	= 350 KN/m <sup>2</sup>
Un-factored	=1300/1.5=866.67 KN

## 1. Size of the footing:-

Load from the column	= 866.67 KN
Self weight of footing	=10% of the column load =866.67/10 = 86.667
Total load of footing	=866.67+86.667 = 953.33 KN
Area of the footing	= Total load/SBC of soil = 953.33/350 = 2.72m <sup>2</sup>

Size of the square footing  
 $B = \sqrt{2.72} = 1.65$

Adopt 1.7m×1.7m square footing.

## 2. Upward soil pressure:-

Factor load = P <sub>u</sub>	= 1300KN
Soil pressure at ultimate load	
$q_u = P_u/\text{area of footing}$	
$q_u = 1300/(1.7 \times 1.7)$	= 449.8 KN/m <sup>2</sup>
$q_u = 0.449 \text{ N/mm}^2$	

## 3. Depth of Footing from Bending Moment consideration:

The critical section for B.M will be at the face of the column as shown in fig below

$M_u = q_u \left[ \frac{B(B-b)^2}{8} \right] = 0.449 \left[ \frac{1700(1700-230)^2}{8} \right]$

$M_u = 206.17 \times 10^6 \text{ N-mm}$

$M_u = 0.138 f_{ck} B d^2$

$206.17 \times 10^6 = 0.138 \times 20 \times 1700 \times d^2$

$d = 209.6 \text{ mm}$

Depth required to resist shear in footings is much higher than required for bending

Let us assume the effective depth as twice the depth required from bending consideration.

Provide effective depth = d = 450mm

Overall depth D = 500mm

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#### 4. Reinforcement:-

$$M_u = 0.87 f_y A_{st} d \left[ 1 - \frac{f_y A_{st}}{f_{ck} B d} \right]$$

$$206.17 \times 10^6 = 0.87 \times 415 \times A_{st} \times 450 \left[ 1 - \frac{415 \times A_{st}}{20 \times 1700 \times 450} \right]$$

Area of steel required,  $A_{st} = 1315 \text{ mm}^2$

Use 12mm diameter bars

$$a_{st} = \frac{\pi}{4} d^2 = 113.04 \text{ mm}^2$$

$$\text{Spacing } S = \frac{a_{st} \times B}{A_{st}} = \frac{113.04 \times 1700}{1315} = 146.13 \text{ mm}$$

Provide spacing =  $S = 140 \text{ mm}$

Provide Area of steel

$$A_{st} = \frac{a_{st} \times B}{S} = \frac{113.04 \times 1700}{140}$$

$$A_{st} = 1372.6 \text{ mm}^2$$

#### 5. Check for one way shear:-

The critical section for one way shear is at a distance “d” from the face of the column as shown in the fig

Factored shear force

$V_u$  = Soil presser from the shaded area

$$V_u = q_u B \left[ \frac{(B-b)}{2} - d \right]$$

$$= 0.449 \times 1700 \left[ \frac{(1700-230)}{2} - 450 \right]$$

$$V_u = 217.92 \times 10^3 \text{ N}$$

Shear stress

$$\tau_v = \frac{V_u}{Bd} = \frac{217920}{1700 \times 450}$$

$$\tau_v = 0.284 \text{ N/mm}^2$$

Percentage of steel

$$P_t = \frac{A_{st} \times 100}{Bd} = \frac{100 \times 1372.6}{1700 \times 450}$$

$$P_t = 0.17 \%$$

From table 19, IS 456:2000

$$\tau_c > \tau_v$$

So it is safe in one way shear

#### 6. Check for two way shear :-

The critical section for two way shear is at a distance  $\frac{d}{2}$  from the face of the column as shown in the fig

Perimeter of the critical section (p) =  $2[(230+450)+(400+450)] = 3060 \text{ mm}$

Area of critical section (A) =  $p \times d = 3060 \times 450 = 1377000.00 \text{ mm}^2$

Two way shear  $V_{u2} = q_u \times \text{area of shaded portion}$

$$V_{u2} = 0.449 [(1700 \times 1700) - (680 \times 850)]$$

$$= 1039.93 \text{ KN}$$

$$\text{Two way shear stress } \tau_{v2} = \frac{V_{u2}}{A} = \frac{1039930}{1377000}$$

$$\tau_{v2} = 0.75 \text{ N/mm}^2$$

Permissible punching shear

$$\tau_p = 0.25 \sqrt{f_{ck}} = 0.25 \sqrt{20}$$

$$\tau_p = 1.11 \text{ N/mm}^2$$

$$\tau_{v2} < \tau_p$$

Hence, it is safe with respect to two way shear.

**$F_3 = 2400 \text{ KN}$**

Factored load = 2400KN

Size of column = 230mm × 500mm

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S.B.C of Soil = 350 KN/m<sup>2</sup>  
Un-factored = 2400/1.5 = 1600 KN

## 1. Size of the footing:-

Load from the column = 1600 KN  
Self weight of footing = 10% of the column load = 1600/10 = 160  
Total load of footing = 1600 + 160 = 1760 KN  
Area of the footing = Total load/SBC of soil  
= 1760/350 = 5.02m<sup>2</sup>

Size of the square footing  
 $B = \sqrt{5.02} = 2.24$

Adopt 2.25m × 2.25m square footing.

## 2. Upward soil pressure:-

Factor load =  $P_u = 2400$ KN  
Soil pressure at ultimate load  
 $q_u = P_u / \text{area of footing}$   
 $q_u = 2400 / (2.25 \times 2.25) = 474.07$  KN/m<sup>2</sup>  
 $q_u = 0.474$  N/mm<sup>2</sup>

## 3. Depth of Footing from Bending Moment consideration:

The critical section for B.M will be at the face of the column as shown in fig below

$$M_u = q_u \left[ \frac{B(B-b)^2}{8} \right] = 0.474 \left[ \frac{2250(2250-230)^2}{8} \right]$$

$$M_u = 543.96 \times 10^6 \text{ N-mm}$$

$$M_u = 0.138 f_{ck} B d^2$$

$$543.96 \times 10^6 = 0.138 \times 20 \times 22500 \times d^2$$

$$d = 295.96 \text{ mm}$$

Depth required to resist shear in footings is much higher than required for bending

Let us assume the effective depth as twice the depth required from bending consideration.

Provide effective depth =  $d = 600$ mm

Overall depth  $D = 650$ mm

## 4. Reinforcement:-

$$M_u = 0.87 f_y A_{st} d \left[ 1 - \frac{f_y A_{st}}{f_{ck} B d} \right]$$

$$543.96 \times 10^6 = 0.87 \times 415 \times A_{st} \times 600 \left[ 1 - \frac{415 \times A_{st}}{20 \times 2250 \times 600} \right]$$

Area of steel required,  $A_{st} = 2615$  mm<sup>2</sup>

Use 12mm diameter bars

$$a_{st} = \frac{\pi}{4} d^2 = 113.04 \text{ mm}^2$$

$$\text{Spacing } S = \frac{a_{st} \times B}{A_{st}} = \frac{113.04 \times 2250}{2615} = 97.26 \text{ mm}$$

Provide spacing =  $S = 90$  mm

Provide Area of steel

$$A_{st} = \frac{a_{st} \times B}{S} = \frac{113.04 \times 2250}{90}$$

$$A_{st} = 2826 \text{ mm}^2$$

## 5. Check for one way shear:-

The critical section for one way shear is at a distance “d” from the face of the column as shown in the fig

Factored shear force

$V_u$  = Soil presser from the shaded area

$$V_u = q_u B \left[ \frac{(B-b)}{2} - d \right]$$

$$= 0.47 \times 2250 \left[ \frac{(2250-230)}{2} - 600 \right]$$

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Shear stress  $V_u = 437.26 \times 10^3 \text{ N}$

$$\tau_v = \frac{V_u}{Bd} = \frac{437260}{2250 \times 600}$$

$$\tau_v = 0.32 \text{ N/mm}^2$$

Percentage of steel

$$P_t = \frac{A_{st} \times 100}{Bd} = \frac{100 \times 2826}{2250 \times 600}$$

$$P_t = 0.20 \%$$

From table 19, IS 456:2000

$$\tau_c > \tau_v$$

So it is safe in one way shear

### 6. Check for two way shear :-

The critical section for two way shear is at a distance  $\frac{d}{2}$  from the face of the column as shown in the fig

Perimeter of the critical section (p) =  $2[(230+600) + (500+600)] = 3860 \text{ mm}$

Area of critical section (A) =  $p \times d = 3860 \times 600 = 2316000.00 \text{ mm}^2$

Two way shear  $V_{u2} = q_u \times \text{area of shaded portion}$   
 $V_{u2} = 0.474[(2250 \times 2250) - (830 \times 1100)]$   
 $= 1966.86 \text{ KN}$

Two way shear stress  $\tau_{v2} = \frac{V_{u2}}{A} = \frac{1966860}{2316000}$

$\tau_{v2} = 0.849 \text{ N/mm}^2$

Permissible punching shear

$$\tau_p = 0.25 \sqrt{f_{ck}} = 0.25 \sqrt{20}$$

$\tau_p = 1.11 \text{ N/mm}^2$

$$\tau_{v2} < \tau_p$$

Hence, it is safe with respect to two way shear.

## DESIGN OF STAIRCASE

### 1. DESIGN FACTORS:

Type of staircase = dog logged staircase.

Rise = 150mm

Tread = 300mm

Width of landing = 300mm

$F_{ck} = 20 \text{ N/mm}^2$

$F_y = 415 \text{ N/mm}^2$

To find total number of steps = (height of floor/rise)

$$= (3/0.15)$$

$$= 20 \text{ nos}$$

Number of steps in each flight = 10 nos

### 2. EFFECTIVE SPAN:

Effective span =  $(9 \times 300) + 300 = 3000 \text{ mm} = 3.0 \text{ m}$

### 3. LOAD CALCULATION:

Thickness of waist slab =  $(3000/20) = 150 \text{ mm}$

Effective depth  $d = 130 \text{ mm}$

Dead loads of slab on slope  $w_s$

$$W_s = (0.150 \times 1 \times 25) = 3.75 \text{ kN/m}$$

Dead loads of slab on horizontal span "w"

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$$W = W_s \sqrt{(R^2 + T^2)} / T$$

$$W = 3.75 \times \sqrt{(150^2 + 300^2)} / 300$$

$$W = 4.91 \text{ kN/m}$$

$$\text{Dead loads of one step} = (0.5 \times 0.3 \times 0.15 \times 25) = 0.56 \text{ kN/m}$$

$$\text{Finish} = 1 \text{ kN/m}$$

$$\text{Self weight of landing Portion} (0.3 \times 1 \times 25) = 7.5 \text{ kN/m}$$

$$\text{Total dead load} = (4.19 + 0.56 + 7.5) = 13.25 \text{ kN/m}$$

$$\text{Live load} = 3 \text{ kN/m}$$

$$\text{Total service load} = 16.25 \text{ kN/m}$$

$$\text{Factor load} = 24.375 \text{ kN/m}$$

Max B.M

$$M_u = 0.125 \mu l^2 = 0.125 \times 24.375 \times 3^2 = 27.42 \text{ kN/m}$$

### CHECK FOR DEPTH OF WAIST SLAB:

$$d = \sqrt{\mu / 0.138 f_{ck} b}$$

$$d = \sqrt{27.42 \times 10^6 / 0.138 \times 20 \times 1000}$$

$$= 99.6 \text{ mm} < \text{provide depth (130 mm)}$$

Hence safe

### MAIN REINFORCEMENT:

$$M_u = (0.87 \times f_y A_{st} d) \{1 - (A_{st} f_y / b d f_{ck})\}$$

$$(27.42 \times 10^6) = (0.87 \times 415 \times A_{st} \times 130) \{1 - (A_{st} \times 415 / 1000 \times 130 \times 20)\}$$

$$7.49 A_{st}^2 - 46936.15 A_{st} + 27.42 \times 10^6 = 0$$

$$A_{st} = 652.03 \text{ mm}^2$$

Provide 16mm dia bars

$$\text{Spacing} = a_{st} \times 1000 / A_{st} = 200.96 \times 1000 / 652.03$$

$$= 308.20 \text{ mm say } 300 \text{ mm c/c}$$

### DISTRIBUTION REINFORCEMENT:

Distribution reinforcement = 0.12% of c/s

$$= (0.12 / 100) \times (1000 \times 150)$$

$$= 180 \text{ mm}^2$$

Provide 8mm dia bars

$$\text{Spacing} = 50.26 \times 1000 / 180$$

$$= 279.20 \text{ mm say } 250 \text{ mm c/c}$$

Provide 8mm dia of bars @ 250mm c/c

## IV. CONCLUSION

Construction of Residential building should be providing safety and it should be economical. This project describes detailed planning and designing of the 3 storied residential building apartments. This project involves design of structures with conventional method. The analysis and design of the entire structure has been completed. The results include the various forces acting on various members as well various schedules for various members. The foundation has been designed as an isolated footing using soil condition as medium.

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