ANALYSIS AND DESIGN OF REINFORCED CONCRETE FLAT SLAB USING VARIOUS CODES

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ANALYSIS AND DESIGN OF REINFORCED CONCRETE FLAT SLAB USING VARIOUS CODES

Major Project

Submitted in partial fulfillment of the requirements

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By

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Guide

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May 2012

Declaration

This is to certify that

i) The thesis comprises my original work towards the degree of Master of Technology in Information and Communication Technology at Nirma University and has not been submitted elsewhere for a degree.

ii) Due acknowledgement has been made in the text to all other material used.

Patel Jecky

Certificate

This is to certify that the Major Project entitled " ANALYSIS AND DESIGN OF REINFORCED CONCRETE FLAT SLAB USING VARIOUS CODES " submitted by Patel Jecky (10MCLC09), towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis And Design) of Nirma University of Science and Technology, Ahmedabad is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.

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Abstract

Beam less floor directly resting on support (column and/ wall) is known as flat slab. Because of this, large bending moment and shear force is developed in the slab, near the column. These stresses produce concrete cracking and may lead to the slab failure, hence there is a need to increase the area at top of column to withstand the stresses. This enlarged truncated portion at the top of column is known as column head/ column capital.

Flat slabs are subjected to gravity and lateral loads. Gravity load analysis of flat slab is carried out by Direct Design Method (DDM) and Equivalent Frame Method (EFM) as prescribed by different standards, however finite element analysis and equivalent frame method of flat slab is carried out for gravity loads using software SAFE (Slab Analysis by Finite Element Method and Equivalent Frame Method).

IS 456-2000, ACI 318-08, BS 8110-1997 & EC2-Part1-2004 prescribed the coefficients for analysis of flat slab as per DDM and EFM. Analysis and design of flat slab are carried out to compare the coefficients prescribed by different standards and by Equivalent frame method using software SAFE for distribution of moments along longitudinal and transverse directions. Slab is divided into column strip and middle strip. IS 456-2000, ACI 318-08, BS 8110-1997 & EC2-Part1-2004 specify the fixed value of column strip and middle strip irrespective of interior span and midspan. The present study also incorporates the comparison of distribution of width of column strip and middle strip as per IS 456, ACI 318, BS 8110-1997 & EC2-Part1-2004 and as per Equivalent frame method for the staggered columns and the without the staggered column. And a comparison is made.

Excel worksheet is also prepared for analysis and design of flat slab(with staggered and without staggered columns) as per EFM. EFM includes analysis as per IS 456-2000,

ACI-318-08, BS 8110-1997 & EC2-Part1-2004 and using the distribution coefficients along longitudinal and transverse directions.

Abbreviation, Notation and Nomenclature

l_x .... Span length in X direction

l_y .... Span length in Y direction

M_0 .... Total moment

ln .... Clear span in the direction of moments

w .... Total factored load

w_d .... Factored dead load

w_l .... Factored live load

D .... Overall depth of beam or slab

α_c .... Ratio of flexural stiffness of exterior columns to flexural Stiffness of slab

K_C .... Flexural stiffness of original column

K_S .... Flexural stiffness of slab

K_{ec} .... Flexural stiffness of equivalent column

K_t .... Torsional stiffness of transverse slab/ beam

L_1 Span in the direction in which moments are determined From center to center of support

L_2 .... Span transverse to L_1

l_n .... Length of clear span in the direction of M, measured face to Face of support

M .... Negative moment at the left end

M_{ul}^- .... Negative moment at the left end

M^+ .... Positive moment

M_{ur}^- .... Negative moment at the right end

f_{ck} .... Characteristic cube compressive strength of concrete

f_y .... Characteristic strength of steel

E_b .... Modulus of elasticity of beam

E_c .... Modulus of elasticity of column

I_b .... Moment of inertia of the effective beam

I_s .... Moment of inertia of the effective slab d .... Effective depth of the slab

C .... Torsional constant of the transverse torsional members

β .... Ratio of clear span in longer direction to shorter direction

β_c .... Ratio of longer side to shorter side of column capital

τ_c .... Shear stress in concrete

E_C .... Modulus of elasticity of concrete

C_1 .... Size of support in direction parallel to lateral load

C_2 .... Size of support in direction transverse to support

K_{ec} .... Flexural stiffness of equivalent column

K_C .... Flexural stiffness of actual column

K_t .... Torsional stiffness of actual column

S .... Software

M .... Manual

FR .... Frame

E.Q FR .... Equivalent frame

C.S .... Column strip

M.S .... Middle strip

L. D .... Longitudinal distribution

T. D .... Transverse Distribution

x .... Location of maximum ‘positive’ moment

K_{NF} .... Stiffness factor

C_{NF} .... Carry-over factor

m_{NF} .... FEM coeffcient

α_v .... Distance from the edge of the loaded area to the perimeter considered

A_{sv} .... Area of shear reinforcement

b_e .... Breadth of effective moment transfer strip

γ_m .... Partial safety factor for strength of materials

h_c .... Effective diameter of a column or column head

l .... Full panel length in the direction of span

M_t .... Design moment transferred between slab and column

υ_c .... Design concrete shear stress

V .... Design ultimate value of the concentrated load

V_t .... Design shear transferred to column

V_{eff} .... Design effective shear including allowance for moment transfer

As,prov .... Area of tension steel provided

As,req'd .... Area of tension steel required

δ .... Ratio of the redistributed moment to the elastic bending moment

f_{cd} .... Design value of concrete compressive strength

l/d .... Limiting span-to-depth ratio

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Chapter 1

Introduction

1.1 General

A flat slab consists of a reinforced concrete slab that is directly supported by concrete columns without the use of intermediate beams.

Claud A. P. Turner[1] was one of the early advocates of flat slab system known as " mushroom" system. C.A.P. Turner constructed flat slabs in U.S.A. in 1906 mainly using intuitive and conceptual ideas, which was start of this type of construction. Many slabs were load-tested between 1910- 20 in U.S.A. It was only in 1914 that Nicholas[2] proposed a method of analysis of flat slabs based on simple statics. This method is used even today for the design of flat slabs and flat plates and is known as the direct design method.

Structural engineers commonly use the equivalent frame method with equivalent beams such as the one proposed by Jacob S. Grossman [3] in practical engineering for the analysis of flat plate structures.

Floor systems consisting of flat slabs are very popular in countries where cast-in-place construction is predominant form of construction because of many advantages in terms of architectural flexibility, use of space, easier formwork, and shorter construction time.Flat slabs are being used mainly in office buildings due to reduced formwork cost, fast excavation, and easy installation.

- **Necessity of Flat Slabs**

  Architectural demand for better illumination, lesser fire resistance of sharp corners present in the form of beams, increase in the formwork cost, optimum use of space leads to the new concept in the field of structural engineering as Reinforced concrete flat slabs.

- **Components of Flat Slab**

  Flat slab means a reinforced concrete slab with or without drop, supported generally without beams by column with or without flared column heads. Main components of flat slab are.

  a. Panel.

  b. Drop.

  c. Column head.

  d. Column strip.

  e. Middle strip.

  Panel

  Panel means that Part of the slab bounded on each of its four sides by the center line of a column or center line of adjacent spans.

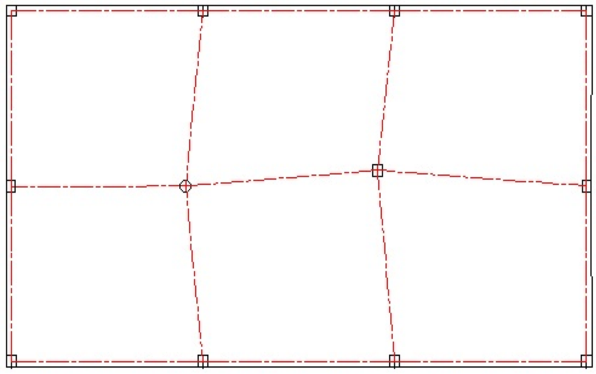


Figure 1.1: Components of Flat slab (panel)

Drop

Moments in the slabs are more near the column hence the slab is thickened near the columns by providing the drop. Drops when provided shall be rectangular in plan and have a length in each direction not less than one third of the panel length in that direction. For exterior panels, width of drops at right angles to the non continuous edge and measured from center line of columns shall be equal to one half the width of drop for interior panels.

Column Head

The column head is widened so as to reduce the punching shear in the slab. The widened portions are called column heads. The column heads may be provided with any angle from the consideration of architecture but for the design, concrete in the portion at 45^0 on either side of vertical only is considered as effective for the design.

Column Strip

Bands of slab in both directions along column lines are considered to act as beams, which are known as strips and the strip which pass through the column is known as Column strip. Width of the column strip is 0.25 L_2, but not greater than 0.25 L_1 on each side of a column center line where L_1 is the span in the direction in which moments are determined and L_2 is the span transverse to L_1, measured center to center of supports.

Middle Strip

A design strip bounded on each of its opposite sides by the column strip. Middle strip behaves as a continuous beam supported on column strip.

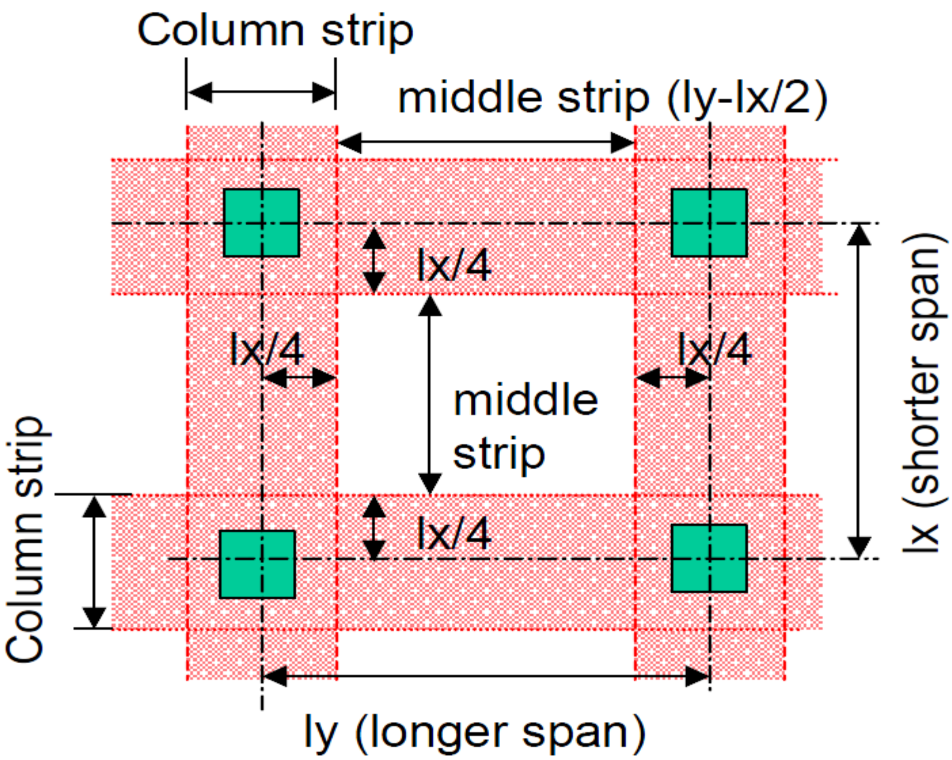


Figure 1.2: Components of Flat slab (column strip, middle strip)[23]

- **Types of Flat Slabs**

  Slabs supported directly on the column with or without beams are known as flat slabs. In such slabs large bending moments and shears develop near the junctions with columns. Therefore there is a need to spread the column at its top end or thicken the slab over column. Flat slab without drop and column head/ column capital is known as flat plate. The flat slabs are classified as

a. Flat slab without Drop and column head figure 1.3.

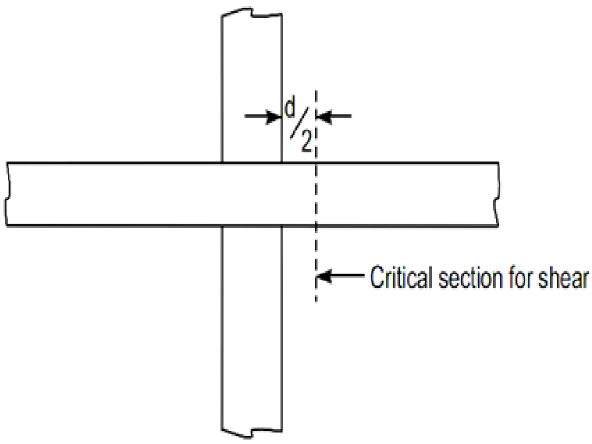


Figure 1.3: Flat slab without Drop and column head

b. Flat slab without drop and column with column head 1.4.

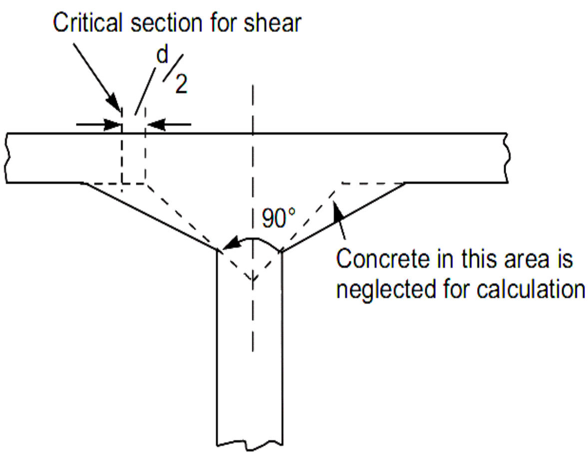


Figure 1.4: Flat slab without Drop and column head

c. Flat slab with drop and column without column head 1.5.

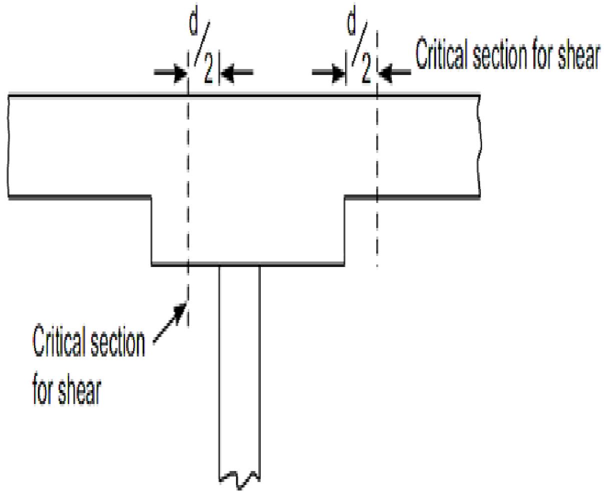


Figure 1.5: Flat slab with drop and column without column head

d. Flat slab with drop and column head 1.6.

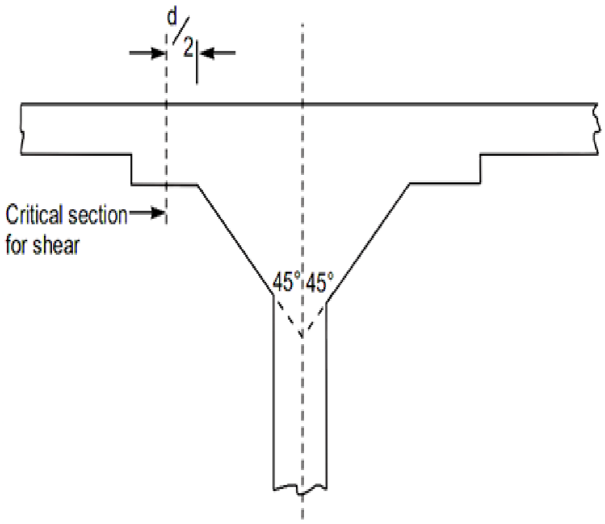


Figure 1.6: Flat slab with drop and column head

- **Behavior of Flat Slab**

  Behavior of flat slab and flat plates are identical to those of two way slab. Bands of slab in both directions along column lines are considered to act as beams.Such bands of slabs are referred as column strips which pass through the columns and middle strips, occur in the middle of two adjacent columns.

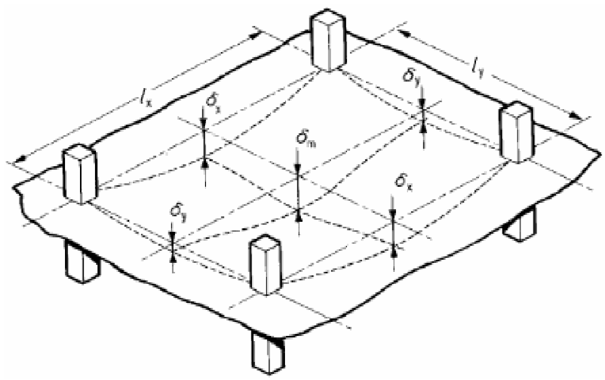


Figure 1.7: Deflection profile of Flat slab

Column strips behave as a continuous beam supported on column. The middle strip also behaves as a continuous beam supported on column strips and deflects as shown in fig1.7. The deflections are minimum at supports and maximum at mid spans. The deflected flat slab at the center of panel shall have saucer shape. Where δ_x and δ_y is the deflection at midspan in X and Y direction and l_x and l_y is the span length in X and Y direction.

Transfer of load from slab to column causes excessive shear stresses in the slab adjacent to column. This causes initiation of shear cracks at a distance of effective depth of slab from the face of column. These cracks propagate from bottom towards top. The failure occurs at the bottom compressed edge surrounding the column through punching.

- **Advantages and Disadvantages of Flat Slabs**

  Advantages

  - Flat slab floors eliminates beams thus allowing for reduction in storey height.
  - Reduce services and cladding costs.
  - As no beams are provided the formwork is simple.
  - Ease of installation of mechanic and electric services.
  - Curing is easy because of flat surface.
  - Reduced loads on foundation because of less thickness and less height of structure.
  - Simple formwork which saves construction time and hence cost.
  - It has plain ceiling which gives an attractive appearance and better illumination to the room.
  - Reduction in total height required for each storey thus increasing the number of floors that can be built in a specified height.
  - The locations of columns and wall are not restricted by the location of beams.

  Disadvantages

  - Thicker slab is needed.
  - Attention required to deflection control.
  - Very serious attention required to punching shear problem at slab to column connections.

1.2 Objective

A Reinforced Concrete flat slab floor is a significant advancement in the building technology. It has been observed that possible failure mode of the Reinforced concrete Flat slabs is punching that occurs in the vicinity of a column. The main objective of the study is to study method of analysis and design of flat slab with staggered column by IS 456-2000 [4] , ACI 318-08 [5] , BS 8110-1997 [6] & EC2:Part1-2004 [7]. Each code has specified the fixed coefficients for lateral and transverse distribution of moments as per direct design method and equivalent frame method. The project is aimed to determine the effect of staggered column spacing and its combination of shapes such as circular, rectangular and square columns. Also check whether those moments are remain the same when we analyze the flat slab with use of software SAFE. Project is also aimed to prepare the Excel worksheet for analysis and design of the flat slab with staggered column by equivalent frame method.

1.3 Scope of Work

The project works is concerned with the Analysis and Design of Flat slab with and without staggered column and to prepare the worksheet for analysis and design of flat slabs. The scope of work will be as below.

- Analysis and design of flat slabs is to be carried out for staggered columns using Equivalent Frame Method with IS 456-2000, ACI 318-08, BS 8110-1997 & EC2:Part1-2004.

- Analysis of flat slab with equivalent frame method using software SAFE.

- Preparation of excel worksheet analysis and design of flat slab with staggered column as per IS 456-2000, ACI 318-08, BS 8110-1997 & EC2:Part1-2004.

1.4 Organization of Major Project

The organization of chapters in this Project is as follows

Chapter 1, *Introduction*, deal with the basic introduction of flat slab and behavior of flat slab. It also includes the objective of study and scope of work.

Chapter 2, *Literature review*, provides an overview of the available books, publications, and papers from various journals on the topic of flat slab analysis and design. This survey gives an idea about the work carried out so far as well as needs to focus on specific topics where very few studies are carried out.

chapter 3, *Methodology of design of flat slab*, Describes the design of both methods Direct Design Method and Equivalent Frame Method. Also describe the different codes(IS 456-2000, ACI 318-08 , BS 8110-1997, EC2:Part1-2004)provisions and design steps.

Chapter 4, *Analysis and Design of Flat Slab*, describes design of flat slab (with & without staggered column) using various codes.In design chapter comparison of moments, punching shear & deflection is done.

Chapter 5, *Analysis and Design of Flat Slab Using Safe Software*, describes Design of flat slab(with & without staggered column) using SAFE software. Also comparison of manual & SAFE results is carried out.

chapter 6, *Conclusion and Future Scope*, includes conclusion and future scope of work.

Chapter 2

Literature Survey

2.1 General

Literature survey is carried out to familiar with the amount of work done in this area throughout the world. The survey gives ideas about the extent of work to be carried out during project. It helps in framing the scope of work. It also helps in deciding the line of action of work. It generates the clear vision of the work and gives the overall scenario of it. During this survey many new things, concepts, and ideas will emerge which improve the clarity of the topic. The literature is summarized as below.

2.2 Literature Review

Initially material related to the topic are searched out and collected through various sources. Papers from ASCE journals, Science direct, ACI etc. are collected and complied. Books related to Flat slabs are referred. The literature is summarized as below.

2.2.1 Methods for design of flat slab

Anitha et al. [8] illustrated the methods used for flat slab design using ACI-318, NZ-3101, and EC2 and IS: 456 design codes. For carrying out this project an interior panel of a flat slab with dimensions 6.6 × 5.6 m and super imposed load 7.75 kN/m^2 was designed using the codes given above. As per local conditions and availability of materials different countries have adopted different methods for design of flat slabs and given their guidelines in their respective codes.

Baskaran [9] discussed three fold. One is to encourage the application of the structural membrane approach to design flat slabs on non-rectangular column grid by providing experimental evidence. Second is to encourage more research on bimoment concepts. Third is to leave some carefully performed experimental evidence for the research community to be used in validating structural assessment tools like non-linear finite element or yield line analysis. In the next section, existing approaches to design flat slabs on non-rectangular column layout are briefly reviewed. Methods involving trial and error, like yield line design (considering the slab with an assumed steel distribution and assessing the load capacity for possible yield line patterns) or elastic finite element analysis (which results in peaky moments above columns and needs experience to perform redistribution) are not considered.

Kim and Lee [10] discussed merits and demerits of conventional methods of analysis of flat slabs such as Equivalent frame method and Finite Element Method. The author introduces the new and efficient method of analysis which is known as analysis of flat slabs by using super elements which is developed by using fictitious beams. This method significantly reduces the computational time and memory required for analysis as compared to conventional methods.

Bharath et al. [11] presented review and design of flat plate/slabs construction in India. They have described seismic design provisions per Indian Standard IS 1893 and Uniform Building Code UBC 2000 for the lateral force design of flat plate/slabs and also conclude by presenting two real world construction projects designed in Bangalore.

Murray et al.[12] described the development by Murray of a prediction method for estimating the ultimate load capacity of reinforced concrete flat slabs in the vicinity of edge columns. The method was developed from a study of the distribution of the total panel moment across the front face of the edge column. A parametric study to estimate the position of the point along the panel width where the longitudinal moment changed from hogging to sagging was conducted. Also include an improved prediction method for estimating the moment transferred to the edge column based on a modified ACI 318 [2] form of analysis. The results from the new prediction method have been compared with the experimental results obtained from a series of one-third-scale experimental test slabs constructed and tested at Queen's University, Belfast (QUB). The new prediction method gives a much less conservative estimate of both the edge column moment and the ultimate load capacity of the flat slab than the current design methods as proposed by BS 8110 [3] and ACI 318.

Corley and Jirsa [13] described the background of the equivalent frame method and presents a numerical equation as per ACI code. In addition,moments calculated with those measured in test slabs.

Simmonds and Misic [14] described the factors required for the cross moment distribution, stiffness factor and the carry-over factors which must be evaluated for the each element of the equivalent fame.

Vepari et al. [**15**] Presented the economical aspects of long span slabs between flat slab and grid slab. The objective function is to come on the proper method of selecting the slab forms on variable span. The flat slab was medelled and analyzed using the direct design method and grid slab was medelled and analyzed using plate theory method. The costing is calculated in three stages, which contain quantity of steel, volume of concrete to be used in slabs & beams and the cost of form work. By adding all these parameters, cost of slabs per square meter is determined.

Van Buren [**16**] Proposed that the coefficients for square and rectangular panels may be used as a basic for the design of slabs having staggered columns with only some minor modifications. while, for simplify, square panels were used in this analysis , the method applies equally to those cases where the span and panel width are different. It was believed that the principles and procedure outlined can be extended to structures with nonuniform column spacing and columns offset in two directions.

2.2.2 Deflection

Santhi et al. [**17**] determined the total deflection including creep and shrinkage effects by suitably modifying the available formula for beams as per Equivalent frame method. A programme is developed in MATLAB to determine the exact deflection for two-way beamless slabs including short-term effects, Creep effects and Shrinkage effects. A numerical example is solved for flat plates by varying the parameters such as total thickness, characteristic compressive strength of concrete, clear cover of reinforcement, creep coefficient and the disparities among BS 8110-1997, ACI 318-2000, and IS 456-2000 are highlighted.

Tavio and Teng [18] concentrated on the deflections of irregular flat plate floors and presented a possible method for computing the deflection. The ACI code formula for torsion stiffness will have to be modified to take accounts the irregular geometry, especially at edge slab-column connections.

Duarte et al.[19] described the experimental research carried out to study a strengthening method for flat slabs under punching using transversal shear reinforcement in the form steel bolts.

2.2.3 Books

P.C Varghese (2009)[20] Describe basic parameters related deflection calculation of reinforced concrete beams and slabs. It also includes the design of flat slabs.

S Unnikrishna and Devdas Menon (2003)[21] This books includes the basic concepts. It also includes the design of flat slabs as per ACI-318-08 provisions. Flat slab design steps using equivalent frame method are given in depth.

Prab Bhatt et al. (2006)[22] Describe parameters related to design of flat slab as per BS 8110-1997. It sets out design theory for concrete elements and structures, and illustrate practical application of the theory. It also includes the design of flat slabs as per BS 8110-1997 provisions.

IStructE (2006)[23] Manual for the design of concrete building structures to EC2:part1 describe parameters related deflection, punching shear calculation of flat slabs. It also includes design steps of deflection & punching shear calculation of flat slabs.

2.2.4 Summery

In this chapter, review of relevant literature is carried out. In the literature review, concepts of Flat slab, Equivalent fame method, Analysis and design of Flat slab are presented. These concepts are useful for understanding the analysis and design of flat slab.

Chapter 3

Methodology of Design of Flat Slab

3.1 General

Flat slabs are subjected to gravity and lateral loads. Analysis of flat slabs is mainly categorized into gravity load analysis and lateral load analysis. Flat slabs subjected to gravity loads are typically analyzed by direct design method and equivalent frame method. However flat slabs are also subjected to lateral loads due to wind and earthquake. The equivalent frame method can also be applied for lateral load analysis. Therefore the lateral load analysis is carried out by equivalent frame method and finite element analysis. Finite Element approach is computationally intensive even for flat slab, hence analysis is carried out using software.

Analysis of flat slab is mainly categorized into three parts as Gravity load analysis, Lateral load analysis and combined gravity and lateral load analysis.

Different standards such as IS 456-2000, ACI 318-08, BS 8110-1997 & EC2:Part1-2004 prescribed the use of different methods for analysis of flat slab.

3.2 Gravity Load Analysis

Codes of different countries prescribed the use of different methods for gravity load analysis such as Direct Design Method and Equivalent Frame Method.

3.2.1 IS:456-2000 CODE

IS 456 code suggests any of the two methods as Direct Design Method and Equivalent Frame Method for gravity load analysis of flat slab.

3.2.1.1 Proportioning of Flat Slab Components

Design of flat slab requires proportioning of dimensions of its different components and determination of reinforcement to satisfy both the serviceability and strength requirements. Proportioning includes deciding the thickness of slab, size of drop and size of column head.

- ***Thickness of Slab***

  Thickness of slab is chosen such that it satisfy both strength and serviceability criteria. Generally the thickness of slab is governed by the serviceability requirement for deflection than the requirements of strength. IS 456-2000 code recommends following limiting values of deflection for reinforced concrete structures, however in no case thickness of slab shall be less than 125 mm.

  Final deflection of horizontal members below the level of casting should not exceed span/250.

  Deflection after the construction of partitions or application of finishes should not exceed span/350 or 20 mm whichever is less.

- ***Drop***

  Drop shall be rectangular in plan whose minimum length in each direction shall not less than one third of the panel length in that direction. The maximum length of drop in each direction shall not greater than half the panel length in that direction. Thickness of the drop shall be 1.25 to 1.5 times thickness of the slab elsewhere.

- ***Column Head***

  It may be rectangular or circular. The length of rectangular column head in each direction shall not be more than one forth of panel length in that direction. In case of circular column head, diameter shall not exceed one forth of the average of the panel length in each direction. The portion of the column head lying within the largest right angle cone or pyramid that has a vertex angle of 90^0.

3.2.1.2 Direct Design Method

Moments in two way slabs can be found using direct design method are subject to the following restrictions.

3.2.1.2.1 Limitations of Direct Design Method

a. There shall be minimum of three continuous spans in each direction.

b. The panels shall be rectangular and the ratio of the longer span to the shorter span within a panel shall not be greater than 2.00

c. The successive span lengths in each direction shall not differ by more than one third of the longer span. The end spans may be shorter but not longer than interior spans.

d. The design live load shall not exceed three times the design dead load.

e. In the two way slabs with beams on all sides, it should also satisfy the following additional condition. The ratio of beam relative stiffness in the two directions is given by the expression $(\alpha_1/\alpha_2)/(L_1/L_2)^2$must lie between 0.2 to 5.0.

f. It shall be permissible to offset columns to a maximum of 10 percent of the span in the direction of the offset notwithstanding the provision in (2),

3.2.1.2.2 Total Static Moment at Factored Loads.

The total static moment M_o should be distributed firstly as positive and negative moments in the longitudinal direction and secondly these assigned positive and negative moments are again distributed transversely to the column and middle strips.

3.2.1.2.3 Longitudinal Distribution of Total Moment M_o.

The absolute sum of the positive and negative moment in each direction is given by

$$M_o = \frac{WL_n}{8} \tag{3.1}$$

Where,
M_0=total moment
W= design load on the area $L_2 \times L_n$
L_1 = length of span in the direction of M_o; and
L_2 = length of span transverse to L_1
In taking the values of L_n , L_1 and L_2 , the following clauses are to carefully noted:
(1) Circular supports shall be treated as square supports having the same area i.e., squares of size 0.886D.
(2) When the transverse span of the panel on either side of the centre line of support varies, L 2 shall be taken as the average of the transverse span. In figure1.2. it is given by $(L_{2a} + L_{2b})/ 2$
(3) When the span adjacent and parallel to an edge is being considered, the distance from the edge to the center-line of the panel shall be substituted for L_2.
L_n=clear span extending from face to face of columns, capitals, brackets or walls but not less than 0.65 L_1.

3.2.1.2.4 Distribution of Bending Moment in to Negative and Positive Moments.

In an interior span, the total design moment Mo shall be distributed in the following proportions

Negative design moment 0.65

Positive design moment 0.35

In the case of end spans, the total static moment among the three critical moment sections (interior negative, positive and end exterior negative) depends upon the flexural restraint provided for the slab by the exterior column or the exterior wall and depends also upon the presence or absence of beams on the column lines. In the end span the total design moment Mo shall be distributed in the following proportions.

Interior Negative Design Moment:

$$= [0.75 - \frac{0.10}{1 + \frac{1}{\alpha_c}}]M_o \tag{3.2}$$

Positive Design Moment:

$$= [0.63 - \frac{0.28}{1 + \frac{1}{\alpha_c}}]M_o \tag{3.3}$$

Exterior Negative Design Moment:

$$= [\frac{0.65}{1 + \frac{1}{\alpha_c}}]M_o \tag{3.4}$$

Where α_c is the ratio of flexural stiffness of the exterior columns to the flexural stiffness of the slab at a joint taken in the direction moments are being determined

Table 3.1: Distribution of moments across the panel

S. No.	*Distributed moment*	*Percent of total moment*
a	Negative BM at exterior support	100
b	Negative BM at interior support	75
c	Positive BM moment	60

is given by

$$\alpha_c = \frac{\sum k_c}{\sum k_s} \tag{3.5}$$

Where, $K_c$is the sum of the flexural stiffness of the columns meeting at the joint and K_s is the flexural stiffness of the slab, expressed as the moment per unit rotation. It shall be permissible to modify these design moments by up to 10 percent, so long as the total design moment, M_o for the panel in the direction considered is not less than that required by equation above.

3.2.1.2.5 Distribution of Bending Moments across the Panel Width

The positive and negative moments found are to be distributed across the column strip in a panel as shown in Table3.1.The moment in the middle strip shall be the difference between panel and the column strip moment.

3.2.1.2.6 Design Loads on the Beams

Beams in the column strip can be considered as rigid or flexible depending on the relative stiffness(α/α) . If this value is unity, the beams can be considered as rigid and it carries 85 percent of the moment in the column strip. If(α/α) <1.0 this value is zero, no moments are transferred to beams. If they are considered as flexible beams and the moments carried will be in proportion to its relative stiffness value as shown in Table3.2.

Table 3.2: Distribution of moments taken by beams

$\frac{\alpha L_2}{L_1}$	*Moment from column strip to beam(%)*
0	0
≥ 1.0	85

3.2.1.2.7 Moment in Columns

At the interior support the supporting members above and below the slab shall be designed to resist the moment M given by the following equation.

$$M = \frac{0.08(W_d + 0.5W_l)L_2L_n^2 - W_d^{'}L_2^{'}L_n^{'}}{1 + \frac{1}{\alpha_c}} \tag{3.6}$$

Where,

W_d,W_L=design dead and live loads respectively, per unit area;

L_2=length of span transverse to direction of M;

L_n=length of the clear span in the direction of M, measured face to face of support

3.2.1.2.8 Shear in Flat Slabs

Flat slabs must be designed to resist the shear as well as moment, when designing by direct design method or equivalent frame method. The critical section for shear shall be at a distance $d/2$ from the periphery of the column/ capital/ drop panel, perpendicular to the plane of the slab where d is the effective depth of the section.

Punching shear produces cracking at top of the slab where negative moment steel is also provided. This will leave only concrete at the bottom of the slab to resist shear. Hence this detailing is very important in flat slabs. However it should also be noted that when adjacent spans are unequal the extension of the negative steel beyond the face of support should be based on the longer span. The spacing of bars in the flat slab shall not exceed two times the slab thickness.

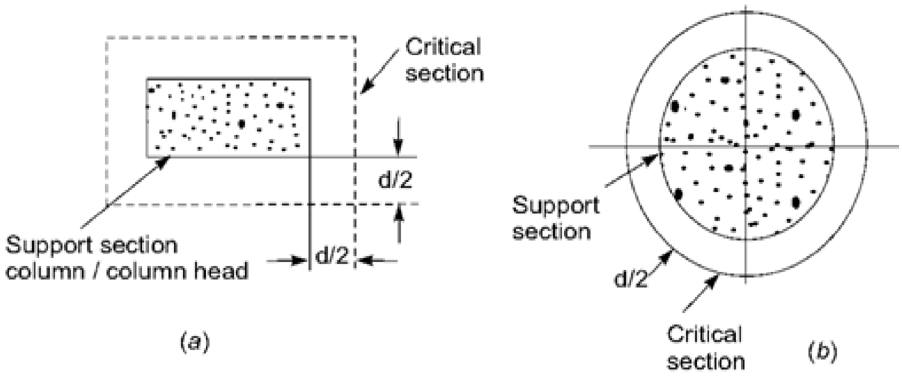


Figure 3.1: Critical section for shear

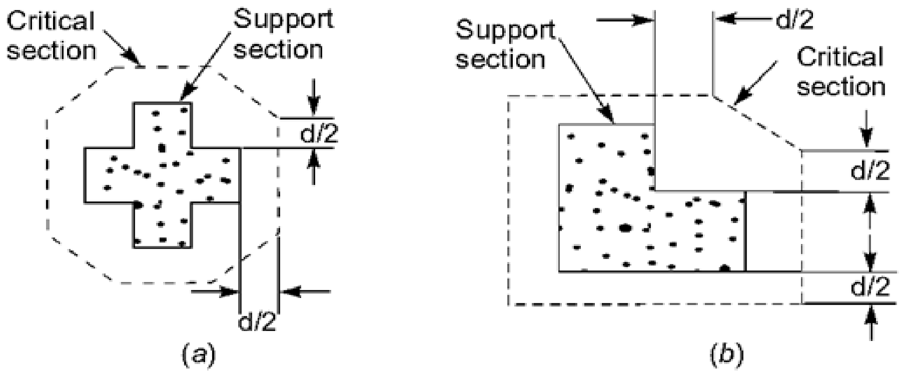


Figure 3.2: For column section with re-entrant angles

The nominal shear stress may be calculated as

$$\tau_v = \frac{V}{b_o d} \tag{3.7}$$

where V = shear force due to design

b_0 = the periphery of the critical section

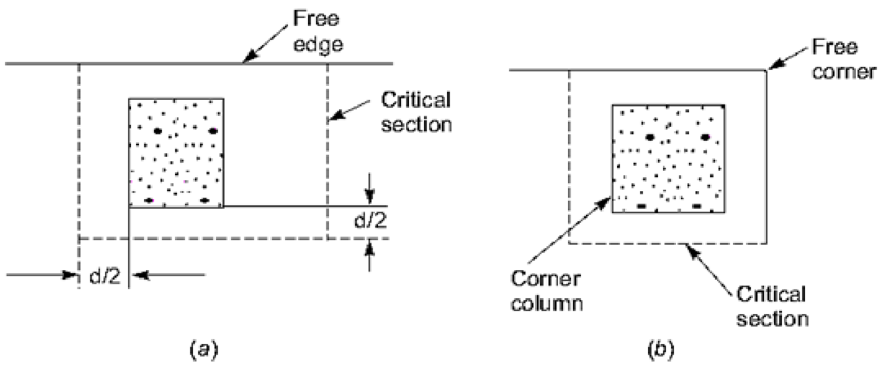


Figure 3.3: In case of columns near the free edge of a slab

d = the effective depth

The permissible shear stress in concrete may be calculated as k_s τ_c,

where $k_s = 0.5 + \beta_c$ but not greater than 1, where β_c is the ratio of short side to long side of the column/capital; and $\tau_c = 0.25\sqrt{fck}$

If shear stress $\tau_v < \tau_c$ no shear reinforcement are required. If $\tau_c < \tau_v < 1.5\tau_c$, shear reinforcement shall be provided. If shear stress exceeds $1.5\tau_c$ flat slab shall be re-designed.

The critical section for shear, column section with re-entrant angels, and column near the free edge of a slab are shown in Fig.3.11, 3.2 and3.3 respectively.

3.2.1.3 Equivalent Frame Method

IS 456-200 recommends the analysis of flat slab and column structure as a rigid frame to get design Moment and shear forces with the following assumptions:

- Beam portion of frame is taken as equivalent to the moment of inertia of flat slab bounded laterally by centre line of the panel on each side of the center line of the column. In frames adjacent and parallel to an edge portion shall be equal to flat slab bounded by the edge and the center line of the adjacent panel.

- Moment of inertia of the members of the frame may be taken as that of the gross section of the concrete alone.

- Variation of moment of inertia along the axis of the slab on account of provision of drops shall be taken into account. In the case of recessed or coffered slab which is made solid in the region of the columns, the stiffening effect may be ignored provided the solid part of the slab does not extend more than 0.15 L_{ef} into the span measured from the center line of the column.The stiffening effect of flared columns heads may be ignored.
- Analysis of frame may be carried out with substitute frame method or any other accepted method like moment distribution or matrix method.

Figure3.4 Shows a Typical Elevation of an Equivalent Frame.

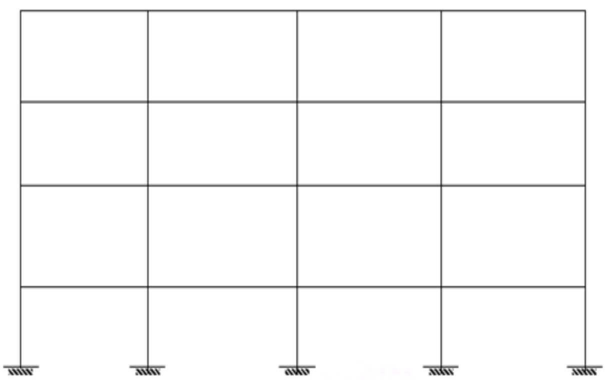


Figure 3.4: Elevation of an equivalent frame

The analysis is one for each typical equivalent frame. An equivalent frame is modeled by slab-beam members and equivalent columns as shown in Fig.3.5. The equivalent frame is analyzed for gravity load and lateral load (if required), by computer or simplified hand calculations. Next, the negative and positive moments at the critical sections of the slab-beam members are distributed along the transverse direction. This provides the design moments per unit width of a slab.

If the analysis is restricted to gravity loads, each floor of the equivalent frame can be analyzed separately with the columns assumed to be fixed at their remote ends, as shown in the following figure. The pattern loading is applied to calculate the moments for the critical load cases.

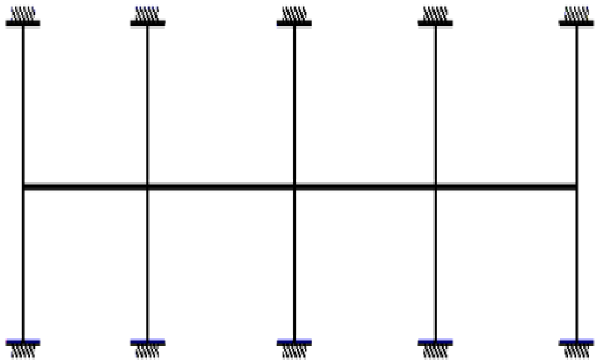


Figure 3.5: Simplified model of an equivalent frame

3.2.1.3.1 The Steps of Analysis & Design of a Two-Way Slab are as Follows.

(1) Determine the factored negative (M_u^-) and positive moment (M_u^+) demands at the critical sections in a slab-beam member from the analysis of an equivalent frame. The values of (M_u^-) are calculated at the faces of the columns. The values of (M_u^+) are calculated at the spans. Fig.3.6 shows a typical moment diagram in a level of an equivalent frame due to gravity loads.

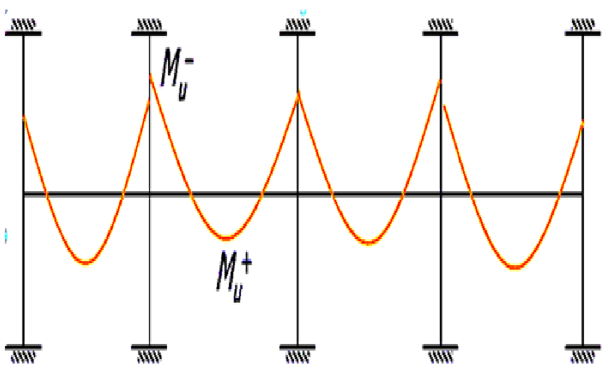


Figure 3.6: Typical moment diagram due to gravity loads

(2) Distribute (M_u^-) to the Column strip and the Middle strip. These components are represented as($M_{u,cs}^-$) and ($M_{u,ms}^-$), respectively. Fig.3.7 shows the distribution (M_u^+) to the Column strip and the Middle strip. These components are represented as ($M_{u,cs}^+$) and ($M_{u,ms}^+$), respectively.

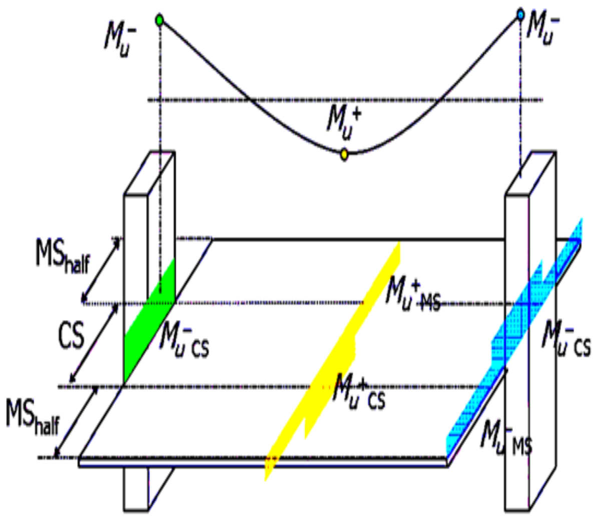


Figure 3.7: Distribution moment to the column strip and the moment strip

(3) If there is a beam in the column line in the spanning direction, distribute each of ($M_{u,cs}^-$) and ($M_{u,cs}^+$) between the beam and rest of the Column strip as shown in Fig.3.8.

(4) Add the moments ($M_{u,ms}^-$) and ($M_{u,ms}^+$) for the two portions of the Middlestrip (from adjacent equivalent frames).

(5) Calculate the design moments per unit width of the Column strip and Middle strip.

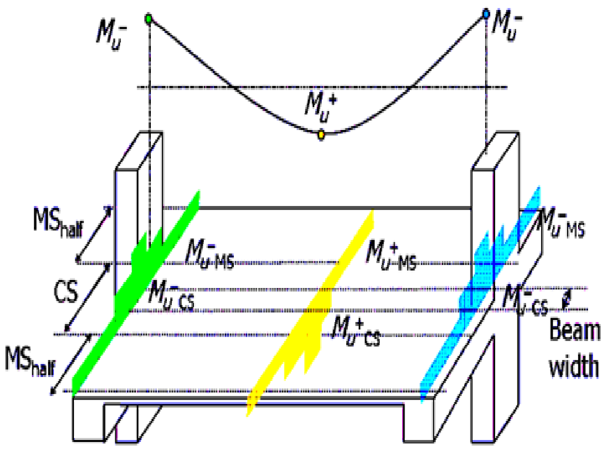


Figure 3.8: Distribution of moment to beam, column strip and middle strip

- **Loading Pattern**

When the live load does not exceed $3/4^{th}$of dead load, the maximum moments may be assumed to occur at all sections when full design live load is on the entire slab.
If live load exceeds $3/4^{th}$ dead load analysis is to be carried out fir the following pattern of loading also:

a. To get maximum moment near mid span $3/4^{th}$ of live load on the panel and full live load on alternate panel

b. To get maximum moment in the slab near the support $3/4^{th}$ of live load is on the adjacent panel only

It is to be carefully noted that in no case design moment shall be taken to be less than those occurring with full design live load on all panels.
The moments determined in the beam of frame (flat slab) may be reduced in such proportion that the numerical sum of positive and average negative moments is not less than the value of total design moment M_o=$WL_n/8$ .The distribution of slab moments into column strips and middle strips is to be made in the same manner as specified in direct design method.

Table 3.3: Effective span to Effective depth ratio

	Support Conditions	Effective span/ Effective depth
	Span $\leq$10 m	Span $>$ 10 m
Cantilever	7	Deflection Calculation Shall be made
Simply Supported	20	$(20 \times 10)/\text{span}$
continuous	26	$(26 \times 10)/\text{span}$

3.2.1.4 Design Requirements

General design requirements in flat slabs include serviceability requirements and strength requirements.

- ***Serviceability Requirements***

  The design of flat slab is made to satisfy the serviceability requirements of deflection and crack. The serviceability requirement of deflection is controlled by effective span to effective depth ratio as shown in Table3.3. For two way slabs shorter of the two spans shall be used for calculating the span to effective depth ratio. The minimum thickness of the slab shall not be less than 125 mm.

  The increasing use of limit state method of design and high strength steel lead to a wide cracks in concrete structures, thus necessitating control for cracking. The maximum width of crack is limited on the basis of appearance of structure, durability and corrosion. IS 456-2000,recommends maximum crack width 0.3mm for structures not subjected to aggressive environment while in members where cracking in the tensile zone is harmful IS 456 code suggests maximum width of crack 0.2mm. The serviceability requirement for crack is controlled by the spacing of reinforcement. Usually the spacing of reinforcement based on design for strength requirement is smaller than the maximum spacing for crack control.

- ***Deflection due to imposed load***

  The total load that comes on a structure is composed of dead and applied live loads. the dead load and that part of the live load that always act on the structure are called permanent loads as outlined in Table.3.4.

Table 3.4: Live load factors to calculate permanent loads to estimate deflection

Item	Long-term deflection	Short-term deflection
Roof with traffic	0.2	0.7
Floor-residential	0.3	0.7
Floor-office	0.2	0.5
Floor-retail	0.3	0.6
Floor-storage	0.5-0.8	1.0

- ***Deflection due to dead load***

  deflection is given by the formula

$$\alpha = \frac{M_{Max}}{EI_{eff}} KL^2 \tag{3.8}$$

$$I_{eff} = \frac{I_r}{1.2 - (M_r/M)(Z/d)[1-(x/d)](b_w/b)} \tag{3.9}$$

where,

$I_r < I_{eff} < I_{gr}$

$M_r = Crackingmoment = (I_{gr}/y_r)f_{cr}$

y_r= Distance of extreme tension fibre from centroid of section

$f_{cr} = modulesofrupture = 0.7\sqrt{f_{ck}}$

z= lever arm in elastic theory = d-(x/3)

- ***Long-term deflection due to creep***

  creep strain= Elastic strain $\times$ Ultimate creep strain $= \varepsilon_c\theta$ Accordingly, the long-term strain $\varepsilon_c\theta$ can be expressed as

$$\varepsilon_c\theta = \varepsilon_c(1+\theta) \tag{3.10}$$

The ultimate creep coefficient θ as shown in Fig.3.5. is considered as the result of a number of factors at which the important factors are humidity.

Table 3.5: Live load factors to calculate permanent loads to estimate deflection

Age of loading	value of θ
7-days	2.2
28-days	1.6
1 year	1.1

- ***Deflection due to shrinkage***

$$\alpha_{cs} = K_3\psi_{cs}L^2 \tag{3.11}$$

The shrinkage coefficient can be taken from Table.3.6.

Table 3.6: Deflection due to shrinkage coefficient

Support condition	Coefficient K_3
cantilever	0.500(1/2)
simple beams	0.125(1/8)
Continuous at one end only	0.086(11/128)
Continuous at both end only	0.063(1/16)

- ***Strength Requirements***

  The design of reinforcement is made to satisfy strength requirements for moment and shear as.

- ***Design For Moment***

  The reinforcement required for positive and negative moments in the column and middle strips can be determined either by limit state method or working stress method.

3.2.1.5 Slab Reinforcement

- ***Spacing***

  The spacing of bars in a flat slab, shall not exceed 2 times the slab thickness.

- ***Area of Reinforcement***

  When the drop panels are used, the thickness of drop panel for determining area of reinforcement shall be the lesser of the following:
  (a) Thickness of drop, and
  (b) Thickness of slab plus one quarter the distance between edge of drop and edge of capital.
  The minimum percentage of the reinforcement is same as that in solid slab i.e., 0.12 percent if HYSD(Fe 415) bars used and 0.15 percent, if mild steel is used.

  Minimum Length of Reinforcement at least 50 percent of bottom bars should be from support to support. The rest may be bent up. The minimum length of different reinforcement in flat slabs should be as shown in Fig.3.9 (Fig. 16 in IS 4562000). If adjacent spans are not equal, the extension of the ve reinforcement beyond each face shall be based on the longer span. All slab reinforcement should be anchored property at discontinuous edges.

[NO SLAB CONTINUITY] [CONTINUITY PROVED] [NO SLAB CONTINUITY]

	Bar Length From Face of Support						
	Minimum Length					Maximum Length	
Mark	a	b	c	d	e	f	g
Length	0.14 l_n	0.20 l_n	0.22 l_n	0.30 l_n	0.33 l_n	0.20 l_n	0.24 l_n

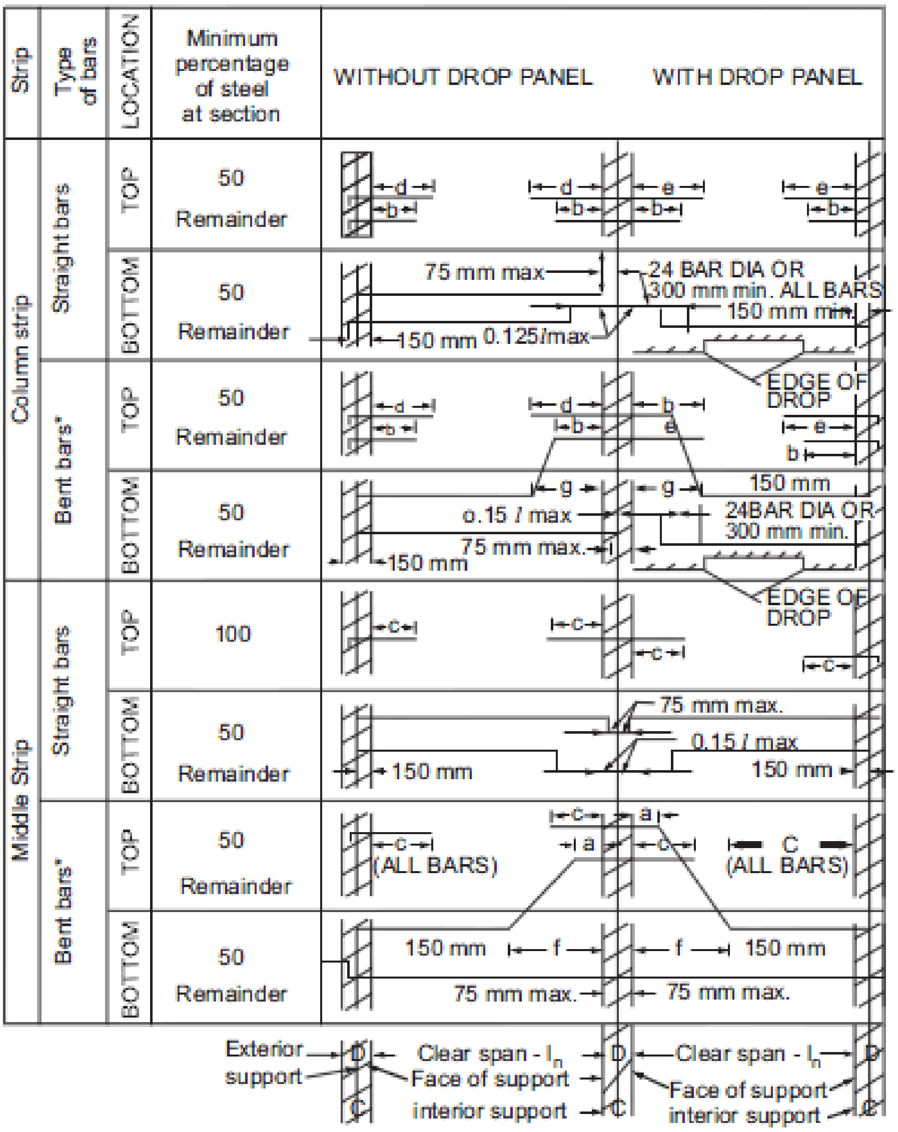


Figure 3.9: Minimum bend joint locations and extensions for reinforcement in flat slabs

3.2.2 ACI 318-08

ACI 318-08 suggests any of the two methods as direct design method and equivalent frame method for gravity load analysis of flat slab.

3.2.2.1 Direct Design Method

Negative and Positive Moments:

Longitudinal distribution of moments in the interior span is in the same proportion as that of IS 456 code as

Negative design moment 0.65

Positive design moment 0.35

ACI 318 prescribed fixed values for distribution of moments in the end span for various cases as shown in Table3.7 that can occur in practice and these co-efficient can be directly chosen for design.

Table 3.7: Longitudinal distribution of moments in end span

	(a)	(b)	(c)	(d)	(e)
	Exterior edge Unrestrained	Slab with Beams Between all Supports	Slab without beams Between interior Supports		Exterior edge Fully Restrained
			Without edge Beam	With edge beam	
Interior Negative Moment	0.75	0.7	0.7	0.7	0.65
Positive Moment	0.63	0.57	0.52	0.5	0.35
Exterior Negative Moment	0	0.16	0.26	0.3	0.65

Transverse Distribution of Moments:

Having distributed the moment M_O longitudinally to the positive and negative moments at the two ends, these moments are again distributed in the column strip and middle strips of respective sections, which carried out using Table3.8. The main factors that affect the transverse distribution of the moments are the relative stiffness of the beam in the column strip to that of slab, L_2/L_1 ratio, torsion resistance of the edge beam (if present) in the exterior span; type of wall in case of the slab whose exterior end is supported on wall.

Table 3.8: Transverse distribution of moments to C.S (percentages)

	Moments to be distributed	Type of bemas present	$\alpha L_2/L_1$	β_t	0.5	1.0	2.0
1	Positive moment in all spans	(a) no internal beam	0	Nil	60	60	60
		(b) with internal beam	≥ 1	Nil	90	75	45
2	Negative moment in interior spans	(a) No internal beam	0	Nil	90	75	75
		(b) with internal beam	≥ 1	Nil	90	75	75
3	Negative moment in exterior support	(a) No internal beam no edge beam	0	0	100	100	100
		(b) No internal beam with edge beam	0	≥ 2.5	75	75	75
		(c) with internal beam no edge beam	≥ 1	0	100	100	100
		(d) with internal beam with edge beam	≥ 1	≥ 2.5	90	75	45

3.2.2.2 Equivalent Frame Method

Flat slab subjected to gravity loads are typically analyzed by direct design method and equivalent frame method. In both methods the two way slab system is converted into series of rigid frames in either of two directions (X and Y direction) by means of vertical cuts through the slabs at section midway between the column lines. The difference in analysis of flat slab by DDM and EFM is that in the direct design method fixed coefficients are used for calculating moments in various parts of the slab while in the equivalent frame method, the actual frames are analyzed by any one of the classical methods of structural analysis, like moment distribution, slope deflection or matrix method. Because of this DDM will only be applicable to more or less symmetrical layout of columns and slabs; however EFM can be used for any layout.

The two way slab system which does not satisfy the limitations of the direct design method (DDM) shall be analyzed by the equivalent frame method (EFM).The equivalent frame method is very similar to DDM but it uses the classical method of analysis, instead of using the coefficients, to give the positive and negative moments in the longitudinal direction. Thus the difference between DDM and EFM analysis for gravity loads lies only in the procedure of getting the magnitude of the longitudinal negative and positive moments.

- ***Definition of Equivalent Frame***

  The equivalent frame is a simple substitute of a two dimensional model for a three dimensional frame consisting of slabs and columns. Building frame is cut vertically from top to bottom first in longitudinal direction and then in the transverse direction along the centre lines of the adjacent panels. In EFM analysis the spans are considered from centre to centre of slabs. In general the Frame used for analysis in EFM is consisting of three members.

(1) Series of slabs (or slab beams) in the longitudinal direction.

(2) Columns extending above and below the slabs.

(3) The transverse moment transfer elements (torsion members) consisting of the slab and beams (if any) at the columns in the transverse direction.

The resulting frames are then reduced to two-dimensional model with columns as vertical members and the slabs compressed to equivalent beams as horizontal members. These two dimensional models are equivalent frames and they are to be analyzed by methods of elastic analysis such as moment distribution, slope deflection or matrix methods. Each floor is analyzed separately with the columns assumed fixed at the floors above and below for the worst live load condition (pattern loading) on the 'slab beams', as this method is considered as an exact method redistribution of moments is allowed. The analysis is made in both the longitudinal and transverse direction.

- ***Limitations of Equivalent Frame Method***

  Equivalent frame method has limitations in the applications and accuracy because the slabs are modeled by an equivalent frame method can be easily applied to a flat slab structures having the regular plan shown in Fig.3.10(a). However, it is hard to apply the equivalent frame method to flat slab structures having irregular spans shown in Fig.3.10(b) this is because of the fact that this method was originally derived from the buildings having regular arrangement of columns and slabs.

  Also it is difficult to apply equivalent frame method to the structures having openings in the slab shown in Fig.3.10(c). Since the equivalent frame method cannot accurately represent the stress distribution that is one of the most important factors in the design of slabs. In spite of the limitations above, the equivalent frame method is widely used by the engineers because there is no appropriately simple analytical method for flat slab structures.

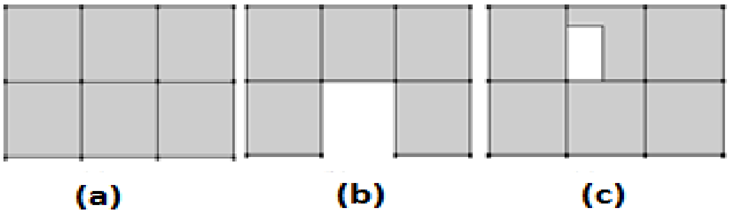


Figure 3.10: Plan of flat slab

- **Design Procedure**

The following procedure is usually recommended for orderly analysis of flat slab by equivalent frame method:

(1) Calculate from the geometry C_1/L_1 ,C_2/L_2

(2) Determine the stiffness of column by

$$K_c = \frac{K_c E_c I_c}{H} \tag{3.12}$$

where,

K_c = column stiffness coefficient

E_c = modulus of elasticity of concrete

H = height of the column

(3) Calculate the torsional stiffness of the attached member by

$$K_t = \frac{(\sum 9E_c C)}{L_2[1-(C_2/L_2)]^3} \tag{3.13}$$

where,

L_2 = span of member subjected to torsion on either side of column

L_1 = The length of the side of the column in the transverse direction in line with L_2

C=torsional constant of transverse torsional member given by

$$C = \sum(1 - 0.63\frac{x}{y})(\frac{x^3y}{3}) \tag{3.14}$$

where x is less than y in the rectangle

(4) Estimate the stiffness of the equivalent column by

$$K_{ec} = \frac{\sum K_c \times \sum K_t}{\sum K_c + \sum K_t} \tag{3.15}$$

(5) determine the B.M coefficient carryover factor and coefficient for stiffness of slab from tables.Determine also the stiffness of the slab by

$$K_{sb} = K_{nf}\frac{E_{cs} \times I_s}{L_1} \tag{3.16}$$

where,

K_{nf} = stiffness factor

(6) From stiffness of slab and column, calculate the distribution factors.

(7) Analyze the equivalent frame by moment distribution method.

(8) Having obtained the negative moments at supports and the positive moments in the span,distribute them to the column strip and middle strips.

- **Calculation of Deflections**

  The ACI method uses the simple approach suggested in 1960 by Yu and winter. It also assumes an effective moment of inertia proposed by Branson in 1977. It is based on a study of beams with M/M_r values from 2.2 to 4.0 and I_{gr}/I_r varying from 1.3 to 3.5. The equation for I_{eff} is given in ACI 318-08 as follows:

$$I_{eff} = (\frac{M_r}{M})^3 I_{gr} + [1 - (\frac{M_r}{M})^3] I_r \tag{3.17}$$

- ***Long-term Deflection***

  Additional long-term deflection due to combined effects of creep and shrinkage i obtained in ACI method by multiplying the Short-term deflection by following factor:

$$\lambda = \frac{t}{1 + 50\rho} \tag{3.18}$$

where,

ρ= Ratio of compression steel A_{sc}/bd

t= Time dependent factor taken as follows:

3 months = 1.0, 6 months= 1.2 , 1 year = 1.4 and 5 year = 2.0

3.2.3 BS 8110-1997

Provisions are given for the design of flat slabs supported by a generally rectangular arrangement of columns using the equivalent frame method and where the ratio of the longer to the shorter spans does not exceed 2.

- **Thickness of Panels**

  The thickness of the slab will generally be controlled by consideration of deflection.the basic span/effective depth ratio for beam ar shown in3.3. these are based on limiting the total deflection to span/250 and this should normally ensure that the part of the deflection occurring after construction of finishes and partitions will be limited to span/500 or 20mm. whichever is the lesser for spans up to 10m. In no case, however, should the thickness of the slab be less than 125mm.

- **Effective Diameter of a Column or Column Head**

  The effective diameter of a column or column head is the diameter of a circle whose area equals the cross-sectional area of the column or, if column heads are used, the area of the column head based on the effective dimensions as defined

$$l_h max = l_c + 2(d_h - 10) \tag{3.19}$$

$$h_c = (\frac{4A}{\pi})^{(\frac{1}{2})} \leq 0.25 l_x \tag{3.20}$$

  In no case should h_c be taken as greater than one-quarter of the shortest span framing into the column.

- **Limitation of Negative Design Moments**

  Negative moments greater than those at a distance $h_c/2$ from the center-line of the column may be ignored providing the sum of the maximum positive design moment and the average of the negative design moments in any one span of the slab for the whole panel width is not less than:

$$\frac{nl_2}{8}(l_1 - \frac{2h_c}{3})^2 \tag{3.21}$$

- **Limitation of Moment Transfer**

  The maximum design moment $M_t$max which can be transferred to a column by this strip is given by:

  $$M_t max = 0.15 b_e d^2 f_{cu} \tag{3.22}$$

  where,

  $M_t$max = should be not less than half the design moment obtained from an equivalent frame analysis or 70% of the design moment if a grillage or finite element analysis has been used.

- **Division of Moments Between Columns and Middle Strips**

  The moments obtained from analysis of frames should be divided as shown in Table.3.9(these percentages are for slabs without drops):

Table 3.9: Coefficient for distribution of moments in column and middle strip

Design moment	column strip	Middle strip
Negative	75%	25%
Positive	55%	45%

- **Shear in Flat Slabs**

  Flat slabs must be designed to resist the shear as well as moment, when designing by direct design method or equivalent frame method. The critical section for shear shall be at a distance 1.5d as shown in Fig.4.54 from the periphery of the column/ capital/ drop panel, perpendicular to the plane of the slab where d is the effective depth of the section.

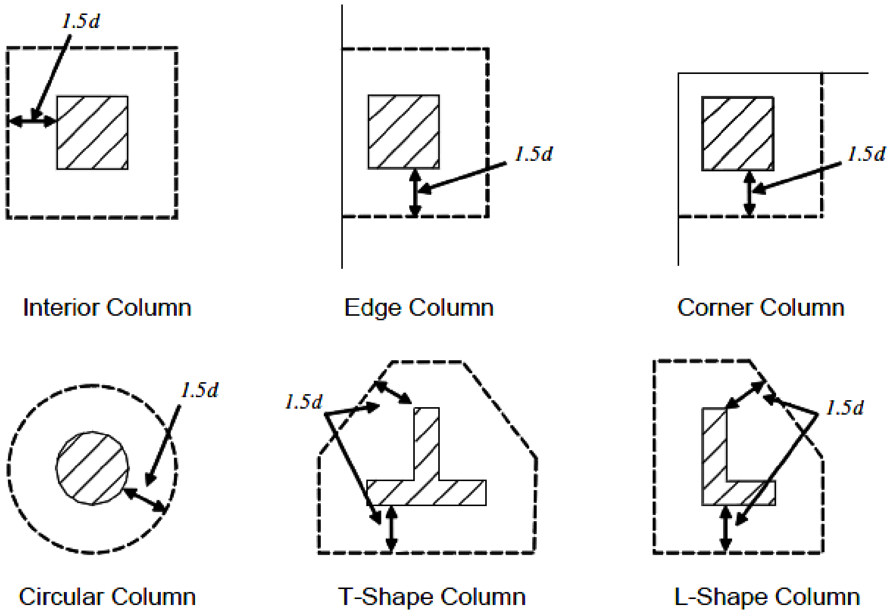


Figure 3.11: Critical section for shear

for internal column connections:

$$V_{eff} = V_t(1 + \frac{1.5M_t}{V_t x}) \tag{3.23}$$

where,

V_t=calculated shear from analysis

M_t=moment transferred to column by

x= length of side of perimeter considered parallel to axis of bending

Alternatively,

$$V_{eff} = 1.15V_t \tag{3.24}$$

for corner column connections:

$$V_{eff} = 1.25V_t \tag{3.25}$$

for edge column connections: for bending about axis parallel to free edge

$$V_{eff} = 1.25V_t \tag{3.26}$$

for bending about axis perpendicular to free edge

$$V_{eff} = V_t(1.25 + \frac{1.5M_t}{V_t x} \tag{3.27}$$

Alternatively,

$$V_{eff} = 1.4V_t \tag{3.28}$$

Deflection due to shrinkage

The shrinkage curvature is calculated by the following formula:

$$\psi_{cs} = \frac{mS_s\varepsilon_{cs}}{I_r} \tag{3.29}$$

where,

ε_{cs}= Free shrinkage strain

m= Modular ratio = E_s/E_{eff}

E_{eff}= Long-term modulus, $E_c/(1+\theta)$

S_s= First moment of area section using the centroid of the cracked section(neutral axis)is equal to $\sum A_S e_s$ the moment of compression steel being taken as negative.

3.2.4 EuroCode2:Part1-2004

If a flat slab has at least three spans or bays in each direction and the ratio of the longest span to the shortest does not exceed 1.2, the design moments obtained from analysis of the frames should be divided between the column and middle strip in the proportions given same as BS code in Tabel3.9.

3.2.4.1 Design Procedure

(1) carry out analysis of slab to determine design moments(M)

(2) determine K from:

$$k = \frac{M_u}{bd^2 f_{ck}} \tag{3.30}$$

(3) determine K′ from Fig.3.12 or

% redistribution	δ (redistribution ratio)	*K′*
0	1.00	0.208[a]
10	0.90	0.182[a]
15	0.85	0.168
20	0.80	0.153
25	0.75	0.137
30	0.70	0.120

Key

a It is often recommended in the UK that *K′* should be llimited to 0.168 to ensure ductile failure

Figure 3.12: Value for k′[24]

$$k' = 0.60\delta - 0.18\delta^2 - 0.21 \quad where\delta \leq 1.0 \tag{3.31}$$

(4) check K $\leq$ k′ ,no compression reinforcement required

(5) obtain lever arm z from Fig.3.13 or

$$z = \frac{d}{2}[1 + \sqrt{1 - 3.53k}] \leq 0.95d \tag{3.32}$$

K	z/d	K	z/d
≤ 0.05	0.950[a]	0.13	0.868
0.06	0.944	0.14	0.856
0.07	0.934	0.15	0.843
0.08	0.924	0.16	0.830
0.09	0.913	0.17	0.816
0.10	0.902	0.18	0.802
0.11	0.891	0.19	0.787
0.12	0.880	0.20	0.771

Key
a Limiting *z* to 0.95*d* is not a requirement of Eurocode 2, but is considered to be good practice

Figure 3.13: Value for lever arm(z)[24]

(6) calculate tension reinforcement required from

$$A_s = \frac{M}{f_y d Z} \tag{3.33}$$

(7) check minimum reinforcement requirement from Fig.3.14 or

$$A_{s,min} = \frac{0.26 f_{ctm} b_t d}{f_{yk}} \quad where f_{yk} \geq 25 \tag{3.34}$$

f_{ck}	f_{ctm}	Minimum % ($0.26 f_{ctm}/f_{yk}$[a])
25	2.6	0.13%
28	2.8	0.14%
30	2.9	0.15%
32	3.0	0.16%
35	3.2	0.17%
40	3.5	0.18%
45	3.8	0.20%
50	4.1	0.21%

Key
a Where f_{yk} = 500 MPa

Figure 3.14: Minimum percentage of reinforcement required[24]

(8) check maximum reinforcement requirements for tension or compression reinforcement outside lap locations

$$A_{s,max} = 0.04 A_c \tag{3.35}$$

3.2.4.2 Procedure for determining punching shear capacity

(1) Determine value of factor β from Fig.3.15.

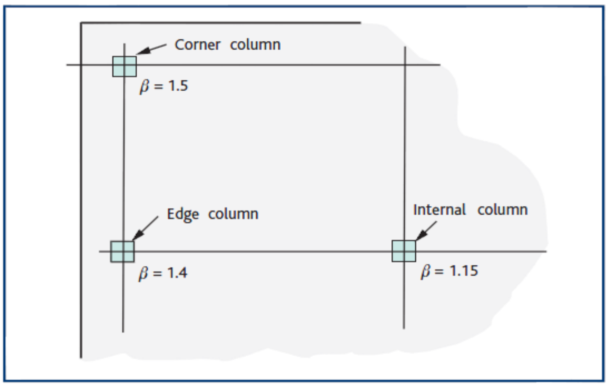


Figure 3.15: Recommended standard values of β[24]

(2) Determine value of $V_{ED,max}$

$$V_{ED,max} = \beta V_{ED}/(U_o d_{eff}) \tag{3.36}$$

where U_o is perimeter of column

For an interior column U_o = Length of the column perimeter

For an edge column $U_o = C_2 + 3\text{d} \leq C_2 + C_1$

For an corner column $U_o = 3\text{d} \leq C_2 + C_1$

$$d_{eff} = (d_y + d_Z)/2 \tag{3.37}$$

d_y and d_Z are the effective depths in orthogonal directions

(3) Determine value of $V_{Rd,max}$ from Fig.3.16.

f_{ck}	$v_{Rd,max}$	d_{eff}	$f_{ywd,ef}$
20	3.31	150	288
25	4.05	175	294
28	4.48	200	300
30	4.75	225	306
32	5.02	250	313
35	5.42	275	319
40	6.05	300	325
45	6.64	325	331
50	7.20	350	338

Figure 3.16: Values of $V_{Rd,max}$ and $f_{ywd,ef}$[24]

(4) If $V_{ED,max} \leq V_{Rd,max}$

(5) Determine value of V_{ED}(design shear stress)

(6) Determine concrete punching shear capacity(without shear reinforcement),$V_{RD,c}$ from Fig.3.17 where,

$$\rho_i = (\rho_{iy}\rho_{iZ})^{0.5} \tag{3.38}$$

(7) Determine area of punching shear reinforcement per perimeter from :

$$A_{sv} = (V_{ED} - 0.75V_{RD,c})S_t U_i/(1.5f_{ywd,ef}) \tag{3.39}$$

determine value of $f_{ywd,ef}$ from Fig.3.16.

(8) Determine the length of the outer perimeter where shear reinforcement not required from:

$$U_{out,ef} = \beta V_{ED}/(V_{RD,c}d) \tag{3.40}$$

(9) Determine layout of punching shear reinforcement

ρ_l	Effective depth, d (mm)										
	≤200	225	250	275	300	350	400	450	500	600	750
0.25%	0.54	0.52	0.50	0.48	0.47	0.45	0.43	0.41	0.40	0.38	0.36
0.50%	0.59	0.57	0.56	0.55	0.54	0.52	0.51	0.49	0.48	0.47	0.45
0.75%	0.68	0.66	0.64	0.63	0.62	0.59	0.58	0.56	0.55	0.53	0.51
1.00%	0.75	0.72	0.71	0.69	0.68	0.65	0.64	0.62	0.61	0.59	0.57
1.25%	0.80	0.78	0.76	0.74	0.73	0.71	0.69	0.67	0.66	0.63	0.61
1.50%	0.85	0.83	0.81	0.79	0.78	0.75	0.73	0.71	0.70	0.67	0.65
1.75%	0.90	0.87	0.85	0.83	0.82	0.79	0.77	0.75	0.73	0.71	0.68
≥ 2.00%	0.94	0.91	0.89	0.87	0.85	0.82	0.80	0.78	0.77	0.74	0.71
k	2.000	1.943	1.894	1.853	1.816	1.756	1.707	1.667	1.632	1.577	1.516

Notes

1 Table derived from: $v_{Rd,c} = 0.12\,k\,(100\rho_l f_{ck})^{1/3} \geq 0.035\,k^{1.5} f_{ck}^{0.5}$ where $k = 1 + \sqrt{(200/d)} \leq 2$ and $\rho_l = \sqrt{(\rho_{ly} + \rho_{lz})} \leq 0.02$, $\rho_{ly} = A_{sy}/(bd)$ and $\rho_{lz} = A_{sz}/(bd)$

2 This table has been prepared for $f_{ck} = 30$;
Where ρ_l exceeds 0.40% the following factors may be used:

f_{ck}	25	28	32	35	40	45	50
Factor	0.94	0.98	1.02	1.05	1.10	1.14	1.19

Figure 3.17: Values of $V_{Rd,c}$[24]

3.2.4.3 Procedure for calculating deflection

(1) calculate the moment, M_{QP} due to quasi-permanent actions at the critical section

(2) Obtain concrete properties, f_{ctm} and E_{c28}

(3) Calculate creep coefficient, $\varphi(\infty, t_o)$.

(4) Calculate long-term elastic modulus, E_{eff} from:

$$Eeff = E_{c28}/[1 + \varphi(\infty, t_o)] \tag{3.41}$$

Calculate effective modulus ratio , α_e from:

$$\alpha_e = E_s/E_{eff} \tag{3.42}$$

calculate depth to neutral axis for uncracked condition, x_u

Calculate second moment of area for uncracked conditions, l_u

calculating cracking moment ,M_{cr} from:

$$M_{cr} = \frac{0.9 f_{ctm} l_u}{h - x_u} \tag{3.43}$$

(5) $If M_{cr} > M_{QP}$ section is uncracked $\zeta = 0$

$If M_{cr} < M_{QP}$ section is cracked $\zeta = 1 - 0.5(M_{cr}/M_{QP})^2$

(6) calculate flexural curvature

$$\frac{1}{r_n} = \zeta \frac{M_{QP}}{E_{eff} I_C} + (1 - \zeta) \frac{M_{QP}}{E_{eff} I_U} \tag{3.44}$$

(7) Calculate total shrinkage strain $\varepsilon_{cs} from \varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{cd}$

$\varepsilon_{cd} = K_h$ $\quad\varepsilon_{cd}$= Drying shrinkage strain

K_h= Coefficient based on notional size

ε_{cdo}= Nominal unrestrained drying shrinkage

$$\varepsilon_{ca} = \beta_{as}(t) \varepsilon_{ca}(\infty) = \varepsilon_{ca}(\infty) \tag{3.45}$$

(8) Calculate curvature due to shrinkage strain $1/r_{cs}$

(9) Calculation total curvature

$$\frac{1}{r_{t,QP}} = \frac{1}{r_n} + \frac{1}{r_{cs}} \tag{3.46}$$

(10) Calculate quasi-permanent deflection from $\delta_{QP} = K L_2 (1/r_{t,QP})$

3.3 summary

In this chapter Describe the flat slab design of both methods Direct Design Method and Equivalent Frame Method. Also describe the different codes (IS 456-2000, ACI 318-08, BS 8110-1997, EC2:Part1-2004) provisions and design steps.

Chapter 4

Analysis and Design of Flat Slab

4.1 General

Flat slabs can be designed by limit state method as well as working stress method. In order to satisfy strength as well as serviceability criteria codes of different countries such as I.S.456-2000, ACI-318-08 , B.S.8110-1997 and Euro2-part1-2004 recommends the design of flat slab by limit state method, however design rules differ considerably in codes of different standards.

4.2 Design Example as per IS 456-2000

4.2.1 With Staggered Column

Design the flat slab with staggered column shown in figure. Using equivalent frame method. It is subjected to live load of 3 kN/m^2 and floor finish of 1 kN/m^2. The grade of steel used is Fe 415.Floor to floor height of column 3.5m.

Given:

L.L = 3 kN/m^2	clear cover = 20 mm
fck = 30 N/mm^2	fy =415 N/mm^2
Column diameter = 390 mm	Floor to floor height of column = 3.5 m

Column		Size, mm	Area,mm^2
Square	x- direction	350 mm	122500
	y-direction	350 mm	
Circular	Diameter	390 mm	119398.5
rectangular	x-direction	300 mm	120000
	y-direction	400 mm	

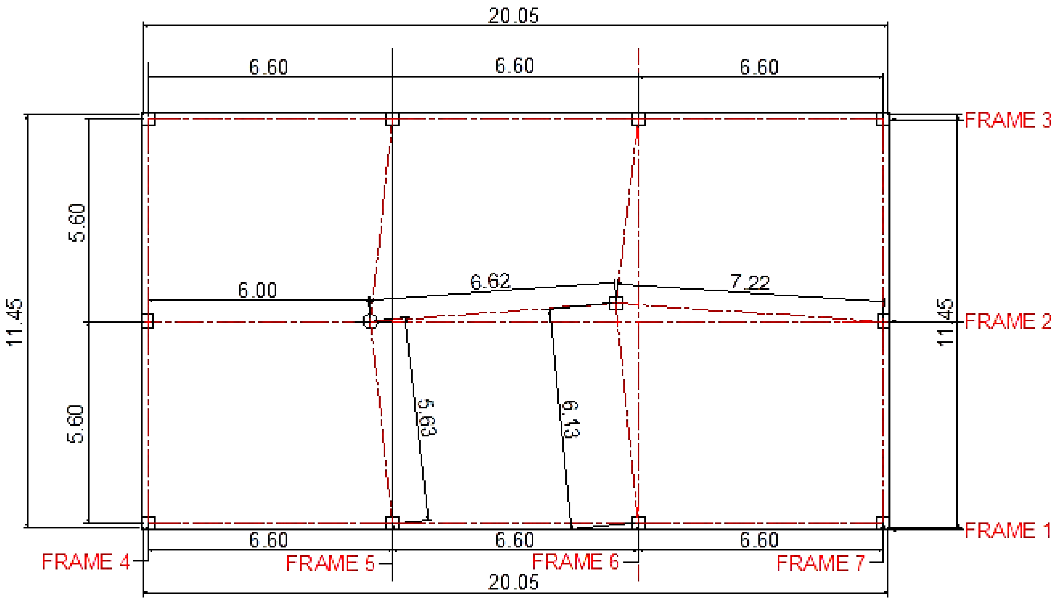


Figure 4.1: Drawing of slab

- **Solution:**

  Basic L/d ratio for continuous slab = 26, assume percentage of steel = 0.4

  So modification factor α = 1.4

  Required d = 6600/ (26 x 1.4) = 181.318 mm

  Required d = 7217/ (26 x 1.4) = 198.269 mm

  Provide total depth D = 225 mm

  Effective depth provided = d = 225 -20-(12/2) = 199 mm

  Effective depth in transverse direction = 199 - 12 = 187 mm

Table 4.1: Span length

Span ,A-B	X-DIRECTION	6.6 m
	Y-DIRECTION	5.616 m
Span ,B-C	X-DIRECTION	6.6 m
	Y-DIRECTION	5.8807 m
Span ,C-D	X-DIRECTION	6.6 m
	Y-DIRECTION	5.8647 m

- **Load calculation:**

Ultimate design load : W_{ud} = 1.5 (25 x 0.225 + 1) = 9.9375 kN/m^2

W_{uL} = 1.5 x 3 = 4.5 kN/m^2

$W_u = W_{ud} + W_{uL}$ = 14.4375 kN/m^2

Check for requirement of direct design method:

(1) Number of span is more than 3

(2) long span/short span = 7.217/5.13 < 2

(3) Columns are not staggered

(4) Successive spans in each direction are equal.

(5) Design live load = 3 kN/m^2 < 3 x D.L (= 3 x 9.9375)

The requirements (1), (3),(4) are not satisfied so direct design method cannot be used.

Using the Equivalent Frame Method, take the frame 1.

Fixed end moment for span:

Fixe end moment =

$$FEM = \frac{WL_2L_1^2}{12} \tag{4.1}$$

Fixed end moment, for span A-B = 147.162015 $kN.m$

Fixed end moment, for span B-C = 154.0982303 $kN.m$

Fixed end moment, for span C-D = 153.6789653 $kN.m$

Figure 4.2: Equivalent frame 1

Table 4.2: Distribution factor

JOINT	MEMBER	STIFFNESS	SUM	D.F
1	A-B	1615397.727	3044564.4	0.530584188
	A-E	1429166.667		0.469415812
2	B-A	1615397.727	4736101	0.341081775
	B-C	1691536.577		0.357158048
	B-F	1429166.667		0.301760177
3	C-B	1691536.577	4807637.5	0.351843616
	C-D	1686934.304		0.350886332
	C-G	1429166.667		0.297270053
4	D-C	1686934.304	3116101	0.541360604
	D-H	1429166.667		0.458639396

- **Stiffness calculation:** *Exterior column:*

  K_c, ext A-E = 1429166.67 mm^4

  K_c, ext D-H = 1429166.67 mm^4

  Interior column:

  K_c, int B-F = 1429167 mm^4

  K_c, int C-G =1429167 mm^4

(1) Exterior span A-B:

The critical section is at a distance = (d/2) = 93.5 m from face of column.

Rate of loading per meter = w $\times$ L_2 = 13.4375 $\times$ 2.808 = 40.5405 kN/m

Table 4.3: Moment distribution

	A-B	B-A	B-C	C-B	C-D	D-C
D.F	0.530	0.341	0.357	0.351	0.35	0.54
F.E.M	-147.162	147.162	-154.098	154.09	-153.67	153.67
B.M	78.081	2.365	2.477	-0.14	-0.14	-83.19
C.M	1.182	39.040	-0.073	1.23	-41.59	-0.07
B.M	-0.627	-13.290	-13.917	14.20	14.16	0.039
C.M	-6.645	-0.313	7.100	-6.95	0.019	7.080
B.M	3.525	-2.314	-2.423	2.44	2.43	-3.83
C.M	-1.157	1.762	1.220	-1.21	-1.91	1.21
B.M	0.614	-1.017	-1.065	1.100	1.09	-0.65
Final Moment	-72.187	173.394	-160.78	164.76	-179.62	74.25

L_2 =2.808 m

L_n = 6.6 - 0.350 =6.25 m

M_1 = 72.18 $kN.m$

M_2 = 173.39 $kN.m$

Reaction at left support = VL = ((w × l)/2-((M_2-M_1)/L)) = 118.44927 kN

Reaction at right support = VR = (W × L)-R_1 = 149.11803 kN

The critical section for the 'negative' design moment are at the column faces; the moments at the left end ($M_{u,l}$) and right end ($M_{u,R}$) are, accordingly given by :

At interior support critical section for flexure occurs at a distance = 175 mm

At exterior support critical section for flexure occurs at a distance = 175 mm

$M_{u,L}$ = -52.07983 $kN.m$

$M_{u,R}$ = -147.9197 $kN.m$

The location of maximum 'positive' moment is given by the location of zero shear, marked x from the left support ;

X=(V_L/W)= 3.6782 m

Positive moment at x distance = 100.85199 $kN.m$

Moment in longitudinal direction:

Total Negative moment at exterior support = -52.07983 $kN.m$

Total Negative moment at interior support = -147.9197 $kN.m$

Total Positive moment at mid span = 100.85199 $kN.m$

Distribution of longitudinal Panel moments into strip moments:

(A) Distribute moment in column strip

Exterior negative moment=-52.07983 $kN.m$

Interior negative moment= -110.9397 $kN.m$

Positive moment= 60.5111 $kN.m$

(B) Distribute moment in middle strip

Exterior negative moment = 0 $kN.m$

Interior negative moment = -36.9799 $kN.m$

Positive moment = 40.340 $kN.m$

(2) Interior span B-C:

The critical section is at a distance = (d/2) = 93.5 m from face of column.

Rate of loading per meter = w $\times$ L_2 = 13.4375 $\times$ 2.94035 = 42.45 kN/m

L_2 = 2.94035 m

L_n = 6.6-0.350 =6.25 m

M_1 = 160.780 $kN.m$

M_2 = 164.761 $kN.m$

Reaction at left support = V_L = ((w $\times$ l)/2-((M_2-M_1)/L)) = 139.486 kN

Reaction at right support = V_R = (W $\times$ L)-R_1 = 140.69237 kN

The critical section for the 'negative' design moment are at the column faces; the moments at the left end ($M_{u,l}$) and right end ($M_{u,R}$) are, accordingly given by :

At interior support critical section for flexure occurs at a distance = 175 mm

At exterior support critical section for flexure occurs at a distance = 175 mm

$M_{u,L}$ = -137.0207 $kN.m$

$M_{u,R}$ = -140.7899 $kN.m$

The location of maximum 'positive' moment is given by the location of zero shear, marked x from the left support ;

X=(V_L/W)= 3.314 m

Positive moment at x distance = 68.380 $kN.m$

Moment in longitudinal direction

Total Negative moment at interior support = -137.0207 $kN.m$

Total Negative moment at interior support = -140.7899 $kN.m$

Total Positive moment at mid span = 68.380 $kN.m$

Distribution of longitudinal Panel moments into strip moments :

(A) Distribute moment in column strip

Interior negative moment =-102.765 $kN.m$

Interior negative moment =-105.5924 $kN.m$

Positive moment =41.02844 $kN.m$

(B) Distribute moment in middle strip

Interior negative moment =-34.255 $kN.m$

Interior negative moment =-35.197 $kN.m$

Positive moment =27.3522 $kN.m$

(3) Exterior span C-D :

The critical section is at a distance n = (d/2) = 93.5 m from face of column.

Rate of loading per meter = w $\times$ L_2 = 13.4375 $\times$ 2.808 = 37.7325 kN/m

L_2 = 2.808 m

L_n = 6.6-0.350 =6.25 m

M_1= 72.18 $kN.m$

M_2 = 173.39 $kN.m$

Reaction at left support = V_L = ((w $\times$ l)/2-((M_2-M_1)/L)) = 118.44927 kN

Reaction at right support = V_R = (W $\times$ L)-R_1 = 149.11803 kN

The critical section for the 'negative' design moment are at the column faces; the moments at the left end ($M_{u,l}$)and right end ($M_{u,R}$) are, accordingly given by :

At interior support critical section for flexure occurs at a distance = 175 mm

At exterior support critical section for flexure occurs at a distance = 175 mm

$M_{u,L}$ = -41.68407 $kN.m$ $M_{u,R}$ = -134.6104 $kN.m$

The location of maximum 'positive' moment is given by the location of zero shear, marked x from the left support ;

X=(V_L/W)= 3.6782 m

Positive moment at x distance = 100.85199 $kN.m$

Moment in longitudinal direction

Total Negative moment at exterior support = -53.24861 $kN.m$

Total Negative moment at interior support = -153.0321 $kN.m$

Total Positive moment at mid span = 106.58781 $kN.m$

Distribution of longitudinal Panel moments into strip moments :

(A) Distribute moment in column strip

Exterior negative moment = -53.24861 $kN.m$

Interior negative moment = -114.7740 $kN.m$

Positive moment = 63.952 $kN.m$

(B) Distribute moment in middle strip

Exterior negative moment =0 $kN.m$

Interior negative moment = -38.258 $kN.m$

Positive moment = 42.6351 $kN.m$

Transfer of moments in columns:

The total unbalanced slab moments at the various supports are transmitted to the respective column. At each support, the unbalanced slab moment is shared by the column above and the column below in proportion to their relative stiffness.

$$M = \frac{0.08(W_d + 0.5W_l)L_2L_n^2 - W_d'L_2'L_n'}{1 + \frac{1}{\alpha_c}} \tag{4.2}$$

Where;

$$(\alpha_c) = \frac{\sum K_c}{K_s} \tag{4.3}$$

$(\sum K_c)$ = (4 E I_c / L_c) = [4 x E (350 x 350^3 /12)]/3500 = 5.09E x 106 $N.mm$
$K_{s,A-B}$ = (4 E I_s / L_s) =[4 x E x 2808 x 225^3/12]/6600 = 1.61E x 106 $N.mm$
$K_{s,B-C}$ = (4 E I_s / L_s) =[4 x E x 2940 x 225^3/12]/6600 = 1.69E x 106 $N.mm$
$K_{s,C-D}$ = (4 E I_s / L_s) =[4 x E x 2932 x 225^3/12]/6600 = 1.68E x 106 $N.mm$
α_c= 0.88472 M= 18.5361 $kN.m$

Check for depth for flexure :

Required d = $\sqrt{(52.0748 \times 106/(2.808 \times 1000))} = 136.18\ mm < 187\ mm$

Unbalanced moment transferred to column by flexure and shear:

When unbalanced gravity load, wind, earthquake, or other lateral loads cause transfer of bending moment between slab and column, the flexural stresses shall be investigated using a fraction,

$$\alpha = \frac{1}{1 + \frac{2}{3}\sqrt{\frac{\alpha_1}{\alpha_2}}} \tag{4.4}$$

Where,

α_1= overall dimension of the critical section for shear in the direction in which moment acts as shown in fig.4.3,

α_2= overall dimension of the critical section for shear transverse to the direction in which moment acts as shown in fig.4.3

α_1= 537 mm α_2= 537 mm α= 0.6

Moments transferred by flexure = 11.122 $kN.m$

Moments transferred by shear = 7.414 $kN.m$

- **Punching shear calculation**

  Column Dimensions = 350 mm × 350 mm

  Effective slab depth (d) = 187.00 mm

  Critical Section Distance factor = 0.5

Table 4.4: Along longer side C.S moment(IS,with staggered column)

	Exterior span A-B			Interior span B-C			Exterior span C-D		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
1	-52.1	60.5	-110.9	-102.8	41.0	-105.6	-114.8	64.0	-53.2
2	-28.8	116.6	-191.4	-185.1	62.9	-243.8	-266.0	169.7	-64.4
3	-53.1	61.3	-107.8	-97.5	36.9	-94.9	-104.0	57.8	-51.8

Table 4.5: Along shorter side C.S moment(IS,with staggered column)

	Exterior span A-B			Interior span B-C		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
4	-29.4	48.7	-94.6	-94.6	48.7	-29.4
5	-25.1	109.5	-214.8	-214.8	109.4	-25.0
6	-42.0	144.6	-235.0	-225.2	86.2	-8.8
7	-29.5	54.1	-104.6	-104.6	54.1	-29.5

Table 4.6: Along longer side M.S moment(IS,with staggered column)

	Exterior span A-B			Interior span B-C			Exterior span C-D		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
1	0	79.2	-68.9	-65.1	48.3	-75.8	-82.6	99.2	0
2	0	79.7	-67.8	-63.4	45.5	-72.3	-79.0	95.1	0

Table 4.7: Along shorter side M.S moment(IS,with staggered column)

	Exterior span A-B			Interior span B-C		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
3	0	69.0	-67.3	-67.3	69.0	0
4	0	84.7	-75.0	-73.3	65.2	0
5	0	84.2	-74.0	-72.4	64.8	0

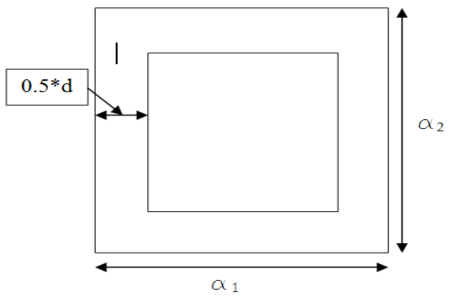


Figure 4.3: Dimension of Critical section for shear

Critical Section Dimensions (A_x,A_y) = 443.50 mm

Unbalanced Moment on column M_o(Including pattern load) = 18.54 $kN.m$

panel area = 9.24 m^2

Shear force on critical section (V_u) = 130.56 kN $\alpha = 0.60$

Unbalanced Moment transferred by shear = 7.41 $kN.m$

Perimeter of critical section (P_c) = 887.00 mm

Area of critical section (A_c) = 165869.00 mm^2

Distance of c.g. of column from center of critical section = 110.88 mm

Polar moment of inertia (J_x,J_y) = 3640130837.46

Punching Shear stress (τ_v) = 1.17

Permissible punching shear stress calculation

$K_S = 1$ $\tau_c = 1.37$ Mpa

Permissible Punching Shear stress (τ_c) = 1.37 Mpa

$\therefore$ $\tau_v < \tau_c$ No shear reinforcement required.

- **Deflection calculation**

(a) Deflection check by allowable L/d ratio approach

mid span maximum moment = 195.06 $kN.m$

pt req % = 0.2566

$A_{st}req$ = 510.62 per meter $A_{st}req$ provided = 565.2 per meter

Table 4.8: Punching shear result(IS,with staggered column)($v_{manual}/v_{permissible}$)

column number	punching ratio	V_u	M_{ux}	M_{uy}	depth	perimeter	location	result
1	0.85618	130.56	18.54	13.08	187.00	887.00	Corner	safe
2	0.8802	264.89	18.14	8.13	187.00	1424.00	Edge	safe
3	0.95882	288.59	19.77	8.83	187.00	1424.00	Edge	safe
4	0.85618	130.56	18.54	13.08	187.00	887.00	Corner	safe
5	0.84405	251.34	7.31	14.28	187.00	1374.00	Edge	safe
6	1.00819	508.58	11.38	7.54	187.00	2130.16	Interior	failed
7	1.10607	558.01	15.59	7.63	187.00	2148.00	Interior	failed
8	0.98805	288.41	12.40	15.55	187.00	1374.00	Edge	safe
9	0.85618	130.56	18.54	13.08	187.00	887.00	Corner	safe
10	0.8802	264.89	18.14	8.13	187.00	1424.00	Edge	safe
11	0.79928	241.21	17.33	6.05	187.00	1424.00	Edge	safe
12	0.85618	130.56	18.54	13.08	187.00	887.00	Corner	safe

f_{ck} = 30 N/mm^2 f_y = 415 N/mm^2

f_s = 0.58 × f_y × (A_{st} requi / A_{st} provided) =217.456

value of modification factor as per IS 456-2000 =1.8

effective length of span = 6600 mm

basic value span to effective depth ratio= 26

effective depth requi. = 141.03 mm

effective depth provided = 187 mm

$\therefore$ provide depth > required

(b) Deflection check

total load = 9.625 kN/m^2

permanent load = D.L + (0.2*L.L) = 7.225 kN/m^2

temporary load = 0.8*L.L kN/m^2

M_o(perm) = 74.0595 $kN.m$

$f_{cr} = 0.7\sqrt{f_{ck}}$ = 3.834057903 N/mm^2

I_{gr} = (b×d^3)/12 = 949218750 mm^4

y_t = (225/2) =112.5 mm

$M_r = f_{cr}\ I_{gr}\ /\ y_t$ = 32349863.55 $N.mm$ = 32.34986355 $kN.m$

- ***Short-term deflection***

  E_c = 5000 * $\sqrt{f_{ck}}$ =27386.12788 N/mm^2

  E_s = 200000 N/mm^2

  m= (E_s /E_c) = 7.302967433

- ***Depth of neutral axis***

  bx_2 /2 = m Ast (d-x) A_{st} = 1005.30 mm^2

  b= 1000 mm x = 47.207 mm

  lever arm = z = d- (x/3) = 183.2643333 mm

  I_r= bx^3 /3 + m A_{st} $(d - x)^2$ = 204.2288964 mm^4

  M_{total} = 40.34 $kN.m$

- ***For total load***

  c= 1.2 - M_r/M * Z/d *(1- X/d)*b_W/b   C= 0.636

  I_{effe} = I_r /C = 320774876.4 mm^4

  I_{effec} = 320.7748764 $\times$ 10^6 mm^4

- ***For permanent load***

  M_o(perm) = 28.2817 $kN.m$

  C= 0.396

  I_{effe} = I_r /C = 515090249.2 mm^4

  I_{effec} = 515.0902492 $\times$ 10^6 mm^4

  $a_{i(perm)}$ = 12.65 mm $a_{i(temp)}$ = 4.20 mm

  $a_{i(total)}$ = 16.85 mm

- ***Long-term deflection due to shrinkage***

  $\alpha_{cs} = K_3\ \psi_{cs}\ L^2$

  $K_3 = 0.063$ for continuous at both ends (1/16)

  $\psi_{cs} = K_4\ \varepsilon_{cs}$/D

  $\varepsilon_{cs} = 0.0003$ pt = 0.404

  $K_4 = 0.72\sqrt{pt} = 0.457$ ψ_{cs}= 5.50862E-07

  ε_{cs}= 1.5117 mm

- ***Long-term deflection due to creep***

  $E_{ce} = E_c\ /(1+\ \theta) = 10533.12611\ N/mm^2$

  $\theta = 1.6$ m = 18.98771533

  depth of neutral axis

  x= 64.16 mm $I_r = 435.102\ mm^4$

  Z = d- (x/3) = 177.613 mm

  for creep , M = M_o (perm) = 28.281 $kN.m$

  C= 0.50824

  I_{effe}= 856095993 $I_{effec} = 856.0 \times 10^6$

  $a_{i(perm)} = 19.79\ mm\ a_{cc(perm)} = 7.141\ mm$

- ***Total deflection***

  $a_t = a_{icc}$(perm) + a_{cs} + a_i(temp) = 19.79 + 1.51 + 4.20 = 25.51 mm

  a_t =a_i(total) + a_{cc}(perm) + a_{cs} = 16.85 + 7.14 + 1.51 = 25.51 mm

  allowable deflection =L /250 = 26.4 mm

4.2.2 Without Staggered Column

Table 4.9: Along longer side C.S moment(IS,without staggered column)

	Exterior span A-B			Interior span B-C			Exterior span C-D		
FRAME	M_{ul}^{-}	M_{u}^{+}	M_{ur}^{-}	M_{ul}^{-}	M_{u}^{+}	M_{ur}^{-}	M_{ul}^{-}	M_{u}^{+}	M_{ur}^{-}
1	-52.57	60.71	-108.95	-99.70	38.67	-99.70	-108.95	60.71	-52.57
2	-44.90	141.59	-225.02	-211.84	68.10	-210.48	-225.10	141.42	-44.82
3	-52.57	60.71	-108.95	-99.70	38.67	-99.70	-108.95	60.71	-52.57

Table 4.10: Along shorter side C.S moment(IS,without staggered column)

	Exterior span A-B			Interior span B-C		
FRAME	M_{ul}^{-}	M_{u}^{+}	M_{ur}^{-}	M_{ul}^{-}	M_{u}^{+}	M_{ur}^{-}
4	-29.48	51.36	-99.53	-99.53	51.36	-29.48
5	-23.89	111.06	-217.48	-217.48	111.06	-23.89
6	-23.89	111.06	-216.98	-216.98	111.06	-23.89
7	-29.48	51.36	-99.53	-99.53	51.36	-29.48

Table 4.11: Along longer side M.S moment(IS,without staggered column)

	Exterior span A-B			Interior span B-C			Exterior span C-D		
FRAME	M_{ul}^{-}	M_{u}^{+}	M_{ur}^{-}	M_{ul}^{-}	M_{u}^{+}	M_{ur}^{-}	M_{ul}^{-}	M_{u}^{+}	M_{ur}^{-}
1	0	87.67	-73.82	-68.54	48.48	-68.31	-73.83	87.62	0
2	0	87.67	-73.82	-68.54	48.48	-68.31	-73.83	87.62	0

Table 4.12: Along shorter side M.S moment(IS,without staggered column)

	Exterior span A-B			Interior span B-C		
FRAME	M_{ul}^{-}	M_{u}^{+}	M_{ur}^{-}	M_{ul}^{-}	M_{u}^{+}	M_{ur}^{-}
3	0	71.26	-69.42	-69.42	71.26	0
4	0	74.04	-72.41	-72.41	74.04	0
5	0	71.26	-69.34	-69.34	71.26	0

Table 4.13: Punching shear result(IS,without staggered column)($v_{manual}/v_{permissible}$)

column number	punching ratio	V_u	M_{ux}	M_{uy}	depth	perimeter	location	result
1	0.85204	130.56	18.51	12.64	187.00	887.00	Corner	safe
2	0.71851	205.62	18.51	7.87	187.00	1424.00	Edge	safe
3	0.87721	263.37	18.51	7.93	187.00	1424.00	Edge	safe
4	0.85204	130.56	18.51	12.64	187.00	887.00	Corner	safe
5	0.72985	205.72	9.54	14.59	187.00	1374.00	Edge	safe
6	1.04542	529.52	11.32	7.30	187.00	2130.16	Interior	failed
7	1.04126	529.45	12.02	7.93	187.00	2148.00	Interior	failed
8	0.72985	205.72	9.54	14.59	187.00	1374.00	Edge	safe
9	0.85204	130.56	18.51	12.64	187.00	887.00	Corner	safe
10	0.71851	205.62	18.51	7.87	187.00	1424.00	Edge	safe
11	0.87721	263.37	18.51	7.93	187.00	1424.00	Edge	safe
12	0.85204	130.56	18.51	12.64	187.00	887.00	Corner	safe

4.3 Design Example as per ACI 318-08

4.3.1 With Staggered Column

Design the flat slab with staggered column shown in figure. Using equivalent frame method. It is subjected to live load of 3 kN/m^2 and floor finish of 1 kN/m^2. The grade of steel used is Fe 415.Floor to floor height of column 3.5m.

Given:

L.L = 3 kN/m^2	clear cover = 20 mm
fck = 30 N/mm^2	fy =415 N/mm^2
Column diameter = 390 mm	Floor to floor height of column = 3.5 m

Column		Size, mm	Area,mm^2
Square	x- direction	350 mm	122500
	y-direction	350 mm	
Circular	Diameter	390 mm	119398.5
rectangular	x-direction	300 mm	120000
	y-direction	400 mm	

- **Solution**

  Minimum thickness of slab D = (L_n/30) for exterior panel

  D = (L_n/33) for interior panel

  Required D = 246.06 in / 30 = 8.202 in = 208.33 mm

  Provide total depth D = 225 mm

  Effective depth provided = d = 225 -20-(12/2) = 199 mm

  Effective depth in transverse direction = 200 - 12 = 187 mm

- **Load calculation**

  Ultimate design load : W_{ud} = 1.2 (25 x 0.225 + 1) = 7.95 kN/m^2

  W_{uL} = 1.6 x 3 = 4.8 kN/m^2

  $W_u = W_{ud} + W_{uL}$ = 12.75 kN/m^2

Using the Equivalent Frame Method, take the frame 1.

(1) Exterior span A-B:

Relative stiffness parameters of equivalent frame:

(A) Column stiffness:

I_c = 1.25E x 109 mm^4

H=height of column =3.5 m

H_c=3.275 m

t_a/t_b =1

The stiffness and carry-over factors are:

K_{AB}=4.52 C_{AB}=0.54

K_C= E_C *I_C /H =1614958.33 E_C

E_c= modulus of elasticity of concrete

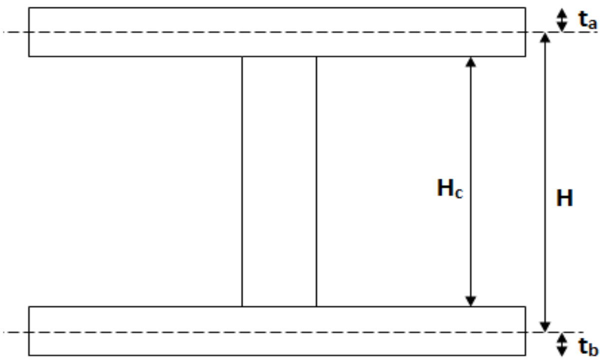


Figure 4.4: Stiffness parameters

(B) Torsional member stiffness, K_t :

$$K_t = \frac{(\sum 9E_cC)}{L_2[1-(C_2/L_2)]^3} \tag{4.5}$$

where,

$$C = \sum(1-0.63\frac{x}{y})(\frac{x^3y}{3}) \tag{4.6}$$

x = $225mm$	y = 350 mm
L_2 = 2800 mm	K_t = 3793775.51 E_c
C = 790699218.8 mm^4	K_t = 3793775.51 E_c

(c) Equivalent column stiffness,K_{ec}

$$K_{ec} = \frac{\sum K_c \times \sum K_t}{\sum K_c + \sum K_t} \tag{4.7}$$

$\sum K_c = 1614958.33\ E_C$

$\sum K_t = 3793775.51\ E_c$

$K_{ec} = 1132758.526$

(D) Slab stiffness K_s and fixed-end moment coefficients

referring to fig.for the slab geometry

C_{N1}= 350 mm C_{F1}= 350 mm

C_{N2}= 350 mm C_{F2}= 350 mm

C_{F1}= C_{N1} C_{F2}= C_{N2}

C_{N1}/L_1= 0.05 C_{N2}/L_2= 0.125

- stiffness factor K_{NF} = 4.18
- carry-over factor C_{NF} = 0.51
- FEM coefficient m_{NF} = 0.0847

Slab stiffness:

$$K_{sb} = K_{nf}\frac{E_{cs} \times I_s}{L_1} \tag{4.8}$$

I_s = 2.65E x 10^9 mm^4

K_s = 1683281.25 E_c

The slab fixed-end moments are:

$$FEM_{NF} = \sum m_{NF} w L_1^2 \tag{4.9}$$

FEM_{NFA-B} = 132.092 $kN.m$

calculating by same method as describe above

FEM_{NFB-C} = 138.318 $kN.m$

FEM_{NFC-D} = 137.942 $kN.m$

(1) Exterior span A-B:

The critical section is at a distance = (d/2) = 93.5 m from face of column.

Rate of loading per meter = w × L_2 = 12.75 × 2.808 = 35.802 kN/m

L_2 = 2.808 m

L_n = 6.6 - 0.350 =6.25 m

Table 4.14: Moment distribution

	A-B	B-A	B-C	C-B	C-D	D-C
D.F	0.60	0.37	0.39	0.38	0.38	0.61
COF	0.51	0.51	0.51	0.51	0.51	0.51
FEM	-132.09	132.09	-138.32	138.32	-137.94	137.94
B.M	79.09	2.29	2.41	-0.14	-0.14	-84.63
C.O	1.15	39.54	-0.07	1.21	-42.32	-0.07
B.M	-0.69	-14.53	-15.29	15.66	15.62	0.04
C.O	-7.26	-0.34	7.83	-7.64	0.02	7.81
B.M	4.35	-2.76	-2.90	2.90	2.89	-4.79
C.O	-1.38	2.17	1.45	-1.45	-2.40	1.45
B.M	0.83	-1.33	-1.40	1.47	1.46	-0.89
C.O	-0.67	0.41	0.73	-0.70	-0.44	0.73
B.M	0.40	-0.42	-0.44	0.44	0.44	-0.45
Final moment	-56.28	157.13	-146.00	150.05	-162.81	57.14

M_1 = 56.280 $kN.m$

M_2 = 157.133 $kN.m$

Reaction at left support = VL = ((w × l)/2-((M_2-M_1)/L)) = 102.865 kN

Reaction at right support = VR = (W × L)-R_1 = 133.427 kN

The critical section for the 'negative' design moment are at the column faces; the moments at the left end ($M_{u,l}$) and right end ($M_{u,r}$) are, accordingly given by :

At interior support critical section for flexure occurs at a distance = 175 mm

At exterior support critical section for flexure occurs at a distance = 175 mm

$M_{u,L}$ = -56.280 $kN.m$

$M_{u,R}$ = -157.133 $kN.m$

The location of maximum 'positive' moment is given by the location of zero shear, marked x from the left support ;

X=(V_L/W)= 3.726 m

Final results:

Table 4.15: Along longer side C.S moment(ACI,with staggered column)

	Exterior span A-B			Interior span B-C			Exterior span C-D		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
1	-38.8	54.9	-100.7	-93.8	33.7	-96.7	-104.2	58.2	-38.9
2	-7.9	109.0	-169.6	-166.6	49.4	-227.9	-238.6	159.0	-27.0
3	-39.7	55.7	-97.7	-89.2	30.2	-86.5	-94.3	52.3	-39.5

Table 4.16: Along shorter side C.S moment(ACI,with staggered column)

	Exterior span A-B			Interior span B-C		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
4	-19.0	44.0	-87.8	-87.8	44.0	-19.0
5	-0.1	101.6	-199.4	-199.4	101.6	-0.1
6	-5.9	136.0	-215.7	-212.4	78.2	7.9
7	-17.6	49.1	-97.5	-97.5	49.1	-17.6

Table 4.17: Along longer side M.S moment(ACI,with staggered column)

	Exterior span A-B			Interior span B-C			Exterior span C-D		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
1	0	72.9	-61.9	-59.0	38.9	-70.2	-74.5	91.8	0
2	0	73.4	-60.8	-57.5	36.6	-66.8	-71.2	87.9	0

Table 4.18: Along shorter side M.S moment(ACI,with staggered column)

	Exterior span A-B			Interior span B-C		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
3	0	63.2	-62.5	-62.5	63.2	0
4	0	79.2	-69.2	-68.6	59.9	0
5	0	78.0	-68.4	-67.9	58.8	0

Table 4.19: Punching shear result(ACI,with staggered column)($v_{manual}/v_{permissible}$)

column number	punching ratio	V_u	M_{ux}	M_{uy}	depth	location	result
1	0.596	102.87	38.83	18.99	205	Corner	safe
2	0.875	256.53	100.75	0.06	205	Edge	safe
3	0.93	263.72	104.24	5.95	205	Edge	safe
4	0.594	107.37	38.92	17.59	205	Corner	safe
5	0.876	175.50	7.92	87.85	205	Edge	safe
6	2.014	473.07	169.62	199.39	205	Interior	failed
7	2.401	546.67	238.56	215.71	205	Interior	failed
8	1.107	215.26	27.01	97.47	205	Edge	failed
9	0.604	103.65	39.67	18.99	205	Corner	safe
10	0.846	246.46	97.72	0.06	205	Edge	safe
11	0.855	239.38	94.33	7.93	205	Edge	safe
12	0.585	99.11	39.48	17.59	205	Corner	safe

- ***Punching shear data***

  Slab Thickness h = 225 mm

  Column Breadth C1 = 350 mm

  Column Depth C2 = 350 mm

  Min. cover to centroid of rebar = 20 mm

  Max. cover to centeroid of rebar = 20 mm

  Effective depth d = h - avg.cover = 205 mm

here interior column no(6.7) & edge column no(8) are failed in punching shear. so additional shear reinforcement are provided

4.3.2 Without Staggered Column

Table 4.20: Along longer side C.S moment(ACI,without staggered column)

	Exterior span A-B			Interior span B-C			Exterior span C-D		
FRAME	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$
1	-39.29	55.1019	-98.86	-91.1	31.71	-91.1	-98.86	55.1	-39.29
2	-16.47	132.406	-200.9	-194.2	54.06	-193.9	-201	132.3	-16.43
3	-39.29	55.1019	-98.86	-91.1	31.71	-91.1	-98.86	55.1	-39.29

Table 4.21: Along shorter side C.S moment(ACI,without staggered column)

	Exterior span A-B			Interior span B-C		
FRAME	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$
4	-18.32	46.5055	-92.59	-92.59	46.51	-18.32
5	1.16	103.065	-202	-202	103.1	1.16
6	1.144	103.114	-201.5	-201.5	103.1	1.144
7	-19.6	41.7	-88.1	-88.1	41.7	-19.6

Table 4.22: Along longer side M.S moment(ACI,without staggered column)

	Exterior span A-B			Interior span B-C			Exterior span C-D		
FRAME	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$
1	0	80.9	-66.4	-62.7	39.2	-62.7	-66.5	80.8	0
2	0	80.9	-66.4	-62.7	39.2	-62.7	-66.5	80.8	0

Table 4.23: Along shorter side M.S moment(ACI,without staggered column)

	Exterior span A-B			Interior span B-C		
FRAME	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$
3	0	65.4	-64.5	-64.5	65.4	0
4	0	68.7	-67.2	-67.2	68.7	0
5	0	65.4	-64.4	-64.4	65.4	0

Table 4.24: Punching shear result(ACI,without staggered column)($v_{manual}/v_{permissible}$)

column number	punching ratio	V_u	M_{ux}	M_{uy}	depth	location	result
1	0.593	103.01	39.29	18.09	205	Corner	safe
2	0.862	250.42	98.86	1.16	205	Edge	safe
3	0.862	250.42	98.86	1.14	205	Edge	safe
4	0.593	103.01	39.29	18.32	205	Corner	safe
5	0.984	194.63	16.43	92.59	205	Edge	safe
6	2.191	512.14	200.87	201.97	205	Interior	failed
7	2.173	512.22	201.05	201.50	205	Interior	failed
8	0.984	194.63	16.43	92.59	205	Edge	safe
9	0.593	103.01	39.29	18.32	205	Corner	safe
10	0.862	250.42	98.86	1.16	205	Edge	safe
11	0.862	250.42	98.86	1.14	205	Edge	safe
12	0.593	103.01	39.29	18.32	205	Corner	safe

- **Deflection check**

Modulus of rupture, modulus of elasticity, modular ratio:

$f_r = 7.5\sqrt{f_c}$

$\lambda = 1 \qquad f_c = 30\ MPA = 4351.14\ psi$

$f_r = 7.5\sqrt{f_c} = 3.410994268\ MPA = 494.7237866\ psi$

$E_{cs} = w_c^{1.5} * 33 * f_c$

$w_c = 150$

$E_{cs} = 27572485.67\ MPA = 3999008.774\ psi$

$E_{cc} = 57000\sqrt{f_c} = 25923763.88\ kN/m^2 = 3759900.778\ psi$

$E_s = 199949200\ kN/m^2 = 29000000\ psi$

n = $(E_s\ /E_{cs}) = 7.712$

Service load moments and cracking moment:

D.L= 5.625kN/m^2 = 117.481203 psf

F.L= 1kN/m^2 = 20.8855472 psf L.L= 3kN/m^2 = 62.6566416 psf

0.4 × l.l =1.2 kN/m^2 = 25.06265664 psf

(M_o)d =(w_d * l_2 * l_n^2)/8 = 181.1523438 $kN.m$ = 133.6118353 $ft - kips$

(M_o)d+l =(w_d * l_2 * l_n^2)/8 = 263.1835938 $kN.m$ = 194.1153079 $ft - kips$

(M_o)sus =(w_d * l_2 * l_n^2)/8 = 213.9648438 $kN.m$ = 157.8132243 $ft - kips$

The gross moment of inertia of a panel, referred to as the total equivalent frame moment of inertia is:

$I_{frame} = l_s h^3/12$

I_s= 0.005315625 m^4 = 12770.84013 in^4

For this case, the moment of inertia of a column strip or a middle strip is equal to half of the moment of inertia of the total equivalent frame:

I_g column strip = 0.001328906 m^4 = 3192.7100 in^4

b= 1.4 m

The cracking moment for either a column strip or a middle strip is obtained from the standard flexure formula based on the uncracked section as follows:

(M_{cr})c/2 = (M_{cr})m/2 = $f_r I_g/y_t$ = 20.14 $kN.m$ = 14.859 $ft - kips$

Effective moments of inertia:

The cracked section moment of inertia is, therefore, only required for the column strips in the negative moment zones. Formulas for computation of the cracked section moment of inertia are:

B =(b/A_s)= 1.473374763 1/in A_s = 9.700396247 $sq.in$

k_d= $\sqrt{(2d * b + 1) - 1/B}$ =0.069976905m = 2.754996254 in

I_{cr} =b$(kd)^3$/3 + nA_s $(d - kd)^2$ = 0.00129528 m^4 = 3111.92204 in^4

To obtain an equivalent moment of inertia for the cracked location, apply the Branson modifi cation to the moments of inertia for cracked and uncracked sections. The approximate moment of inertia in the cracked sections is given by the general formula in ACI 318.The ratios of the cracking moment to dead load plus live load and sustained load moments are found as follows:

For dead load plus live load:

M_{cr} = 40.29245084 $kN.m$ = 29.71818063 $ft-kips$

column moment M_a = 76.0556 $kN.m$ = 56.09572045 $ft-kips$

(M_{cr}/M_a)= 0.529776

$(M_{cr}/M_a)^3$ =0.1486885

and for the sustained load case (dead load plus 40% live load):

M_{cr}= 40.29245084 $kN.m$ = 29.71818063 $ft-kips$

column moment M_a= 61.8322 $kN.m$ = 45.6050811 $ft-kips$

(M_{cr}/M_a)= 0.6516418

$(M_{cr}/M_a)^3$ = 0.27671

The equivalent moment of inertia for the two cases are now computed

For dead load plus live load:

I_e = (0.134)I_g + (1-0.134)(I_{cr}) = 0.00130028 m^4 = 3123.934288 in^4

For sustained load (dead load + 40% live load):

I_e = (0.249)I_g + (1-0.249)(I_{cr}) = 0.001304585 m^4 = 3134.276994 in^4

Finally, the equivalent moment of inertia for the uncracked sections is just the moment of inertia of the gross section, I_g.To obtain an average moment of inertia for calculation of deflection, the "end and "midspan

For dead load plus live load:

$A_{vg.}$ I_e = 0.85(I_g) + 0.15(I_e)] = 0.001324612 m^4 = 3182.393674 in^4

For sustained load (dead load + 40% live load):

$A_{vg.}$ I_e = 0.85(I_g) + 0.15(I_e)] = 0.001325258 m^4 = 3183.94508 in^4

To obtain the equivalent moment of inertia for the equivalent frame, which consists of a column and a middle strip, add the average moments of inertia for the respective strips. For the middle strips, the moment of inertia is that of the gross section, I_g, and for the column strips, the average values computed above are used:

For dead load only:

(I_e)frame = I_g+ I_g = 0.002657813 m^4 = 6385.420073 in^4

For dead load plus live load:

(I_e)frame = I_g+ $A_{vg.}I_e$ = 0.002653519 m^4 = 6375.103711 in^4

For dead load plus 40% live load:

(I_e)frame = I_g + $A_{vg.}I_e$ = 0.002654164 m^4 = 6376.655117 in^4

Flexural stiffness (Kec) of an exterior equivalent column:

K_b = 0 (no beams)

Fixed Δframe = w l_2 l^4 / 384 E_{cs} I_{frame}

(Fixed Δframe)d = 0.002660732 m = 0.104753214 in

(Fixed Δframe)d+l = 0.003871847 m = 0.152434907 in

(Fixed Δframe)sus = 0.003146995 m = 0.123897449 in

Fixed Δc,m = (LDF)c,m (Fixed Δframe) (I_{frame}/I_c,m)

For the column strip:

LDFc = 1/2 [1/2 (M_{int} + M_{ext}) + M_+] = 0.7375

For the middle strip:

LDFm = 1- LDFc = 0.2625

(FixedΔc)d = 0.003924579 m = 0.154510991 in

(Fixed Δc)d+l = 0.00572023 m = 0.225205922 in

(Fixed Δc)l = 0.001795651 m = 0.070694932 in

(Fixed Δc)sus = 0.004648207 m = 0.183000278 in

(Fixed Δm)d = 0.001396884 m = 0.054995437 in

(Fixed Δm)d+l = 0.002036014 m = 0.08015804 in

(Fixed Δm)l = 0.00063913 m = 0.025162603 in

(Fixed Δm)sus = 0.001654447 m = 0.065135692 in

The net moments on a corner column for the three loading cases are:

(M_{net})d = 29.2439 $kN.m$ = 21.56918937 $ft - kips$

(M_{net})d+l = 42.486 $kN.m$ = 31.33605913 $ft - kips$

(M_{net})sus = 34.54 $kN.m$ = 25.47539148 $ft - kips$

For both column and middle strips,

avg.K_{ec} = 18860 $ft - kips/rad$

End θd = (M_{net})d/avg.K_{ec} = 0.001143647 rad

End θd+l = 0.001661509 rad

End θsus = 0.001350763 rad

$\Delta\theta$ = (Endθ) (l/8) (I_g /I_e)frame

$(\Delta\theta)$d = 0.000943509 m = 0.0371 in

$(\Delta\theta)$d+l = 0.001375188 m = 0.054 in

$(\Delta\theta)$l = 0.000431679 m = 0.016 in

$(\Delta\theta)$sus = 0.00111438 m = 0.0438 in

The deflections due to rotation calculated above are for column strips. The deflections due to end rotations for the middle strips will be assumed to be equal to that in the column strips.Therefore, the strip deflections are calculated by the general relationship:

Δc,m = Fixed Δc,m + $(\Delta\theta)$ $(\Delta$c)d = 0.004868088 m 0.191657017 in

$(\Delta$m)d = 0.002340393 m 0.092141464 in

$(\Delta$c)l = 0.002227331 m 0.087690182 in

$(\Delta$m)l = 0.001070809 m 0.042157853 in

$(\Delta$c)sus =0.005762587 m 0.226873488 in

$(\Delta$m)sus = 0.002768826m 0.109008902 in

Δ = Δcx + Δmy = midpanel deflection of corner panel

$(\Delta$i)d = 0.007208481 m 0.283798481 in

$(\Delta$i)l = 0.00329814 m 0.129848036 in

$(\Delta$i)sus = 0.008531413 m 0.33588239 in

The long term deflections may be calculated as per ACI-318 (Note: $\rho = 0$):

For dead load only:

$(\Delta$cp+sh)d = 2.0 * $(\Delta$i)d = 0.014416963 m = 0.567596963 in

For sustained load (dead load + 40% live load)

$(\Delta$cp+sh)sus = 2.0 $(\Delta$i)sus = 0.017062825 m = 0.671764781 in

The long term deflection due to sustained load plus live load is calculated as:

$(\Delta$cp+sh)sus + $(\Delta$i)l = 0.020360966 m = 0.801612816 in

These computed defl ections are compared with the code allowable deflections as follows:

Flat roofs not supporting and not attached to nonstructural elements likely to be damaged by large deflections:

$(\Delta$i)l $\leq$ (ln or l) /180 = $(\Delta$i)l = 0.129848036 in = 0.00329814 mm $\therefore$ safe

(ln or l) /180 = 1.443569556 in = 0.036666667 mm

Floors not supporting and not attached to nonstructural elements likely to be damaged by large deflections:

$(\Delta$i)l $\leq$ (ln or l) / 360 = $(\Delta$i)l = 0.129848036 in = 0.00329814 mm $\therefore$ safe

(ln or l) /360 = 0.721784778 in = 0.018333333 mm

Roof or floor construction supporting or attached to non-structural elements not likely to be damaged by large deflections:

Δ(cp+sh) + $(\Delta$i)l $\leq$ (ln or l) / 240 = Δ(cp+sh) + $(\Delta$i)l = 0.801612816 in = 0.020360966 mm $\therefore$ safe (ln or l) / 240 = 1.082677167 in = 0.0275 mm All computed deflections are found to be satisfactory in all four categories.

4.4 Design Example as per BS 8110-1997

4.4.1 With Staggered Column

Design the flat slab with staggered column shown in figure. Using equivalent frame method. It is subjected to live load of 3 kN/m^2 and floor finish of 1 kN/m^2. The grade of steel used is Fe 415.Floor to floor height of column 3.5m.

- **Solution**

  Basic L/d ratio for continuous slab = 26 Provide total depth D = 225 mm

  Effective depth provided = d = 225 -20-(12/2) = 199 mm

  Effective depth in transverse direction = 199 - 12 = 187 mm

- **Load calculation**

  Ultimate design load : W_{ud} = 1.4 (25 x 0.225 + 1) = 9.275 kN/m^2

  W_{uL} = 1.6 x 3 = 4.8 kN/m^2

  $W_u = W_{ud} + W_{uL}$ = 14.075 kN/m^2

  Fixed end moment for span:

  Fixe end moment =

  $$FEM = \frac{WL_2L_1^2}{12} \tag{4.10}$$

  Fixed end moment, for span A-B = 143.4670 $kN.m$

  Fixed end moment, for span B-C = 150.2290 $kN.m$

  Fixed end moment, for span C-D = 149.8203 $kN.m$

  Effective diameter of column ,h_c =((4A)/3.14)$^{0.5}$ < 0.25 L_y

  column square = (350×350)= 395.0328536 mm < 1404mm

  column rectangular = (300×400)= 390.9811275 mm < 1404 mm

  column circular = 390dia = 390 mm < 1404 mm

  or(1/4)*L_y = 1404 mm

- **Check moment connection to edge column**

  $M_{t,max}$ = 0.15* b_e * d^2 *F_{cu}

  b_e = C_x +C_y = 700 mm

  C_x= 350 mm C_y= 350 mm

  d= 199 mm F_{cu} = 30 N/mm^2

  $M_{t,max}$= 0.15 * b_e * d^2 *F_{cu} =124.74315 $kN.m$

  frame 1 ,joint 1 = 124.74315 $kN.m >$ 35.18758349 ∴ok

  frame 1 ,joint 2 = 124.74315 $kN.m >$ 87.55828756 ∴ok

  frame 1 ,joint 4 = 124.74315 $kN.m >$ 36.1954568 ∴ok

  frame 3 ,joint 1 = 124.74315 $kN.m >$ 35.75314932 ∴ok

  frame 3 ,joint 2 = 124.74315 $kN.m >$ 82.39220967 ∴ok

  frame 3 ,joint 3 = 124.74315 $kN.m >$ 79.42373364 ∴ok

  frame 3 ,joint 4 = 124.74315 $kN.m >$ 34.70505742 ∴ok

  frame 2 ,joint 1 = 124.74315 $kN.m >$ 28.40078463 ∴ok

  frame 2,joint 4 = 124.74315 $kN.m >$ 49.20614368 ∴ok

  $M_{t,max}$ > half design moment hence ∴ok

- **Limitation of negative design moment**

  M' = (nL_2/8)*(L_1-(2h_c)/3)^2

  n= loading per unit area on slab = 14.075 kN/m^2

  $h_c/2$ square column = 197.5164268 mm

  $h_c/2$ circular column = 195 mm

  $h_c/2$ rectangular column = 195.4905637 mm

  M' span A-B = 206.6992183 $kN.m$

  M' span B-C = 216.441612 $kN.m$

$M^{'}$ span C-D = 215.8527253 $kN.m$

Average moment = 193.037 $kN.m$

revised moment = 13.6614 $kN.m$

- **Punching shear check**

  Column Dimensions = 350.00 × 350 mm

  Eff. slab depth (d) = 187.00 mm

  Critical Section Distance factor = 1.50

  Critical Section Dimensions (A_x,A_y) = 630.50 mm^2

  Unbalanced Moment on column(M_o)(Including pattern load) = 70.38 $kN.m$

  30 % Reduction is allowed = 49.26 $kN.m$

  panel area = 9.24 m^2

  Shear force on critical section (V_u) = 124.46 kN

  V_t= 115.48 kN V_{eff}= 144.34 kN

  u_o= 700.00kN

  (v_{eff}/u_o*d) = 1.10 N/m^2

  U= 1261.00

  shear stress v = V_{eff} /ud = 0.61

- **Permissible punching shear stress calculation**

A_{SC}= 602.88	b = 1000
d = 225	PT % = 0.27
400/d= 1.78	V= 0.50

Final results:

Table 4.25: Along longer side C.S moment(BS,with staggered column)

	Exterior span A-B			Interior span B-C			Exterior span C-D		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
1	-46.5	54.1	-116.1	-111.0	36.7	-113.7	-120.2	57.2	-47.7
2	-29.3	104.2	-196.8	-206.9	56.2	-263.6	-272.6	151.6	-57.9
3	-47.4	54.8	-113.2	-104.9	32.9	-102.4	-109.5	51.7	-45.9

Table 4.26: Along shorter side C.S moment(BS,with staggered column)

	Exterior span A-B			Interior span B-C		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
4	-28.5	43.6	-101.3	-101.3	43.6	-28.5
5	-29.4	97.9	-218.8	-218.8	97.8	-29.3
6	-46.1	129.2	-243.0	-223.7	77.0	-12.6
7	-29.0	48.3	-111.8	-111.8	48.3	-29.0

Table 4.27: Along longer side M.S moment(BS,with staggered column)

	Exterior span A-B			Interior span B-C			Exterior span C-D		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
1	-20.4	86.9	-71.5	-71.5	53.0	-81.8	-85.5	108.8	-25.6
2	-20.7	87.5	-70.5	-69.5	49.9	-78.1	-81.9	104.3	-24.9

Table 4.28: Along shorter side M.S moment(BS,with staggered column)

	Exterior span A-B			Interior span B-C		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
3	-14.4	75.7	-70.2	-70.2	75.6	-14.4
4	-12.6	92.9	-77.0	-73.7	71.5	-7.0
5	-17.4	92.4	-77.8	-74.5	71.0	-11.8

Table 4.29: Punching shear result(BS,with staggered column)($v_{manual}/v_{permissible}$)

column number	punching ratio	V_u	M_{ux}	M_{uy}	depth	location	result
1	1.224	115.48	70.38	46.45	187	Corner	failed
2	1.375	281.36	12.30	59.38	187	Edge	failed
3	1.412	288.92	10.14	57.73	187	Edge	failed
4	1.279	120.64	72.39	48.15	187	Corner	failed
5	1.712	197.33	56.80	0.00	187	Edge	failed
6	1.345	558.52	13.88	0.03	187	Interior	failed
7	1.418	595.73	39.12	18.60	187	Interior	failed
8	2.109	243.20	98.41	0.00	187	Edge	failed
9	1.233	116.29	71.51	46.45	187	Corner	failed
10	1.320	270.12	16.60	59.29	187	Edge	failed
11	1.283	262.65	14.33	39.51	187	Edge	failed
12	1.178	111.10	69.41	48.15	187	Corner	failed

4.4.2 Without Staggered Column

Fixed end moment for span:

Fixed end moment, for span A-B = 143.4670 $kN.m$

Fixed end moment, for span B-C = 150.2290 $kN.m$

Fixed end moment, for span C-D = 149.8203 $kN.m$

Limitation of negative design moment

M' = (nL_2/8)*(L_1-(2h_c)/3)^2

n= loading per unit area on slab = 14.075 kN/m^2

$h_c/2$ square column = 197.51 mm

$h_c/2$ circular column = 195 mm $h_c/2$ rectangular column = 195.49 mm

M' span A-B = 206.69 $kN.m$

M' span B-C = 206.11 $kN.m$

M' span C-D = 206.11 $kN.m$

Average moment = 192.32$kN.m$

revised moment = 13.789 $kN.m$

Table 4.30: Along longer side C.S moment(BS,without staggered column)

	Exterior span A-B			Interior span B-C			Exterior span C-D		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
1	-46.95	54.26	-114.24	-107.46	34.56	-107.46	-114.24	54.26	-46.95
2	-42.23	126.53	-231.03	-226.86	60.86	-225.49	-231.26	126.38	-42.40
3	-46.95	54.26	-114.24	-107.46	34.56	-107.46	-114.53	54.26	-46.66

Table 4.31: Along shorter side C.S moment(BS,without staggered column)

	Exterior span A-B			Interior span B-C		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
4	-28.79	45.90	-106.46	-106.46	45.90	-28.79
5	-28.60	99.25	-221.55	-221.55	99.25	-28.60
6	-28.88	99.25	-221.27	-221.27	99.25	-28.88
7	-28.79	45.90	-106.46	-106.46	45.90	-28.79

Table 4.32: Along longer side M.S moment(BS,without staggered column)

	Exterior span A-B			Interior span B-C			Exterior span C-D		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
1	-22.7	96.2	-76.6	-73.6	53.2	-73.4	-76.6	96.1	-22.7
2	-22.7	96.2	-76.6	-73.6	53.2	-73.4	-76.7	96.1	-22.6

Table 4.33: Along shorter side M.S moment(BS,without staggered column)

	Exterior span A-B			Interior span B-C		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
3	-14.4	78.2	-72.4	-72.4	78.2	-14.4
4	-9.6	81.2	-73.8	-73.8	81.2	-9.6
5	-14.4	78.2	-72.4	-72.4	78.2	-14.4

Table 4.34: Punching shear result(BS,without staggered column)($v_{manual}/v_{permissible}$)

column number	punching ratio	V_u	M_{ux}	M_{uy}	depth	location	result
1	1.226	115.59	70.88	47.31	187.00	Corner	failed
2	1.341	274.56	14.55	58.77	187.00	Edge	failed
3	1.341	274.56	14.55	58.77	187.00	Edge	failed
4	1.226	115.59	70.88	47.31	187.00	Corner	failed
5	1.814	209.11	75.82	0.00	187.00	Edge	failed
6	1.379	560.93	24.10	0.00	187.00	Interior	failed
7	1.376	560.67	26.15	0.00	187.00	Interior	failed
8	1.903	219.46	75.73	0.00	187.00	Edge	failed
9	1.226	115.59	70.88	47.31	187.00	Corner	failed
10	1.341	274.56	14.55	58.77	187.00	Edge	failed
11	1.341	274.56	14.55	58.77	187.00	Edge	failed
12	1.226	115.59	70.88	47.31	187.00	Corner	failed

4.5 Design Example as per EuroCode2:Part1-2004

4.5.1 With Staggered Column

Design the flat slab with staggered column shown in figure. Using equivalent frame method. It is subjected to live load of 3 kN/m^2 and floor finish of 1 kN/m^2. The grade of steel used is Fe 415.Floor to floor height of column 3.5m.

- **Solution**

  Basic L/d ratio for continuous slab = 26 Provide total depth D = 225 mm

  Effective depth provided = d = 225 -20-(12/2) = 199 mm

  Effective depth in transverse direction = 199 - 12 = 187 mm

- **Load calculation**

  Ultimate design load : W_{ud} = 1.25 (25 x 0.225 + 1) = 8.28 kN/m^2

  W_{uL} = 1.5 x 3 = 3.75 kN/m^2

  $W_u = W_{ud} + W_{uL}$ = 12.78125 kN/m^2

Fixed end moment for span:

Fixe end moment =

$$FEM = \frac{WL_2L_1^2}{12} \tag{4.11}$$

Fixed end moment, for span A-B = 130.279 $kN.m$

Fixed end moment, for span B-C = 136.420 $kN.m$

Fixed end moment, for span C-D = 136.049 $kN.m$

Effective diameter of column ,h_c =((4A)/3.14)$^{0.5}$ < 0.25 L_y

column square = (350×350)= 395.0328536 mm < 1404 mm

column rectangular = (300×400)= 390.9811275 mm < 1404 mm

column circular = 390dia = 390 mm < 1404 mm

or(1/4)*L_y = 1404 mm

- **Check moment connection to edge column**

  $M_{t,max}$ = 0.17* b_e * d^2 *F_{cu}

  b_e = C_x +C_y = 700 mm

  C_x= 350mm C_y= 350 mm

  d= 199 mm F_{cu} = 30 N/mm^2

  $M_{t,max}$= 0.17 * b_e * d^2 *F_{cu} = 141.37557 $kN.m$

- **Punching shear check**

  Column Dimensions = 350.00 mm × 350 mm

  effective slab depth (d)= 193.00 mm

  determine β at corner column as shown in Fig.3.15 = 1.50

  V_{eff} = 104.86 kN

$c_2 + c_1 = 700.00\ mm$

U_o = 3d $\leq (c_2 + c_1) = 579.00$

V_{ed}= βV_{eff}/ U_o d =1.41

V_{rd},max = 4.75

$V_{ed} < V_{rd}$,max = ok

For critical section at 2d distance:

U_o = 3d + $(\pi/2)$*2d $\leq (c_2 + c_1) = 1185.02$

V_{ed}= βV_{eff}/ U_o d = 0.69

γc = 1.5

K = 1+ (200/d)$^{0.5}$ $\leq$ 2 = 2.017973197 K = 1+ (200/d)$^{0.5}$ $\leq$ 2 = 2 b =1000

A_{sx}= 602.88 A_{sy}= 602.88

ρix perpendicular to corner = 0.003123731

ρiy parallel to corner = 0.003123731

ρi = (ρix *ρiy)$^{0.5}$ = 0.002082487

V_{rd},c = 0.18/ρc * K *(100 ρi f_{ck})$^{0.333}$ = 0.505613799

check = $V_{ed} > V_{rd}$,c $\therefore$ Shear Reinforcement required.

Perimeter Required such that punching shear links are no longer required.

V_{eff} = 104.86 kN

$V_{rd,c}$ = 0.505

U_{out} = V_{ed}* β/(d*V_{rd},c) = 1611.865

Length of column face = C_1+C_2 = 700.00 mm

Radius to U_{out} = 580.805 mm

Perimeters of shear rein. May stop = 410.5 mm

Table 4.35: Punching shear result(EC2,with staggered column)($v_{manual}/v_{permissible}$)

column number	punching ratio	V_u	depth	location	result
1	1.360	104.86	193	Corner	failed
2	1.444	255.50	193	Edge	failed
3	1.483	262.37	193	Edge	failed
4	1.421	109.55	193	Corner	failed
5	1.067	179.19	193	Edge	failed
6	1.163	472.95	193	Interior	failed
7	1.290	540.97	193	Interior	failed
8	1.276	220.85	193	Edge	failed
9	1.370	105.60	193	Corner	failed
10	1.386	245.29	193	Edge	failed
11	1.348	238.51	193	Edge	failed
12	1.309	100.89	193	Corner	failed

- **Shear reinforcement**

  $S_{r,max}$ = 0.75 * d = 144.75

  provide S_r = 115

  inside 2d control perimeter $S_{t,max}$ = 289.5

  outside basic perimeter $S_{t,max}$ = 386

  A_{sw} $\geq$(V_{ed}-0.75*$V_{rd,c}$)S_r,U_o /1.5*$f_y w_d$,e_f) $f_y$wd from Fig.3.16 = 292.32

  $f_y$wd,e_f = 250 + 0.25 d_{eff} $\leq$ $f_y$wd = 298.25

  take min $f_y wd$,e_f = 292.32

  A_{sw} = 95.88

  A_{sw},min $\geq$ 0.08 ${f_{ck}}^{0.5}$(S_r*S_t)/(1.5 f_{yk} sinα + cosα)

  A_{sw},min = 23.434

  the minimum spacing of bar should be greater of bar diameter = 12ϕ

  Aggregate size plus 5 mm = 9.75   min spacing = 20 mm

  use 12 $mm\phi$ bars = 0.84856

  C/C spacing = 1649.83 mm

Final results:

Table 4.36: Along longer side C.S moment(EC2,with staggered column)

	Exterior span A-B			Interior span B-C			Exterior span C-D		
FRAME	${M_{ul}}^{-}$	${M_u}^{+}$	${M_{ur}}^{-}$	${M_{ul}}^{-}$	${M_u}^{+}$	${M_{ur}}^{-}$	${M_{ul}}^{-}$	${M_u}^{+}$	${M_{ur}}^{-}$
1	-34.6	49.1	-98.2	-91.0	33.3	-93.5	-101.6	51.9	-35.4
2	-19.1	94.7	-169.5	-215.8	51.0	-215.8	-235.5	137.7	-42.8
3	-35.3	49.7	-95.4	-86.4	29.9	-84.0	-92.1	46.9	-34.4

Table 4.37: Along shorter side C.S moment(EC2,with staggered column)

	Exterior span A-B			Interior span B-C		
FRAME	${M_{ul}}^{-}$	${M_u}^{+}$	${M_{ur}}^{-}$	${M_{ul}}^{-}$	${M_u}^{+}$	${M_{ur}}^{-}$
4	-19.5	39.5	-83.7	-83.7	39.5	-19.5
5	-16.6	88.9	-190.1	-190.1	88.8	-16.6
6	-27.9	117.3	-208.0	-199.4	69.9	-5.9
7	-19.6	43.9	-92.6	-92.6	43.9	-19.6

Table 4.38: Along longer side M.S moment(EC2,with staggered column)

	Exterior span A-B			Interior span B-C			Exterior span C-D		
FRAME	${M_{ul}}^{-}$	${M_u}^{+}$	${M_{ur}}^{-}$	${M_{ul}}^{-}$	${M_u}^{+}$	${M_{ur}}^{-}$	${M_{ul}}^{-}$	${M_u}^{+}$	${M_{ur}}^{-}$
1	-14.7	78.9	-61.0	-57.6	48.1	-67.1	-73.1	98.8	-18.9
2	-14.9	79.4	-60.0	-56.1	45.4	-64.0	-69.9	94.7	-18.6

Table 4.39: Along shorter side M.S moment(EC2,with staggered column)

	Exterior span A-B			Interior span B-C		
FRAME	${M_{ul}}^{-}$	${M_u}^{+}$	${M_{ur}}^{-}$	${M_{ul}}^{-}$	${M_u}^{+}$	${M_{ur}}^{-}$
3	-9.28	68.72	-59.59	-59.59	68.69	-9.27
4	-7.4	84.4	-66.4	-64.9	64.9	-3.7
5	-11.2	83.9	-65.5	-64.1	64.5	-7.5

4.5.2 Without Staggered Column

Fixed end moment for span:

Fixed end moment, for span A-B = 129.908 $kN.m$

Fixed end moment, for span B-C = 129.908 $kN.m$

Fixed end moment, for span C-D = 129.908 $kN.m$

Limitation of negative design moment

$M' = (\mathrm{n}L_2/8)*(L_1\text{-}(2h_c)/3)^2$

n= loading per unit area on slab = 12.781 kN/m^2

$h_c/2$ square column = 197.5164268 mm

$h_c/2$ circular column = 195 mm

$h_c/2$ rectangular column = 195.4905637 mm

M' span A-B = 187.165 $kN.m$

M' span B-C = 187.165 $kN.m$

M' span C-D = 187.165 $kN.m$

Average moment = 174.64

revised moment = 12.521

Check moment connection to edge column

$M_{t,max} = 0.17 * b_e * d^2 * F_{cu}$

$b_e = C_x + C_y = 700$ mm

C_x= 350 mm C_y= 350 mm

d= 199 mm

$F_{cu} = 30$ N/mm^2

$M_{t,max}$= 0.17 * b_e * d^2 *F_{cu} = 141.37557 $kN.m$

Final results:

Table 4.40: Along longer side C.S moment(EC2,without staggered column)

	Exterior span A-B			Interior span B-C			Exterior span C-D		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
1	-34.9	49.3	-96.4	-88.3	31.4	-88.3	-96.4	49.3	-34.9
2	-29.8	114.9	-199.2	-187.5	55.3	-186.3	-199.3	114.8	-29.8
3	-34.9	49.3	-96.4	-88.3	31.4	-88.3	-96.4	49.3	-34.9

Table 4.41: Along shorter side C.S moment(EC2,without staggered column)

	Exterior span A-B			Interior span B-C		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
4	-19.6	41.7	-88.1	-88.1	41.7	-19.6
5	-15.9	90.1	-192.5	-192.5	90.1	-15.9
6	-15.9	90.1	-192.1	-192.1	90.1	-15.9
7	-19.6	41.7	-88.1	-88.1	41.7	-19.6

Table 4.42: Along longer side M.S moment(EC2,without staggered column)

	Exterior span A-B			Interior span B-C			Exterior span C-D		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
1	-18.4	82.7	-63.0	-58.3	47.7	-58.1	-63.0	82.6	-18.4
2	-18.4	82.7	-63.0	-58.3	47.7	-58.1	-63.0	82.6	-18.4

Table 4.43: Along shorter side M.S moment(EC2,without staggered column)

	Exterior span A-B			Interior span B-C		
FRAME	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
3	-10.9	67.5	-58.7	-58.7	67.5	-10.9
4	-5.3	73.7	-64.1	-64.1	73.7	-5.3
5	-10.9	67.5	-58.6	-58.6	67.5	-10.9

Table 4.44: Punching shear result(EC2,without staggered column)($v_{manual}/v_{permissible}$)

column number	punching ratio	V_u	depth	location	result
1	1.362	104.96	193	Corner	failed
2	1.409	249.33	193	Edge	failed
3	1.409	249.33	193	Edge	failed
4	1.362	104.97	193	Corner	failed
5	1.188	199.40	193	Edge	failed
6	1.252	509.37	193	Interior	failed
7	1.246	509.13	193	Interior	failed
8	1.187	199.28	193	Edge	failed
9	1.362	104.97	193	Corner	failed
10	1.409	249.33	193	Edge	failed
11	1.409	249.33	193	Edge	failed
12	1.362	104.97	193	Corner	failed

- **Deflection Calculation As per L/d ratio:**

  Allowable L/d= N x Kx F1 x F2 x F3

  L.L < 1.5 D.l = ok

  Where N = L/d as per figure 4.5.

  pt % = 0.23

  N = 39

  K= 1.2 for flat slab

  Determine Factor 1 (F1)

  For ribbed or waffle slabs , F1 = 1 0.1 $((b_f/b_w)$ 1)$\geq$ 0.8

  Where b_f = flange breadth and b_w = rib breadth

  Otherwise F1 = 1.0

  Determine Factor 2 (F2)

  Where the slab span exceeds 7 m and it supports brittle partitions, F2 = 7/ l_{eff}

  Otherwise F2 = 1.0

  Determine Factor 3 (F3)

F3 = 310/Ss

Where Ss = Stress in reinforcement at serviceability limit state or Ss may be assumed to be 310 MPa (i.e. F3 = 1.0)

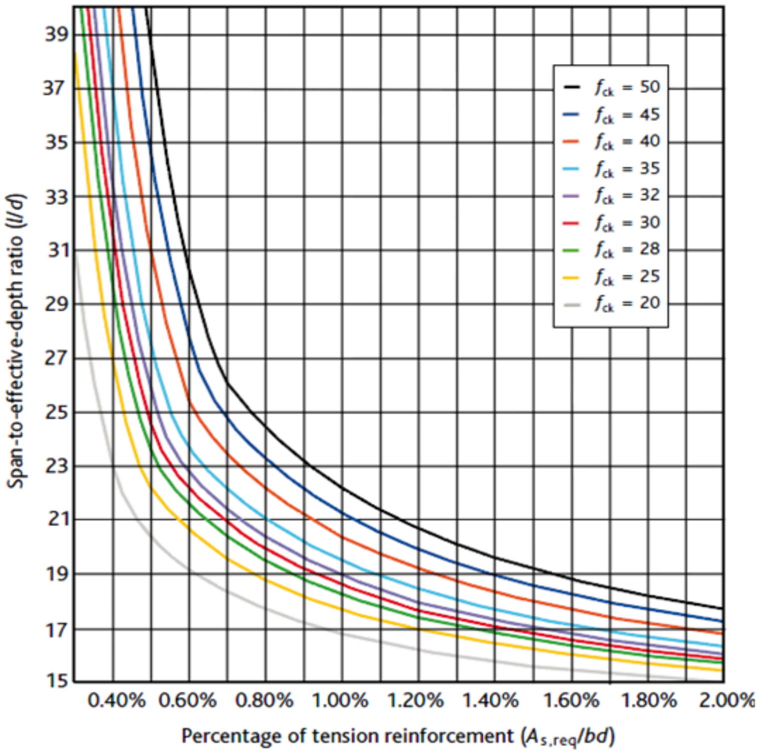


Figure 4.5: Basic span- to - effective-depth ratios for flat slabs

F1 = 1

F2 = 1

F3 = 1

Allowable L/d = 46.8 L= 6250 mm

Actual L/d = (L/260) = 24.038

Allowable L/d > Actual L/d $\therefore$ safe

Days	Load(kN/m^2)	Change in load
3	5.625	5.625
7	10.6125	4.9875
10	5.625	-4.9875
28	7.525	1.9
60	8.525	1

- **Deflection Calculation As per simple method:**

loading

strike 3 days=5.625 kN/m^2

cast floor above 7 days = 70 % (self weight +1.5) = 4.9875 kN/m^2

strike floor above 10 days = load relief = -4.9875

install briffle partitions 28 days = 1 kN/m^2

other dead loads 60 days = 1 kN/m^2

permanent imposed load 28 days = 0.3 * L.L = 0.9 kN/m^2

Variable imposed load in future = 0.7 * L.L = 2.1 kN/m^2

Material

f_{ck} = 30 N/mm^2

F_{cm} = f_{ck} +8 = 38 N/mm^2

f_{ctm} = 0.3 ${f_{ck}}^{(2/3)}$ = 2.896 N/mm^2

slab thikness= 225 mm

f_{ctm},f_t =(1.6-h/1000)f_{ctm} = 3.982643711 N/mm^2

for restraint of 50 % use = 3.555 N/mm^2

E_{cm} =22(f_{cm}/10)$^{0.3}$ = 32.836 kN/mm^2

secant modulus E_{c28} =1.05*E_{cm} = 34.4783 kN/mm^2

Critical load stage for construction

using vollum's approach the critical load stage corresponds with the minimum K value, where K is given by:

age	β	w(kN/m^2)	f_{ct}	K
3	0.5	5.625	2.06	0.258
7	0.5	10.6125	2.68	0.178
28	0.5	7.525	3.44	0.323
60	0.5	8.525	3.63	0.301

K = f_{ct}/W *($\sqrt{\beta}$)

critical load stage is when slab above is cast at 7 days.As construction loading is critical and deflection is required after construction of partitions ,three sets of creep factors are required for deflection :10 days , 28 days & 50years

Long-term creep coefficients

Effect of self -weight 50 years $t_o = 3$

$$\phi(t, t_o) = \phi\beta c(t, t_o) \tag{4.12}$$

$$\phi o = \phi RH\beta(f_{cm})\beta(t_o) \tag{4.13}$$

for $f_{cm} > 35\ MPA\ h_o = 225$ mm

$$\alpha_1 = [\frac{35}{f_{cm}}]^{0.7} \tag{4.14}$$

α1= 0.944058949

$$\alpha_2 = [\frac{35}{f_{cm}}]^{0.2} \tag{4.15}$$

α2= 0.983686904

ϕRH= 1.761017525

$$\beta f_{cm} = \frac{16.8}{\sqrt{f_{cm}}} \tag{4.16}$$

$\beta(f_{cm})$= 2.725319875

$$\beta t_o = \frac{1}{0.1 + t_o^{0.2}} \tag{4.17}$$

$\beta(t_o)$= 0.743090592

ϕo = 3.566341474

t=50 years= 50*365 = 18250 days t_o= striking time

t=7 days = 7 days

t=10 days = 10 days

t=28 days = 28 days

$$\beta_c(t - t_o) = [(t - t_o)/(\beta H + (t - t_o))]^{0.3} \tag{4.18}$$

for $f_{cm} > 35$ Nm/m^2

$$\beta H = 1.5[1 + (0.012RH)^{18}]h_o + 250\alpha_3 \tag{4.19}$$

β H=577.462994

$$\alpha_3 = [\frac{35}{f_{cm}}]^{0.5} \tag{4.20}$$

α3 = 0.95971487

fore 50 years	for 28 days	for 10 days
β c= 0.9906	β c= 0.3849	β c= 0.26515
ϕ(t1t0)= 3.533	ϕ(t1t0)= 1.372	ϕ(t1t0)= 0.945

Similarly for ϕ for deflection other ages

loading age	10days	28days	50years
3	0.9456	1.372	3.533
7	0.6515	1.156	3.071
10	0	1.0345	2.873
28	0	0	2.364
60	0	0	2.0442

Composite creep coefficients

$$\frac{\sum w}{Ecomp} = \frac{W3}{E_{eef}3} + \frac{W7}{E_{eef}7} + \frac{W10}{E_{eef}10} + \frac{W28}{E_{eef}28} + \frac{W60}{E_{eef}60} \tag{4.21}$$

Age(days)	w(kN/m^2)	ϕ	E_{ceff}	W/E_{ceff}
3	5.625	0.945638685	17.72086292	0.317422466
7	4.9875	0.651517124	20.87680226	0.23890153
10	-4.9875			
28	1.9			
60	1			

w for 10 days = 9.55(kN/m^2) $\sum$ w for 10 days = 17.166(kN/m^2)

w for 28 days = 5(kN/m^2) $\sum$ w for 28 days = 12.102(kN/m^2)

w for 50 years = 8.525(kN/m^2) $\sum$ w for 50 years =8.09(kN/m^2)

Properties at critical stage(7-10 days)

E_{eff} = 17.16625577 KN/m^2

$$\alpha e = \frac{E_s}{E_{eff}} \tag{4.22}$$

αe= 11.6507643

b= 1408 d= 199

A_s= 565.2

$$\rho = \frac{A_s}{bd} \tag{4.23}$$

ρ= 0.002017188

Uncracked properties

$$x = [\frac{1 + 2(\alpha_e - 1)(\rho\frac{d}{h} + \rho'\frac{d}{h})}{2(1 + (\alpha_e - 1)(\rho + \rho'))}]h \tag{4.24}$$

x = 114.319

I= 1039573

M_{cr} = 27.20522383 $kN.m$

M_{cri}= wL^2 /8 = 49.005 $kN.m$

$M_{cri} > M_{cr}$ section is cracked

Calculate cracked x from

x= 30.863

I= 199955378.1 mm^4

$$I = [\frac{1}{3}(\frac{x}{d})^3 + \alpha_e\rho[1 - \frac{x}{d}]^2]bd^3 \tag{4.25}$$

and distribution factor =

$$\zeta cri = 1 - 0.5[\frac{Mcr}{M}]^2 \tag{4.26}$$

ζ cri= 0.845903128

this ζ factor will be used also for long-term deflection calculations,in conjunction with the 7 day value for f_{ctm} =2.8965 N/mm^2

Long-term properties

uncracked

E_{eff} = 8.099919024 kN/m^2 αe= 27.778

b= 1408 A_s= 565.2

ρ= 0.002017188

uncracked properties

M_{cr} = 30.0547948 $kN.m$

M_{cri}= wL^2 /8 = 49.005 $kN.m$

$M_{cri} > M_{cr}$ section is cracked

cracked

I= 409286911.5

E_{eff}= 8.09

M_o= 39.082

I_{unc} = 1121342414

I_{crac} =409286911.5

Flexural curvatures

$$\frac{I}{runcracked} = \frac{M}{I_{ceff}I_{unc}} \tag{4.27}$$

=4.30288 x 10^{-6}

$$\frac{I}{rcracked} = \frac{M}{I_{ceff}I_{crac}} \tag{4.28}$$

=1.17888 x 10^{-5}

ζcri = 0.84590

$$\frac{1}{r} = \zeta\frac{1}{Icrac} + (1-\zeta)\frac{1}{Iuncr} \tag{4.29}$$

=1.06352 x 10^{-5}

Shrinkage curvatures

$$\frac{1}{rcs} = \frac{\varepsilon cs \alpha_e S}{I} \tag{4.30}$$

εcs = 482 μs

Autogenous shrinkage

$$\varepsilon cs = 2.5(f_{ck} - 10) \times 10^{-3} = 50 \mu strain \tag{4.31}$$

$$\beta_{cc}(t) = 1 - exp(-0.2\sqrt{t}) = 1.000 \tag{4.32}$$

$$\varepsilon_{cs}(t) = 1.000 \times 50 = 50 \mu strain \tag{4.33}$$

$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca} = 532 \mu strain \tag{4.34}$$

Uncracked section

A_s= 565.2

suncrack = 46384.28873 mm^3

x= 116.932964

αe= 27.77805548

I= 1121342414

Cracked section

$$Scrack = A_s(d - x) - A_s^{'}(x - d^{'}) \tag{4.35}$$

Scrack = 85114.66586

Total curvature

$$\frac{1}{rtotal} = \frac{1}{r} + \frac{1}{rcs} \tag{4.36}$$

= 13.32905944

- **Deflection at mid span**

  M_a= 48.12511 $kN.m$

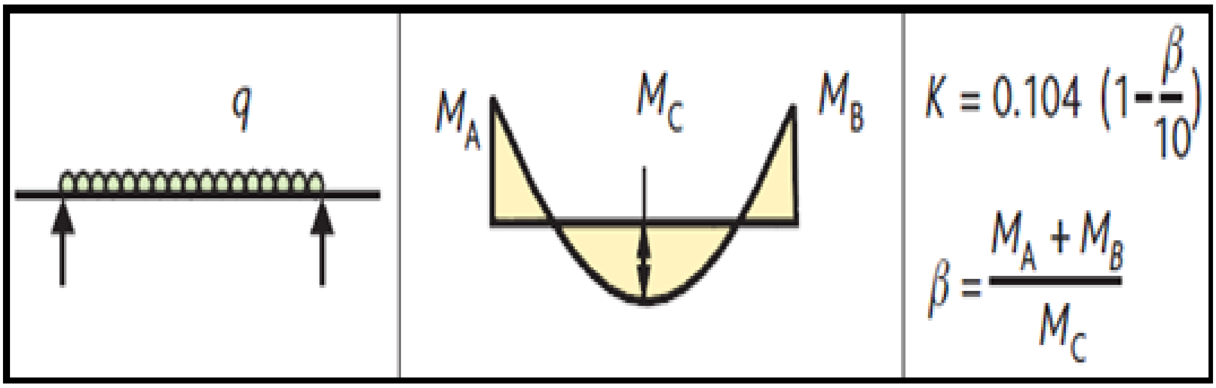


Figure 4.6: Deflection at mid-span

M_b= 115.5963 $kN.m$

M_c= 67.234661 $kN.m$

$$\delta = KL^2 \frac{1}{rtotal} \tag{4.37}$$

β = 2.435074522

K= 0.078675225

δ = 45.67992364

28 days= 22.83996182 mm

allowable deflection =L /250 = 26.4 mm

actual deflection = 22.83 mm

$\therefore$ safe

4.6 Comparison of Moments

4.6.1 With Staggered Column

Table 4.45: Along longer side frame(C.S moment)

frame	Code	Exterior span A-B			Interior span B-C			Exterior span C-D		
		$M_{ul}{}^-$	$M_u{}^+$	$M_{ur}{}^-$	$M_{ul}{}^-$	$M_u{}^+$	$M_{ur}{}^-$	$M_{ul}{}^-$	$M_u{}^+$	$M_{ur}{}^-$
1	IS	-52.1	60.5	-110.9	-102.8	41.0	-105.6	-114.8	64.0	-53.2
	ACI	-38.8	54.9	-100.7	-93.8	33.7	-96.7	-104.2	58.2	-38.9
	BS	-46.5	54.1	-116.1	-111.0	36.7	-113.7	-120.2	57.2	-47.7
	EC2	-34.6	49.1	-98.2	-91.0	33.3	-93.5	-101.6	51.9	-35.4
2	IS	-28.8	116.6	-191.4	-185.1	62.9	-243.8	-266.0	169.7	-64.4
	ACI	-7.9	109.0	-169.6	-166.6	49.4	-227.9	-238.6	159.0	-27.0
	BS	-29.3	104.2	-196.8	-206.9	56.2	-263.6	-272.6	151.6	-57.9
	EC2	-19.1	94.7	-169.5	-215.8	51.0	-215.8	-235.5	137.7	-42.8
3	IS	-53.1	61.3	-107.8	-97.5	36.9	-94.9	-104.0	57.8	-51.8
	ACI	-39.7	55.7	-97.7	-89.2	30.2	-86.5	-94.3	52.3	-39.5
	BS	-47.4	54.8	-113.2	-104.9	32.9	-102.4	-109.5	51.7	-45.9
	EC2	-35.3	49.7	-95.4	-86.4	29.9	-84.0	-92.1	46.9	-34.4

Table 4.46: Along shorter side frame(C.S moment)

FRAME	Code	Exterior span A-B			Interior span B-C		
		$M_{ul}{}^-$	$M_u{}^+$	$M_{ur}{}^-$	$M_{ul}{}^-$	$M_u{}^+$	$M_{ur}{}^-$
4	IS	-29.4	48.7	-94.6	-94.6	48.7	-29.4
	ACI	-19.0	44.0	-87.8	-87.8	44.0	-19.0
	BS	-28.5	43.6	-101.3	-101.3	43.6	-28.5
	EC2	-19.5	39.5	-83.7	-83.7	39.5	-19.5
5	IS	-25.1	109.5	-214.8	-214.8	109.4	-25.0
	ACI	-0.1	101.6	-199.4	-199.4	101.6	-0.1
	BS	-29.4	97.9	-218.8	-218.8	97.8	-29.3
	EC2	-16.6	88.9	-190.1	-190.1	88.8	-16.6
6	IS	-42.0	144.6	-235.0	-225.2	86.2	-8.8
	ACI	-5.9	136.0	-215.7	-212.4	78.2	7.9
	BS	-46.1	129.2	-243.0	-223.7	77.0	-12.6
	EC2	-27.9	117.3	-208.0	-199.4	69.9	-5.9
7	IS	-29.5	54.1	-104.6	-104.6	54.1	-29.5
	ACI	-17.6	49.1	-97.5	-97.5	49.1	-17.6
	BS	-29.0	48.3	-111.8	-111.8	48.3	-29.0
	EC2	-19.6	43.9	-92.6	-92.6	43.9	-19.6

Table 4.47: Along longer side frame(M.S moment)

frame	Code	Exterior span A-B			Interior span B-C			Exterior span C-D		
		${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$
1	IS	0	79.2	-68.9	-65.1	48.3	-75.8	-82.6	99.2	0
	ACI	0	72.9	-61.9	-59.0	38.9	-70.2	-74.5	91.8	0
	BS	-20.4	86.9	-71.5	-71.5	53.0	-81.8	-85.5	108.8	-25.6
	EC2	-14.7	78.9	-61.0	-57.6	48.1	-67.1	-73.1	98.8	-18.9
2	IS	0	79.7	-67.8	-63.4	45.5	-72.3	-79.0	95.1	0
	ACI	0	73.4	-60.8	-57.5	36.6	-66.8	-71.2	87.9	0
	BS	-20.7	87.5	-70.5	-69.5	49.9	-78.1	-81.9	104.3	-24.9
	EC2	-14.9	79.4	-60.0	-56.1	45.4	-64.0	-69.9	94.7	-18.6

Table 4.48: Along shorter side frame(C.S moment)

FRAME	Code	Exterior span A-B			Interior span B-C		
		${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$
3	IS	0	69.0	-67.3	-67.3	69.0	0
	ACI	0	63.2	-62.5	-62.5	63.2	0
	BS	-14.4	75.7	-70.2	-70.2	75.6	-14.4
	EC2	-9.28	68.72	-59.59	-59.59	68.69	-9.27
4	IS	0	84.7	-75.0	-73.3	65.2	0
	ACI	0	79.2	-69.2	-68.6	59.9	0
	BS	-12.6	92.9	-77.0	-73.7	71.5	-7.0
	EC2	-7.4	84.4	-66.4	-64.9	64.9	-3.7
5	IS	0	84.2	-74.0	-72.4	64.8	0
	ACI	0	78.0	-68.4	-67.9	58.8	0
	BS	-17.4	92.4	-77.8	-74.5	71.0	-11.8
	EC2	-11.2	83.9	-65.5	-64.1	64.5	-7.5

Table 4.49: Punching shear results(with staggered column)($v_{manual}/v_{permissible}$)

Column no		IS	ACI	BS	EC2
1	Pun. ratio	0.86	0.60	1.22	1.36
	Depth	187	205	187	193
	Result	safe	safe	failed	failed
2	Pun. ratio	0.88	0.88	1.37	1.44
	Depth	187	205	187	193
	Result	safe	safe	failed	failed
3	Pun. ratio	0.96	0.93	1.41	1.48
	Depth	187	205	187	193
	Result	safe	safe	failed	failed
4	Pun. ratio	0.86	0.59	1.27	1.42
	Depth	187	205	187	193
	Result	safe	safe	failed	failed
5	Pun. ratio	0.84	0.88	1.71	1.06
	Depth	187	205	187	193
	Result	safe	safe	failed	failed
6	Pun. ratio	1.01	2.01	1.34	1.16
	Depth	187	205	187	193
	Result	failed	failed	failed	failed
7	Pun. ratio	1.11	2.40	1.41	1.29
	Depth	187	205	187	193
	Result	failed	failed	failed	failed
8	Pun. ratio	0.99	1.11	2.10	1.27
	Depth	187	205	187	193
	Result	safe	failed	failed	failed
9	Pun. ratio	0.86	0.60	1.23	1.37
	Depth	187	205	187	193
	Result	safe	safe	failed	failed
10	Pun. ratio	0.9	0.8	1.32	1.38
	Depth	187	205	187	193
	Result	safe	safe	failed	failed
11	Pun. ratio	0.80	0.86	1.28	1.34
	Depth	187	205	187	193
	Result	safe	safe	failed	failed
12	Pun. ratio	0.86	0.59	1.17	1.30
	Depth	187	205	187	193
	Result	safe	safe	failed	failed

4.6.2 Without Staggered Column

Table 4.50: Along longer side frame(C.S moment)

frame	Code	Exterior span A-B			Interior span B-C			Exterior span C-D		
		${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$
1	IS	-52.5	60.71	-108.95	-99.70	38.67	-99.70	-108.95	60.7	-52.5
	ACI	-39.2	55.10	-98.86	-91.1	31.71	-91.1	-98.86	55.1	-39.2
	BS	-46.9	54.26	-114.24	-107.46	34.56	-107.46	-114.24	54.2	-46.9
	EC2	-34.9	49.3	-96.4	-88.3	31.4	-88.3	-96.4	49.3	-34.9
2	IS	-44.9	141.59	-225.02	-211.84	68.10	-210.48	-225.10	141.4	-44.8
	ACI	-16.4	132.406	-200.9	-194.2	54.06	-193.9	-201	132.3	-16.4
	BS	-42.2	126.53	-231.03	-226.86	60.86	-225.49	-231.26	126.3	-42.4
	EC2	-29.8	114.9	-199.2	-187.5	55.3	-186.3	-199.3	114.8	-29.8
3	IS	-52.5	60.71	-108.95	-99.70	38.67	-99.70	-108.95	60.7	-52.5
	ACI	-39.2	55.10	-98.86	-91.1	31.71	-91.1	-98.86	55.1	-39.2
	BS	-46.9	54.26	-114.24	-107.46	34.56	-107.46	-114.53	54.2	-46.6
	EC2	-34.9	49.3	-96.4	-88.3	31.4	-88.3	-96.4	49.3	-34.9

Table 4.51: Along shorter side frame(C.S moment)

FRAME	Code	Exterior span A-B			Interior span B-C		
		${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$
4	IS	-29.4	51.36	-99.53	-99.53	51.36	-29.4
	ACI	-18.3	46.5055	-92.59	-92.59	46.51	-18.3
	BS	-28.7	45.90	-106.46	-106.46	45.90	-28.7
	EC2	-19.6	41.7	-88.1	-88.1	41.7	-19.6
5	IS	-23.8	111.06	-217.48	-217.48	111.06	-23.8
	ACI	1.1	103.065	-202	-202	103.1	1.1
	BS	-28.6	99.25	-221.55	-221.55	99.25	-28.6
	EC2	-15.9	90.1	-192.5	-192.5	90.1	-15.9
6	IS	-23.8	111.06	-216.98	-216.98	111.06	-23.8
	ACI	1.1	103.114	-201.5	-201.5	103.1	1.1
	BS	-28.8	99.25	-221.27	-221.27	99.25	-28.8
	EC2	-15.9	90.1	-192.1	-192.1	90.1	-15.9
7	IS	-29.4	51.36	-99.53	-99.53	51.36	-29.4
	ACI	-19.6	41.7	-88.1	-88.1	41.7	-19.6
	BS	-28.7	45.90	-106.46	-106.46	45.90	-28.7
	EC2	-19.6	41.7	-88.1	-88.1	41.7	-19.6

Table 4.52: Along longer side frame(M.S moment)

frame	Code	Exterior span A-B			Interior span B-C			Exterior span C-D		
		M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
1	IS	0	87.67	-73.82	-68.54	48.48	-68.31	-73.83	87.62	0
	ACI	0	80.9	-66.4	-62.7	39.2	-62.7	-66.5	80.8	0
	BS	-22.7	96.2	-76.6	-73.6	53.2	-73.4	-76.6	96.1	-22.7
	EC2	-18.4	82.7	-63.0	-58.3	47.7	-58.1	-63.0	82.6	-18.4
2	IS	0	87.67	-73.82	-68.54	48.48	-68.31	-73.83	87.62	0
	ACI	0	80.9	-66.4	-62.7	39.2	-62.7	-66.5	80.8	0
	BS	-22.7	96.2	-76.6	-73.6	53.2	-73.4	-76.7	96.1	-22.6
	EC2	-18.4	82.7	-63.0	-58.3	47.7	-58.1	-63.0	82.6	-18.4

Table 4.53: Along shorter side frame(M.S moment)

FRAME	Code	Exterior span A-B			Interior span B-C		
		M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
3	IS	0	71.26	-69.42	-69.42	71.26	0
	ACI	0	65.4	-64.5	-64.5	65.4	0
	BS	-14.4	78.2	-72.4	-72.4	78.2	-14.4
	EC2	-10.9	67.5	-58.7	-58.7	67.5	-10.9
4	IS	0	74.04	-72.41	-72.41	74.04	0
	ACI	0	68.7	-67.2	-67.2	68.7	0
	BS	-9.6	81.2	-73.8	-73.8	81.2	-9.6
	EC2	-5.3	73.7	-64.1	-64.1	73.7	-5.3
5	IS	0	71.26	-69.34	-69.34	71.26	0
	ACI	0	65.4	-64.4	-64.4	65.4	0
	BS	-14.4	78.2	-72.4	-72.4	78.2	-14.4
	EC2	-10.9	67.5	-58.6	-58.6	67.5	-10.9

Table 4.54: Punching shear results(without staggered column)($v_{manual}/v_{permissible}$)

Column no		IS	ACI	BS	EC2
1	Pun. ratio	0.85	0.593	1.22	1.36
	Depth	187	205	187	193
	Result	safe	safe	failed	failed
2	Pun. ratio	0.71	0.862	1.34	1.40
	Depth	187	205	187	193
	Result	safe	safe	failed	failed
3	Pun. ratio	0.87	0.862	1.34	1.40
	Depth	187	205	187	193
	Result	safe	safe	failed	failed
4	Pun. ratio	0.85	0.593	1.22	1.36
	Depth	187	205	187	193
	Result	safe	safe	failed	failed
5	Pun. ratio	0.72	0.984	1.81	1.18
	Depth	187	205	187	193
	Result	safe	safe	failed	failed
6	Pun. ratio	1.04	2.191	1.37	1.25
	Depth	187	205	187	193
	Result	failed	failed	failed	failed
7	Pun. ratio	1.04	2.173	1.37	1.24
	Depth	187	205	187	193
	Result	failed	failed	failed	failed
8	Pun. ratio	0.72	0.984	1.90	1.18
	Depth	187	205	187	193
	Result	safe	safe	failed	failed
9	Pun. ratio	0.85	0.593	1.22	1.36
	Depth	187	205	187	193
	Result	safe	safe	failed	failed
10	Pun. ratio	0.71	0.862	1.34	1.40
	Depth	187	205	187	193
	Result	safe	safe	failed	failed
11	Pun. ratio	0.87	0.862	1.34	1.40
	Depth	187	205	187	193
	Result	safe	safe	failed	failed
12	Pun. ratio	0.85	0.593	1.22	1.36
	Depth	187	205	187	193
	Result	safe	safe	failed	failed

4.7 Design Detailing

- **General notes**

L.L = 3 kN/m^2	clear cover = 20 mm
concrete strength = 30 N/mm^2	reinforcement strength =415 N/mm^2
Column diameter = 390 mm	Floor to floor height of column = 3.5 m
square column = 350 × 350 mm	rectangular column = 300 × 400 mm

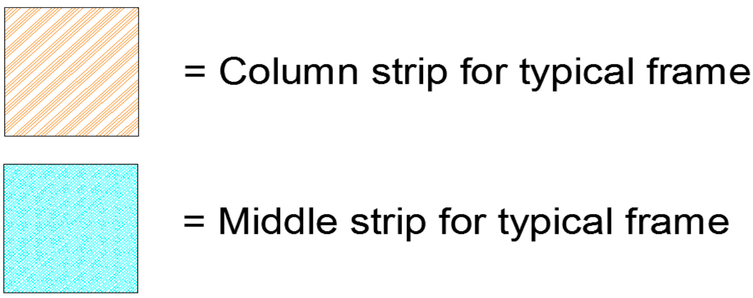
- As shown in Fig.4.13 & 4.14 provided A_{st} are mm^2/m

- As shown in Fig.4.15 & 4.16Value of top and bottom reinforcement are per meter.

  where,

  12-4-200 mm = 12ϕbar - 4 No. of bars -200 mm spacing c/c

- **Column & Middle strip for typical plan**



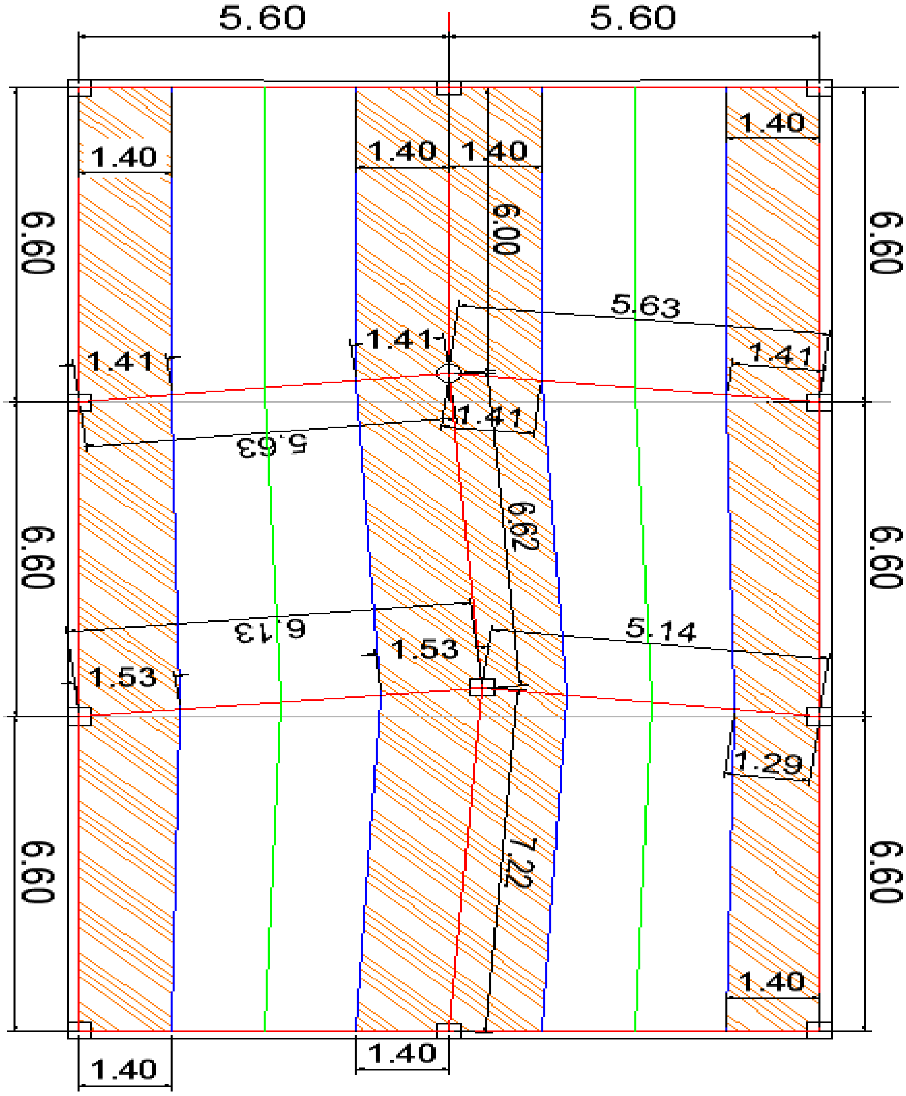


Figure 4.7: C.S for typical frame along longer side

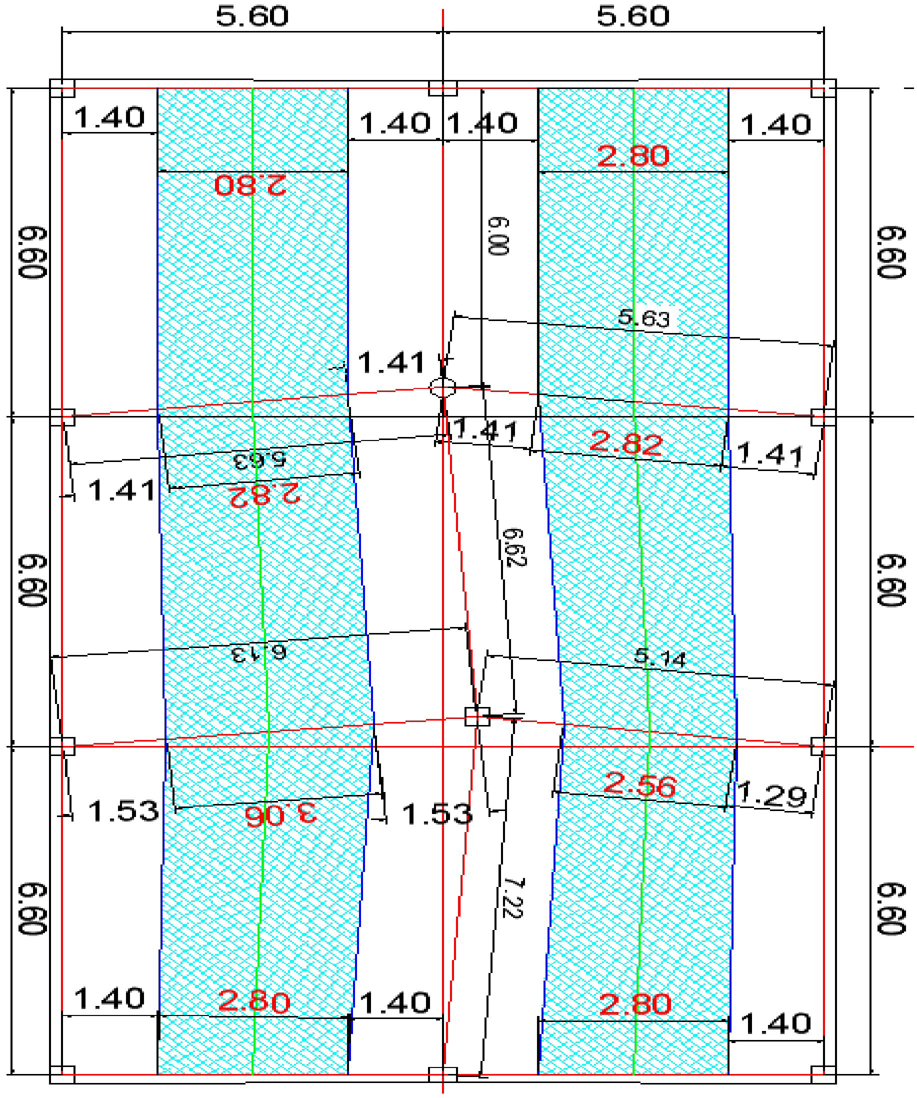


Figure 4.8: M.S for typical frame along longer side

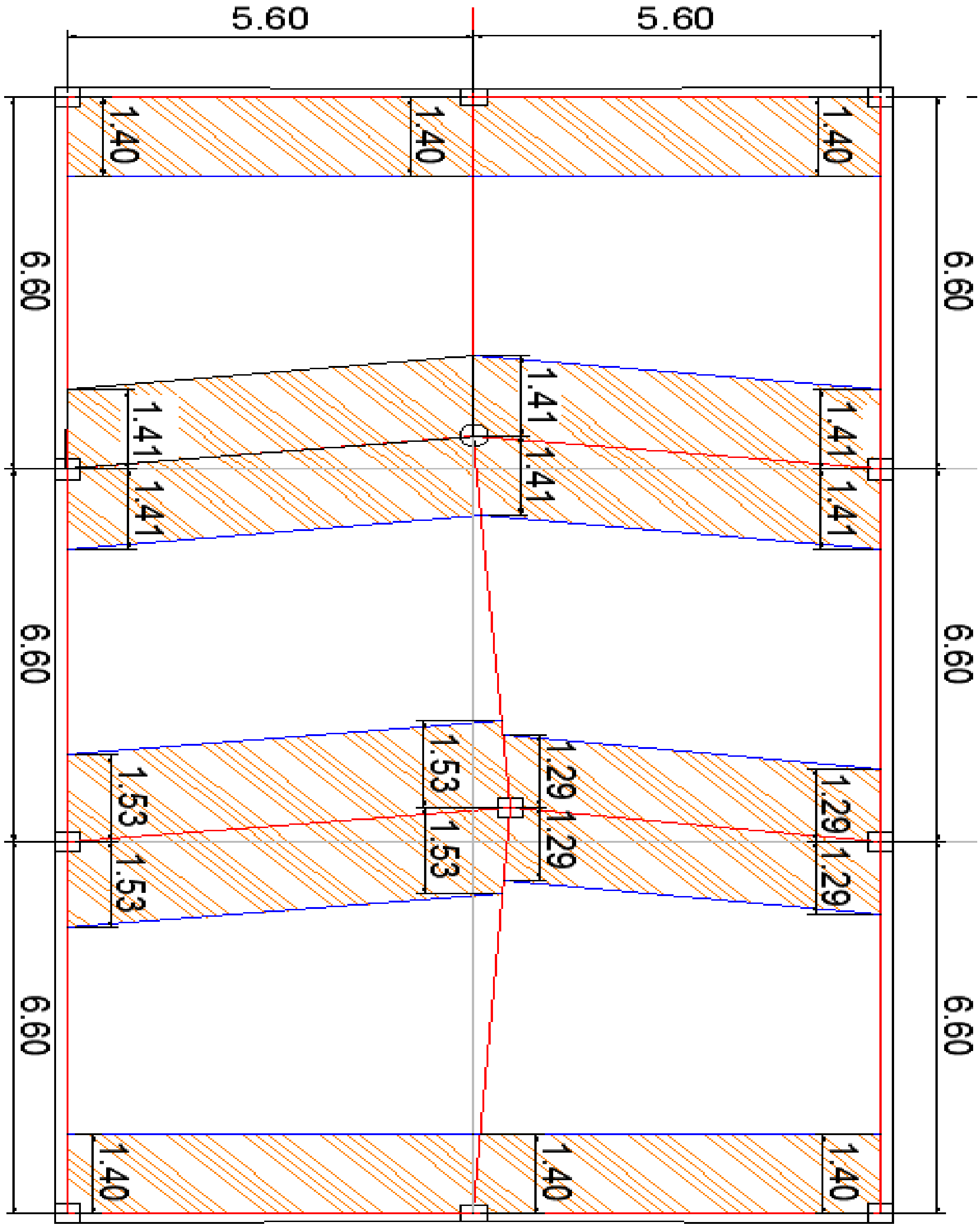


Figure 4.9: C.S for typical frame along shorter side

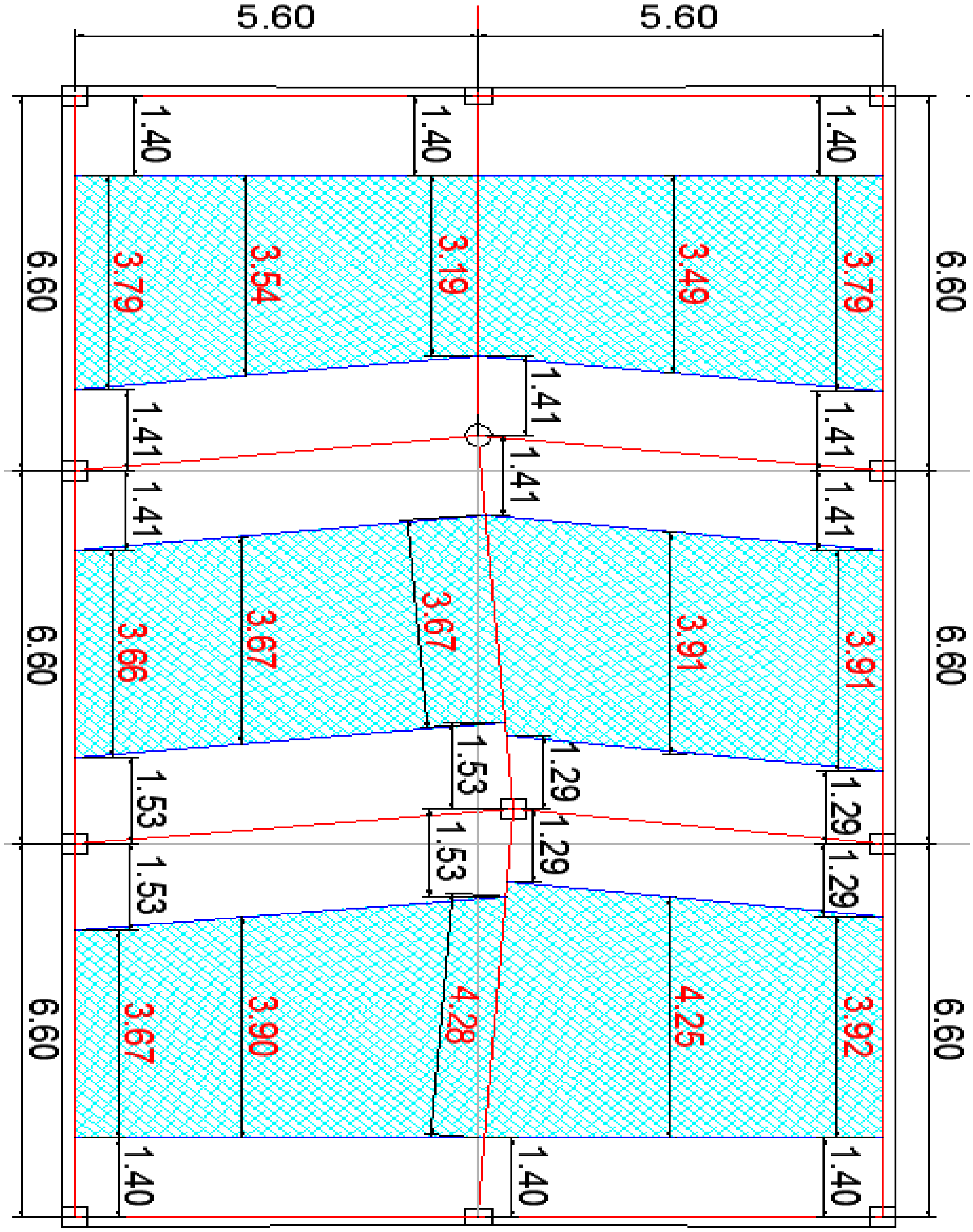


Figure 4.10: M.S for typical frame along shorter side

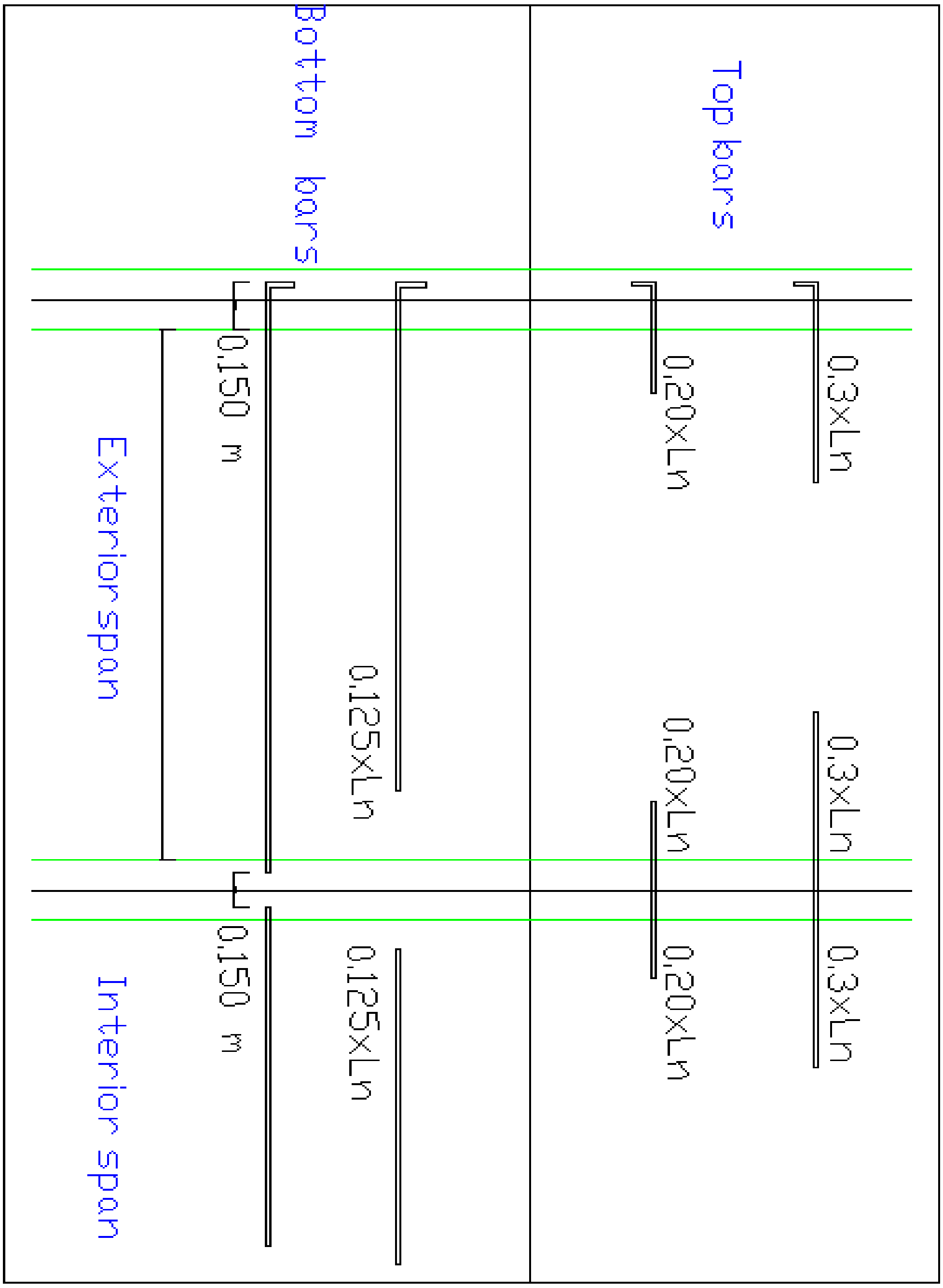


Figure 4.11: Slab rebars for column strip

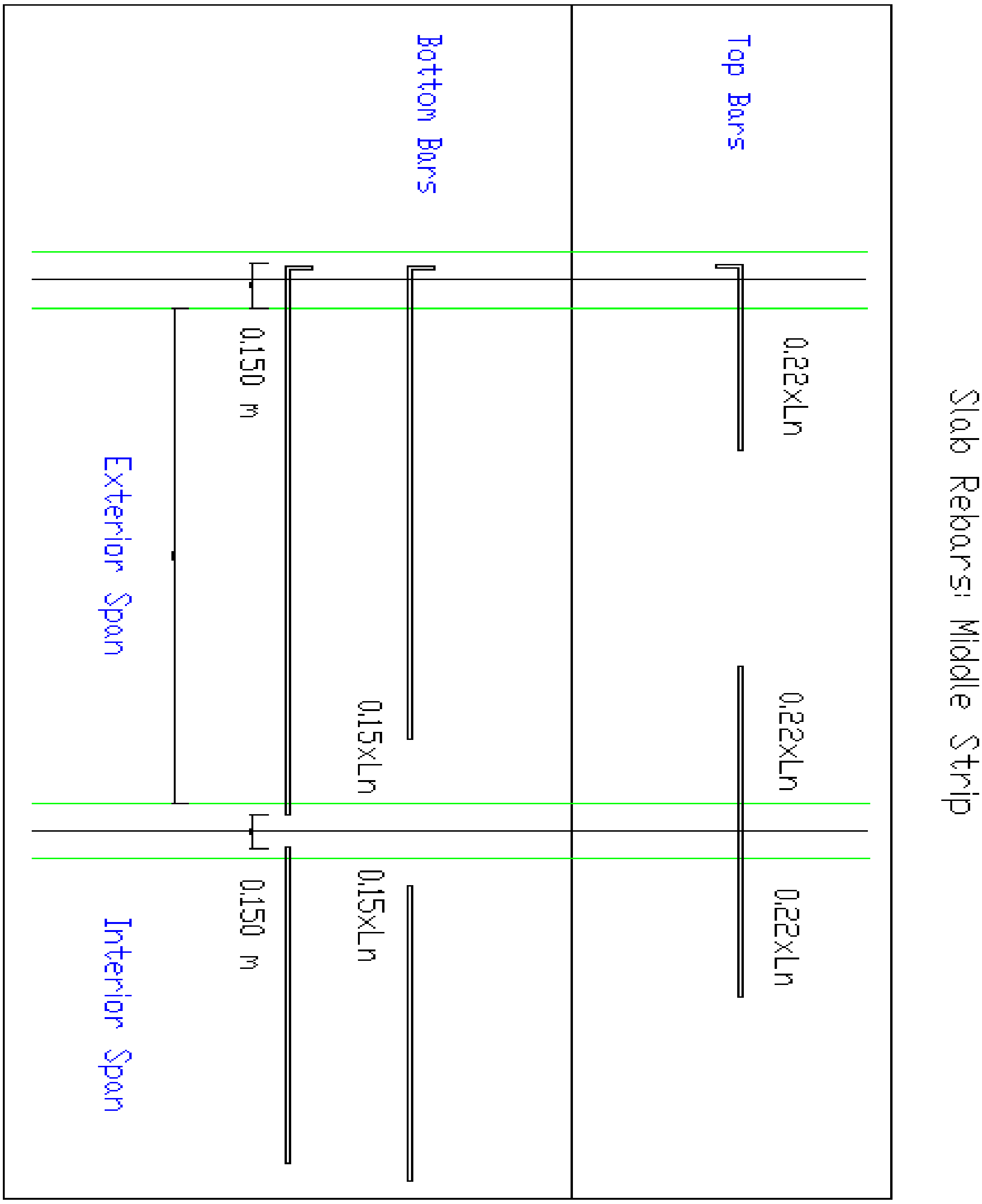


Figure 4.12: Slab rebars for middle strip

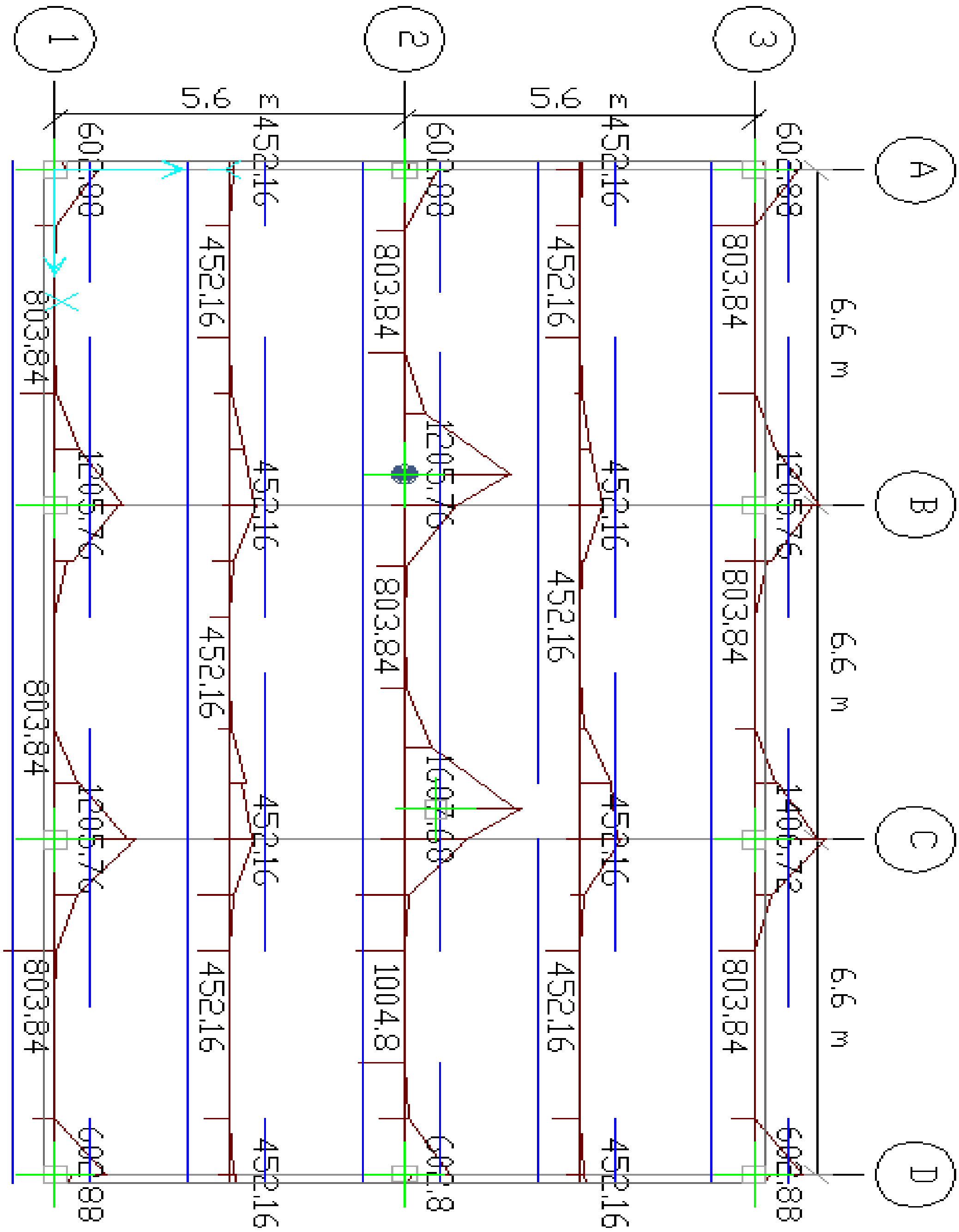


Figure 4.13: Provided $A_{st}(mm^2/m)$ along longer side

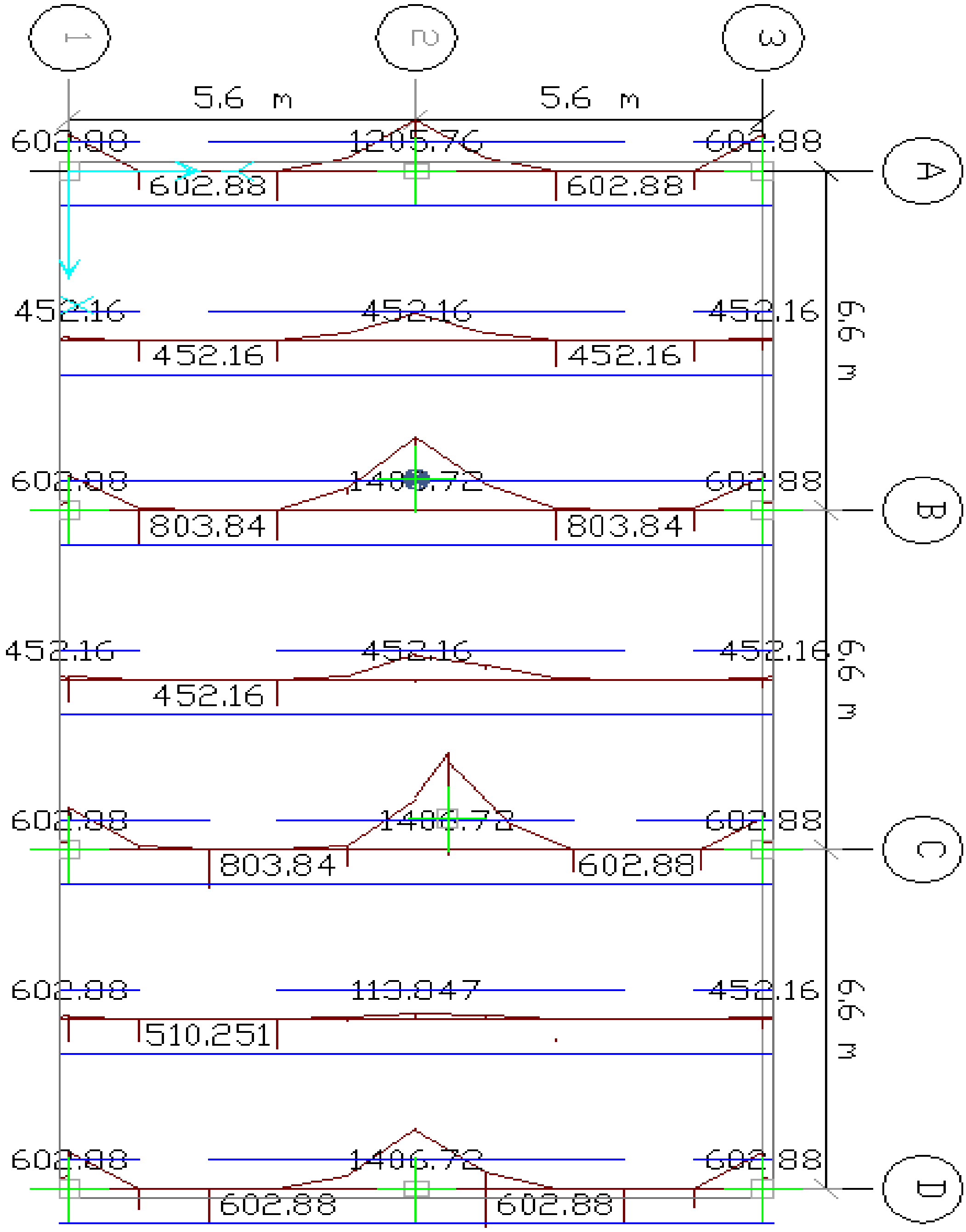


Figure 4.14: Provided $A_{st}(mm^2/m)$ along shorter side

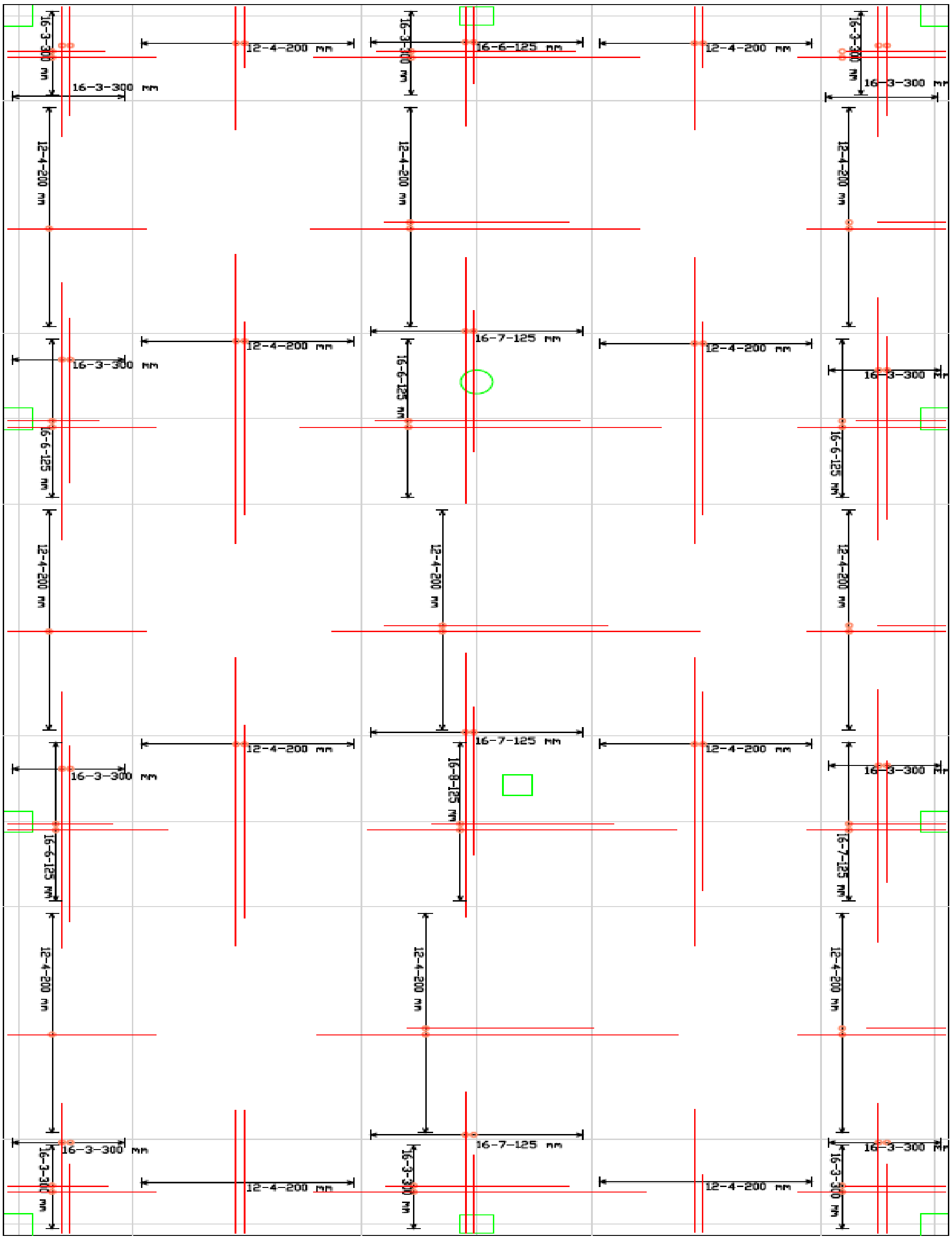


Figure 4.15: Top reinforcement per meter

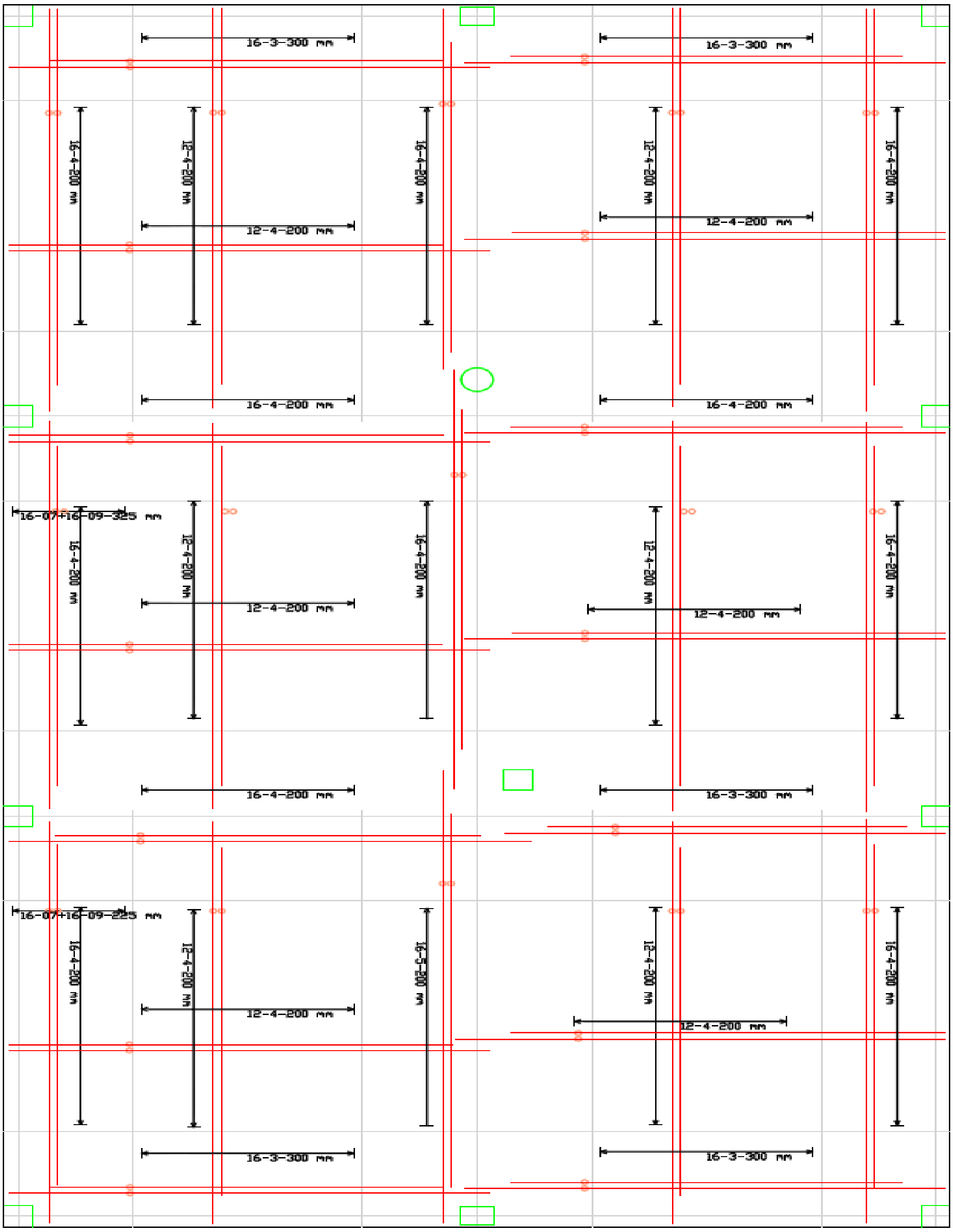


Figure 4.16: Bottom reinforcement per meter

4.8 summary

In this chapter describe design of flat slab (with & without staggered column) using various codes. Also comparison of moments and punching shear is carried out.

Chapter 5

Analysis and Design of Flat Slab Using SAFE

5.1 Analysis of Flat Slab as per IS 456-2000

Design the flat slab with staggered column shown in figure. Using equivalent frame method. It is subjected to live load of 3 kN/m^2 and floor finish of 1 kN/m^2. The grade of steel used is Fe 415.Floor to floor height of column 3.5m.

- **Flat slab analysis in SAFE.**

  ETABS software is used for modeling of at slabs, while Software SAFE is used for design of at slab which uses the analysis results of the ETABS software for design purpose as the SAFE software gives the detailed design of at slabs, for gravity loading as well as lateral loading and the results of the different methods are compared.

5.1.1 With Staggered Column

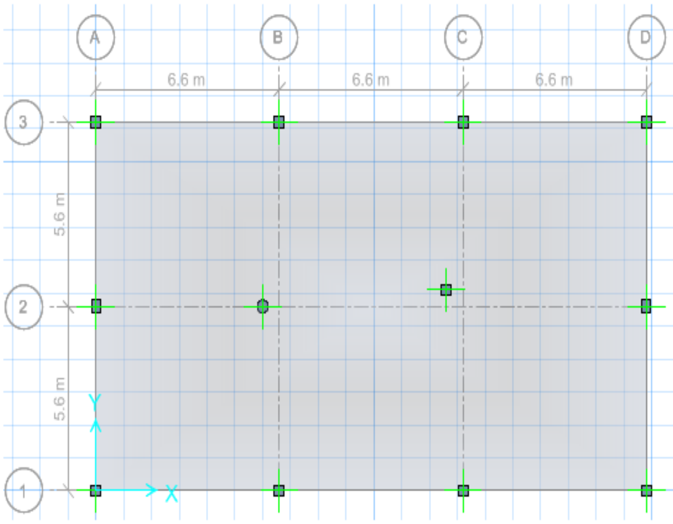


Figure 5.1: Slab model in SAFE(with staggered column)

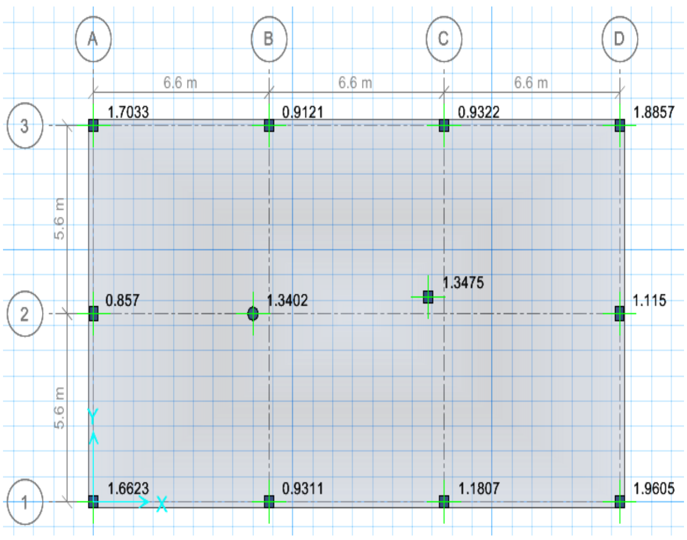


Figure 5.2: Punching shear ratio(SAFE:IS,with staggered column)

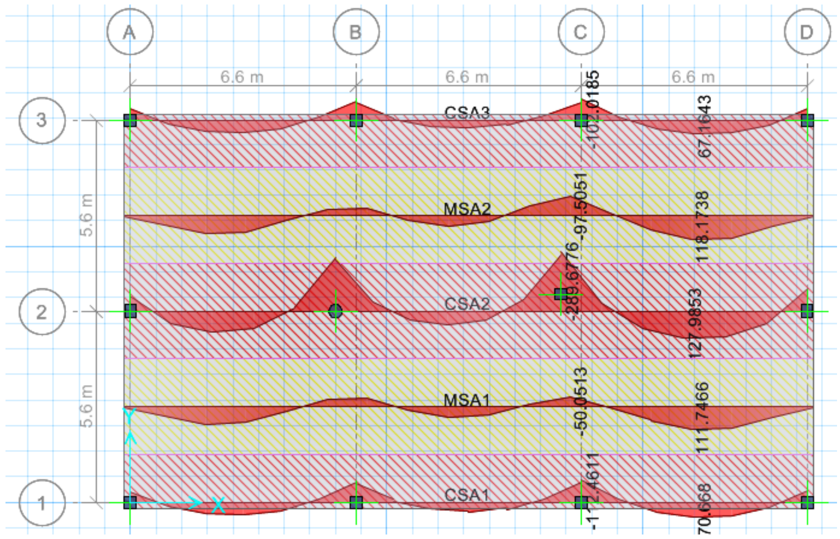


Figure 5.3: C.S & M.S along longer direction(SAFE:IS ,with staggered column)

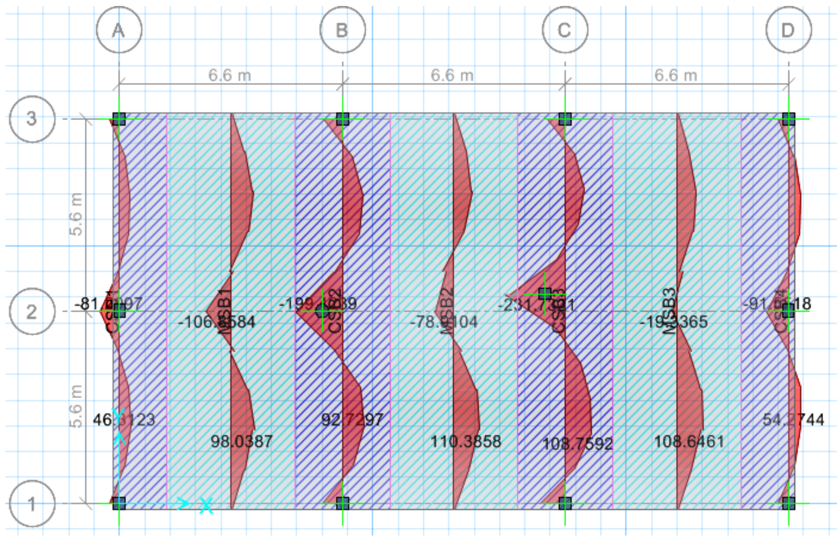


Figure 5.4: C.S & M.S along shorter direction(SAFE:IS,with staggered column)

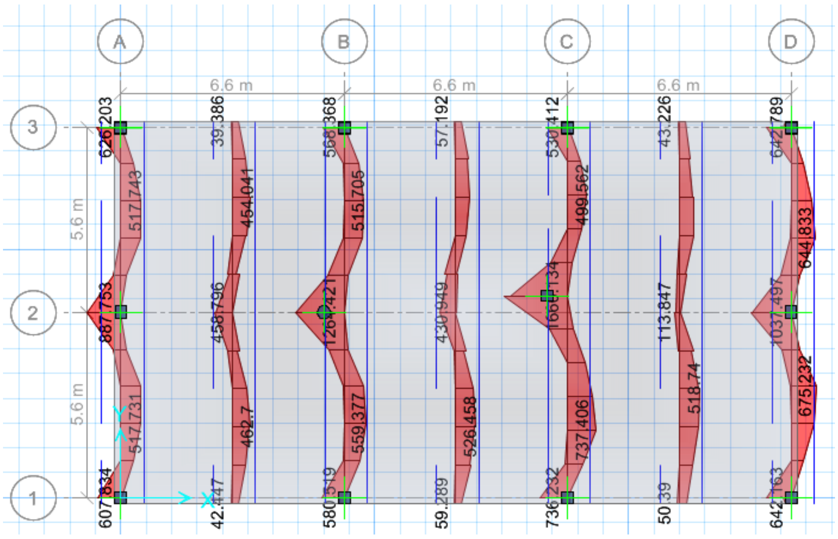


Figure 5.5: Along shorter side reinforcement(SAFE:IS ,with staggered column)

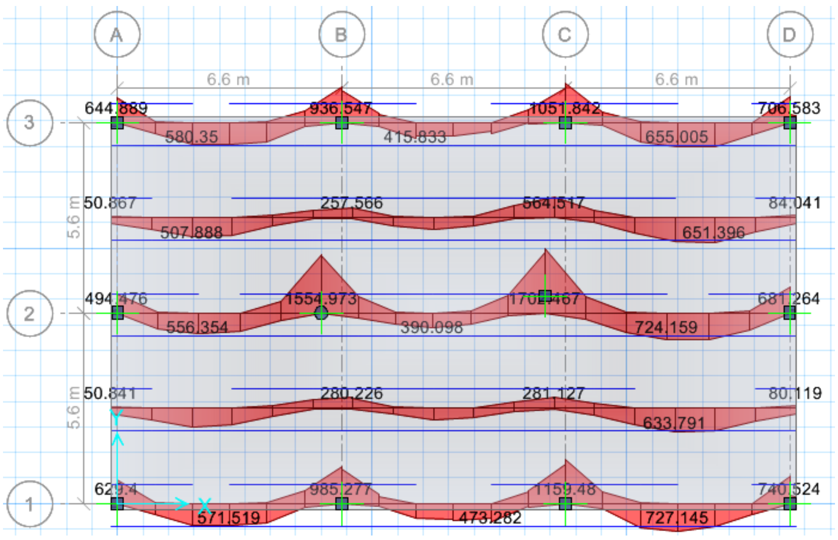


Figure 5.6: Along longer side reinforcement(SAFE:IS ,with staggered column)

Table 5.1: Punching shear result(SAFE:IS ,with staggered column)

column number	ratio	V_u	M_{ux}	M_{uy}	depth	perimeter	location
1	1.662	111.4	8.6	-15.1	189.0	889.2	Corner
2	0.931	240.3	15.3	3.0	189.0	1428.2	Edge
3	1.181	275.9	21.6	-7.0	189.0	1428.2	Edge
4	1.961	115.4	10.8	18.6	189.0	889.2	Corner
5	0.857	232.1	-0.4	-14.1	189.0	1428.2	Edge
6	1.340	606.2	-1.4	3.4	189.0	1818.0	Interior
7	1.348	661.3	-6.6	-10.2	189.0	2156.0	Interior
8	1.115	277.9	-1.7	21.6	189.0	1428.2	Edge
9	1.703	113.0	-9.0	-15.5	189.0	889.2	Corner
10	0.912	232.1	-14.1	4.7	189.0	1428.2	Edge
11	0.932	223.2	-13.8	-8.4	189.0	1428.2	Edge
12	1.886	114.4	-10.3	17.6	189.0	889.2	Corner

5.1.2 Without Staggered Column

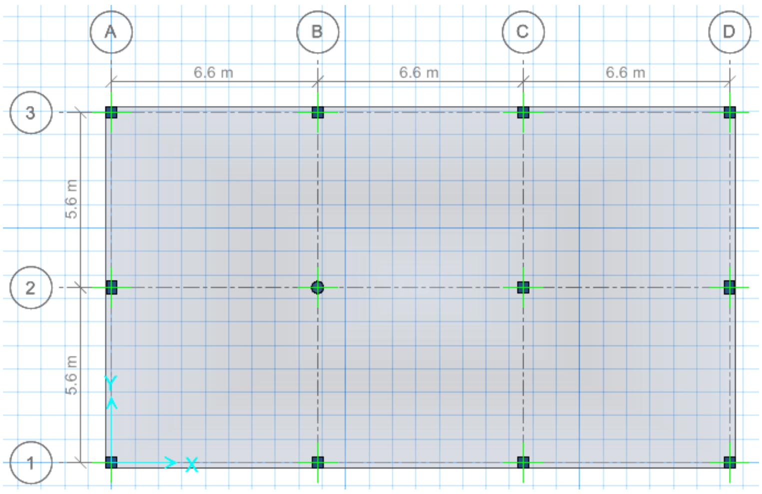


Figure 5.7: Slab model in SAFE(without staggered column)

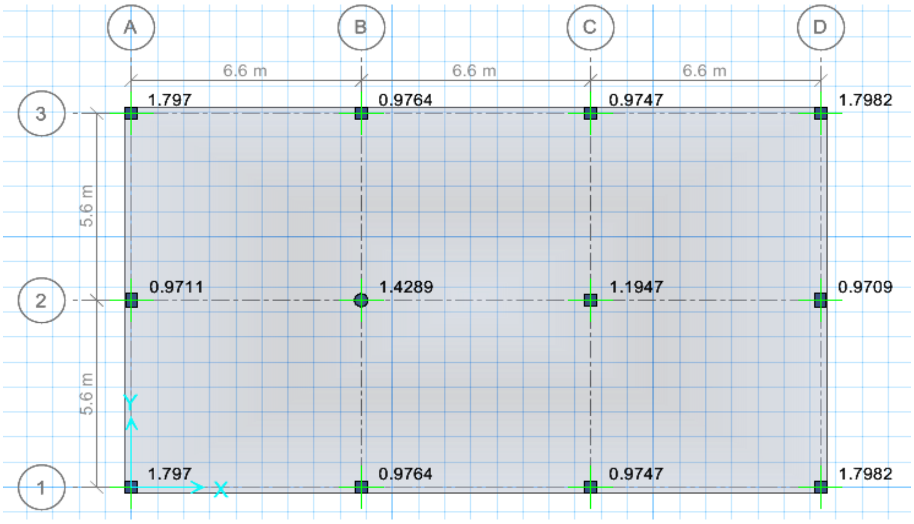


Figure 5.8: Punching shear ratio(SAFE:IS ,without staggered column)

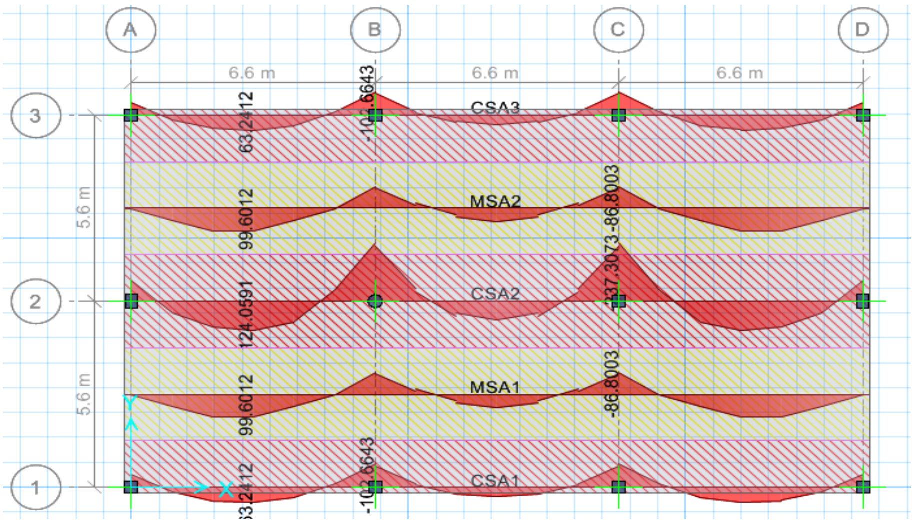


Figure 5.9: C.S & M.S along longer direction(SAFE:IS ,without staggered column)

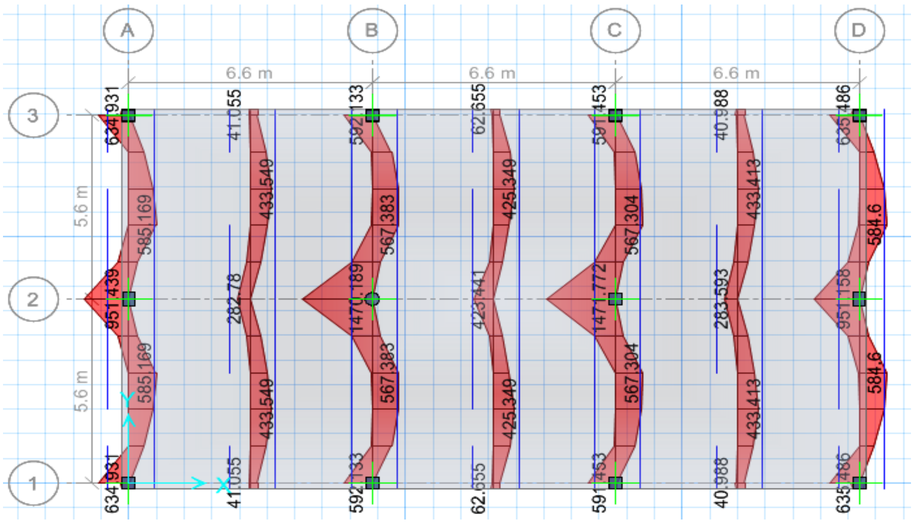


Figure 5.10: C.S & M.S along shorter direction(SAFE:IS ,without staggered column)

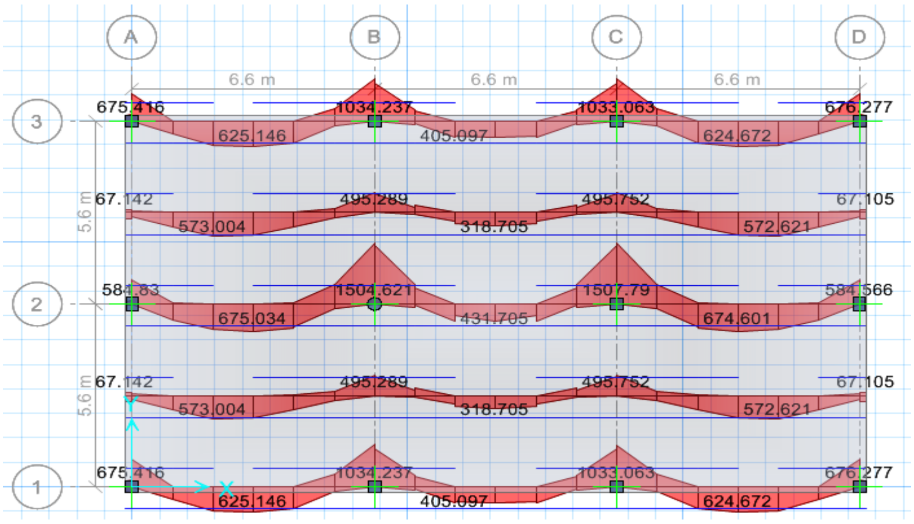


Figure 5.11: Along longer side reinforcement(SAFE:IS ,without staggered column)

Table 5.2: Punching shear result(SAFE:IS ,without staggered column)

column number	ratio	V_u	M_{ux}	M_{uy}	depth	perimeter	location
1	1.797	113.9	9.7	-16.6	189.0	889.2	Corner
2	0.976	242.7	15.7	5.5	189.0	1428.2	Edge
3	0.975	242.5	15.6	-5.4	189.0	1428.2	Edge
4	1.798	114.0	9.6	16.6	189.0	889.2	Corner
5	0.857	232.1	-0.4	-14.1	189.0	1428.2	Edge
6	1.429	628.2	0.0	6.7	189.0	1818.0	Interior
7	1.195	628.5	0.0	-7.1	189.0	2156.0	Interior
8	0.971	255.3	0.0	17.7	189.0	1428.2	Edge
9	1.797	113.9	-9.7	-16.6	189.0	889.2	Corner
10	0.976	242.7	-15.7	5.5	189.0	1428.2	Edge
11	0.975	242.5	-15.6	-5.4	189.0	1428.2	Edge
12	1.798	114.0	-9.6	16.6	189.0	889.2	Corner

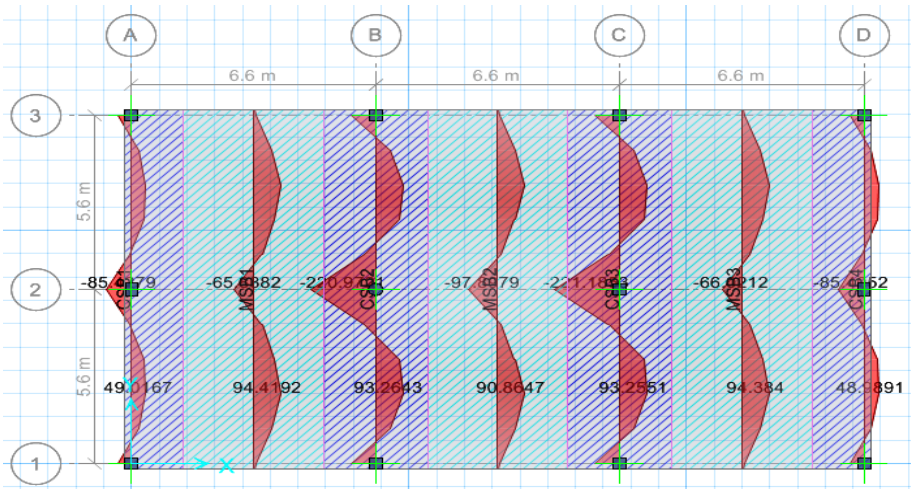


Figure 5.12: Along shorter side reinforcement(SAFE:IS,without staggered column)

5.2 Analysis of Flat Slab as per ACI 318-08

5.2.1 With Staggered Column

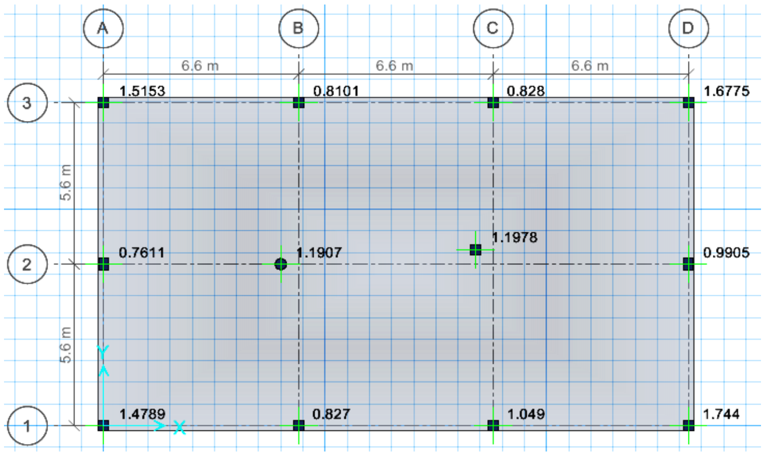


Figure 5.13: Punching shear ratio(SAFE:ACI ,with staggered column)

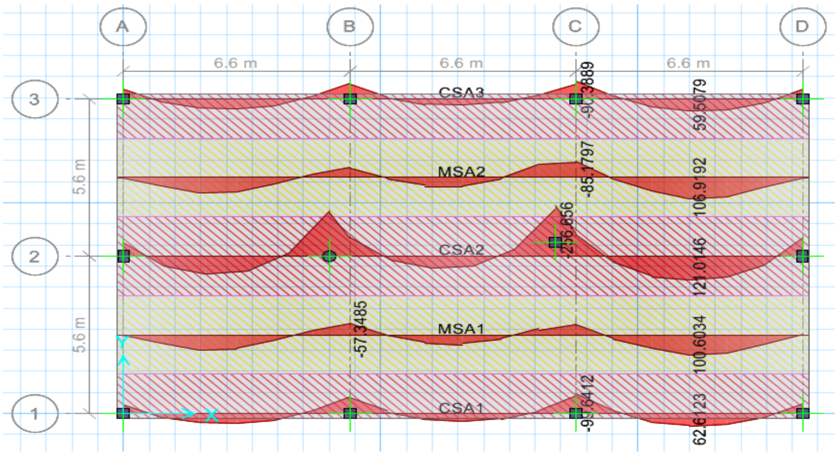


Figure 5.14: C.S & M.S along longer direction(SAFE:ACI ,with staggered column)

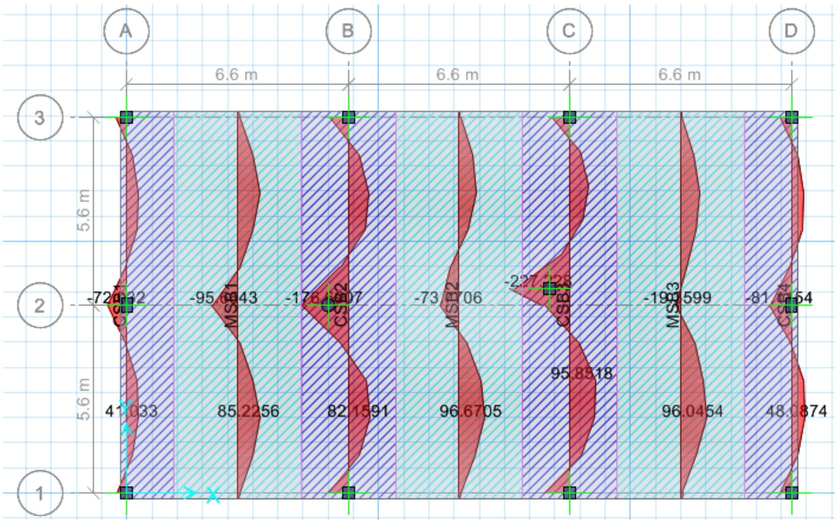


Figure 5.15: C.S & M.S along shorter direction(SAFE:ACI ,with staggered column)

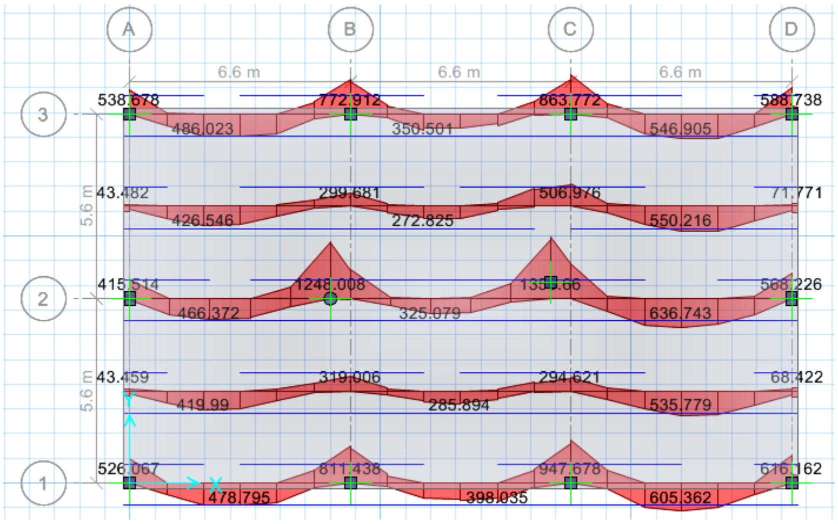


Figure 5.16: Along longer side reinforcement(SAFE:ACI ,with staggered column)

Table 5.3: Punching shear result(SAFE:ACI ,with staggered column)

column number	ratio	V_u	M_{ux}	M_{uy}	depth	perimeter	location
1	1.479	98.5	7.63	-13.42	189.0	889.2	Corner
2	0.827	212.4	13.56	2.68	189.0	1428.2	Edge
3	1.049	244.0	19.21	-6.23	189.0	1428.2	Edge
4	1.744	102.0	9.58	16.49	189.0	889.2	Corner
5	0.761	205.1	-0.39	-12.53	189.0	1428.2	Edge
6	1.191	536.6	-1.23	3.03	189.0	1818.0	Interior
7	1.198	585.7	-5.84	-9.02	189.0	2156.0	Interior
8	0.990	245.7	-1.50	19.15	189.0	1428.2	Edge
9	1.515	99.8	-7.95	-13.71	189.0	889.2	Corner
10	0.810	205.1	-12.50	4.18	189.0	1428.2	Edge
11	0.828	197.2	-12.27	-7.44	189.0	1428.2	Edge
12	1.678	101.1	-9.18	15.63	189.0	889.2	Corner

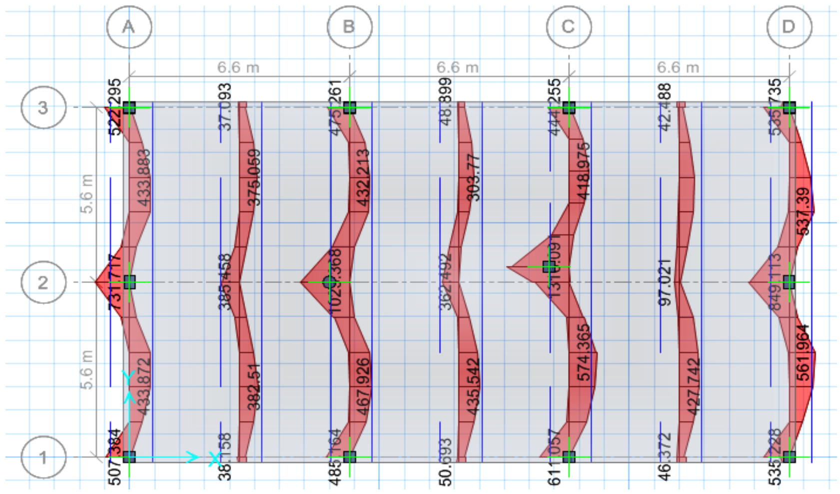


Figure 5.17: Along shorter side reinforcement(SAFE:ACI ,with staggered column)

5.2.2 Without Staggered Column

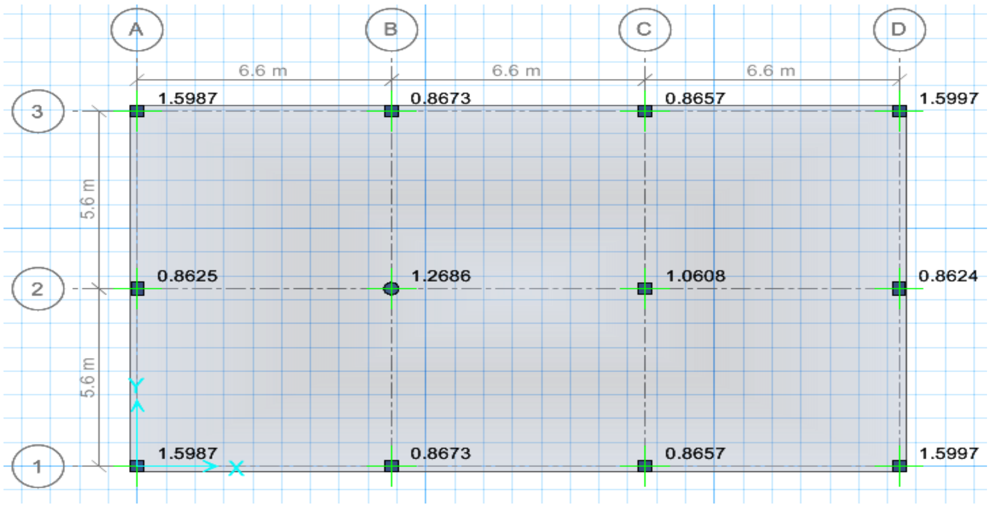


Figure 5.18: Punching shear ratio(SAFE:ACI ,without staggered column)

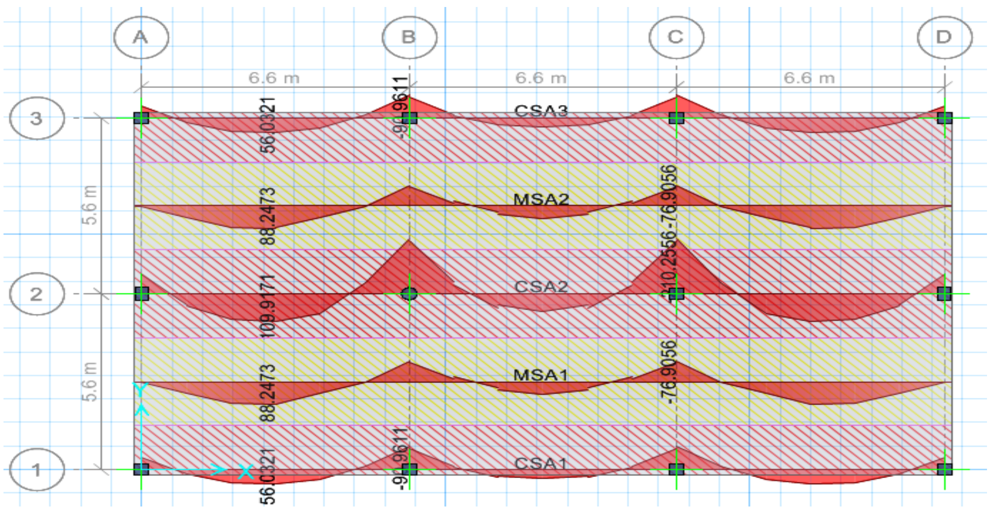


Figure 5.19: C.S & M.S along longer direction(SAFE:ACI ,without staggered column)

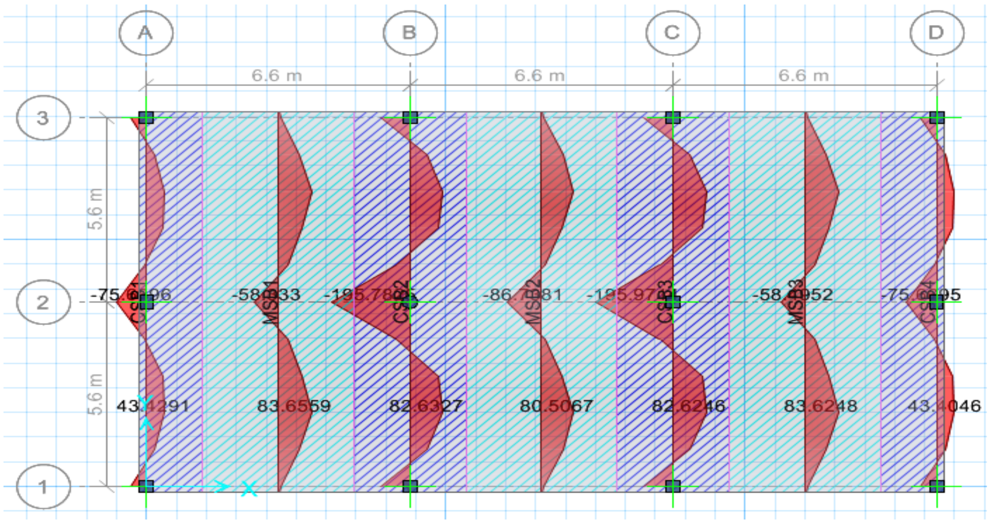


Figure 5.20: C.S & M.S along shorter direction(SAFE:ACI ,without staggered column)

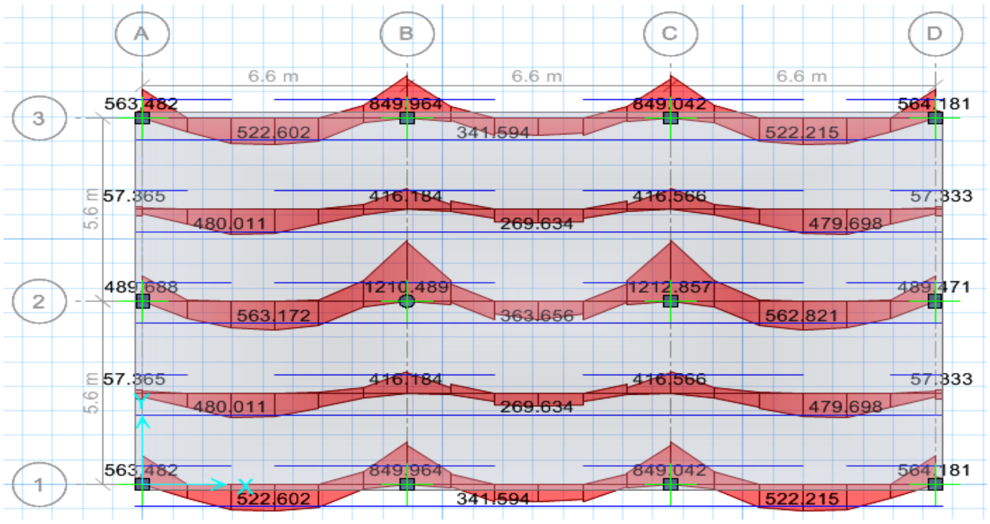


Figure 5.21: Along longer side reinforcement(SAFE:ACI ,without staggered column)

Table 5.4: Punching shear result(SAFE:ACI ,without staggered column)

column number	ratio	V_u	M_{ux}	M_{uy}	depth	perimeter	location
1	1.599	100.7	8.6	-14.7	189.0	889.2	Corner
2	0.867	214.5	13.9	4.9	189.0	1428.2	Edge
3	0.866	214.4	13.9	-4.8	189.0	1428.2	Edge
4	1.600	100.7	8.6	14.7	189.0	889.2	Corner
5	0.862	225.8	0.0	-15.7	189.0	1428.2	Edge
6	1.269	555.7	0.0	5.9	189.0	1818.0	Interior
7	1.061	556.0	0.0	-6.2	189.0	2156.0	Interior
8	0.862	225.7	0.0	15.7	189.0	1428.2	Edge
9	1.599	100.7	-8.6	-14.7	189.0	889.2	Corner
10	0.867	214.5	-13.9	4.9	189.0	1428.2	Edge
11	0.866	214.4	-13.9	-4.8	189.0	1428.2	Edge
12	1.600	100.7	-8.6	14.7	189.0	889.2	Corner

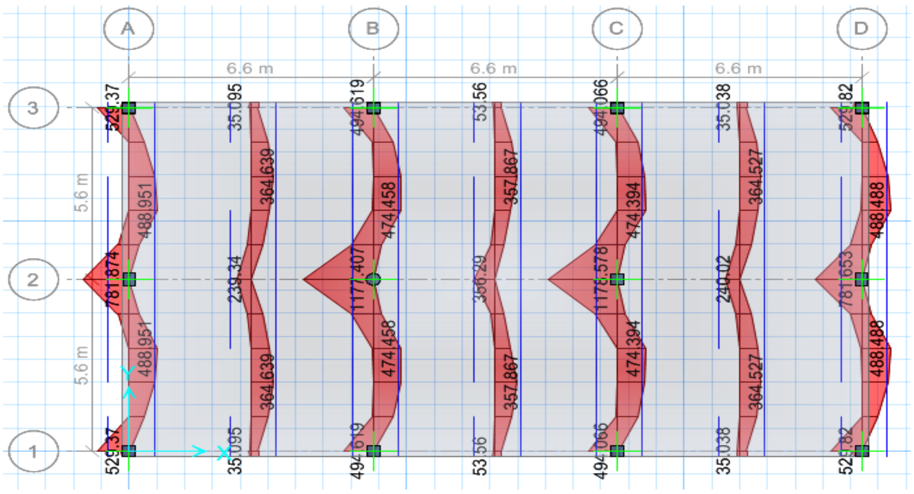


Figure 5.22: Along shorter side reinforcement(SAFE:ACI ,without staggered column)

5.3 Analysis of Flat Slab as per BS 8110-1997

5.3.1 With Staggered Column

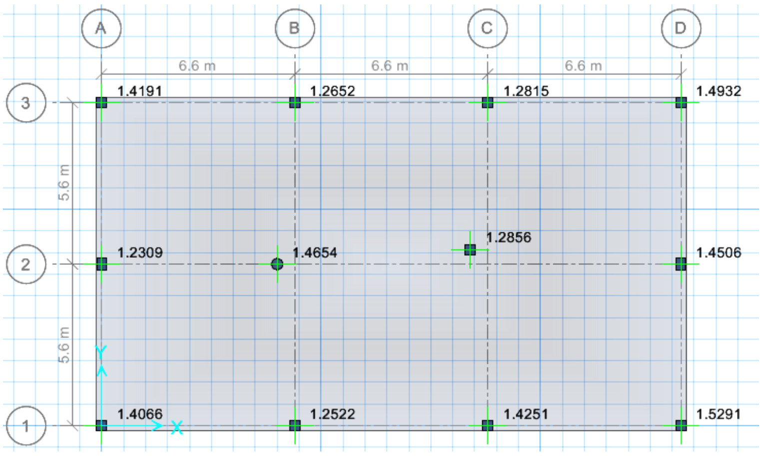


Figure 5.23: Punching shear ratio(SAFE:BS ,with staggered column)

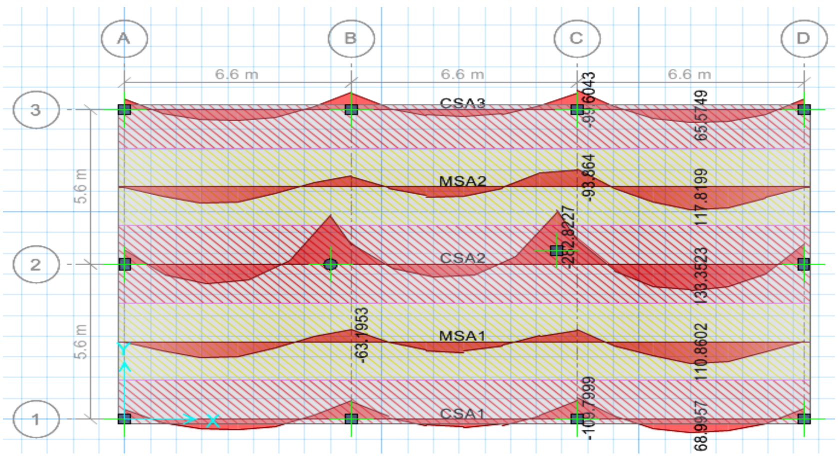


Figure 5.24: C.S & M.S along longer direction(SAFE,with staggered column)

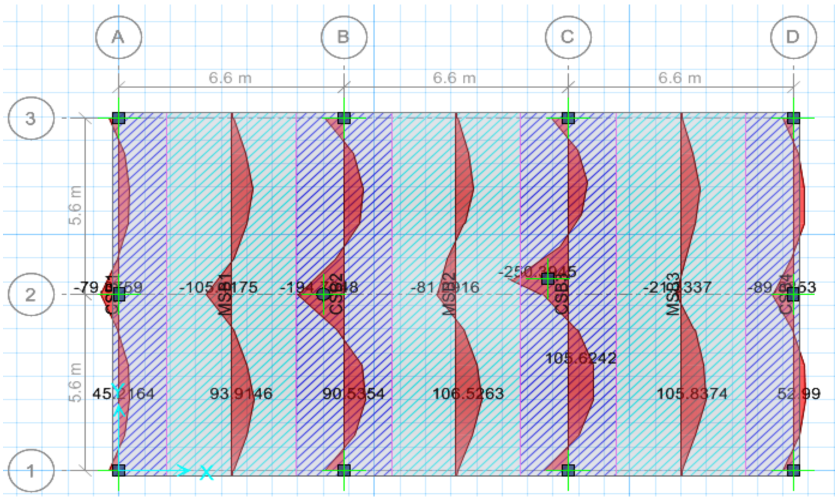


Figure 5.25: C.S & M.S along shorter direction(SAFE:BS ,with staggered column)

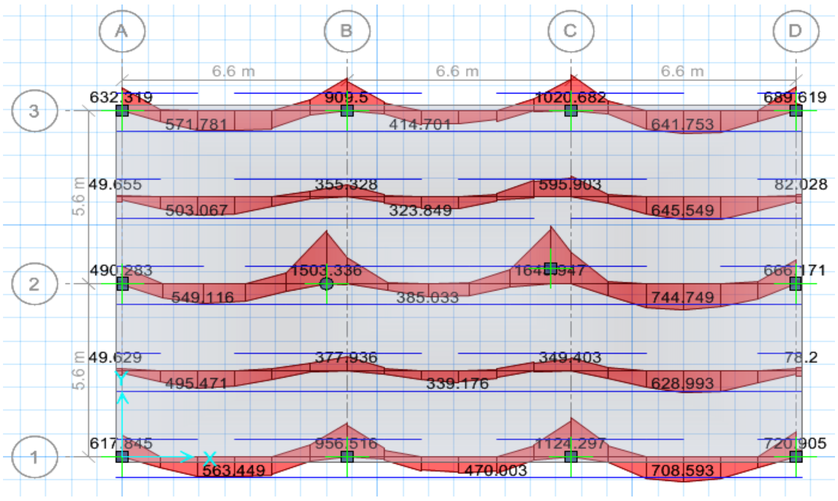


Figure 5.26: Along longer side reinforcement(SAFE:BS ,with staggered column)

Table 5.5: Punching shear result(SAFE:BS ,with staggered column)

column number	ratio	V_u	M_{ux}	M_{uy}	depth	perimeter	location
1	1.407	108.67	5.59	-21.55	189	1267.2	Corner
2	1.252	234.39	5.92	6.97	189	2184.2	Edge
3	1.425	269.16	17.42	-16.23	189	2184.2	Edge
4	1.529	112.56	10.40	29.43	189	1267.2	Corner
5	1.231	226.32	-0.97	-5.99	189	2184.2	Edge
6	1.465	591.61	-3.40	8.34	189	3004.927	Interior
7	1.286	645.55	-16.09	-24.85	189	3668	Interior
8	1.451	271.02	-3.74	19.89	189	2184.2	Edge
9	1.419	110.18	-6.26	-22.13	189	1267.2	Corner
10	1.265	226.33	-3.97	10.88	189	2184.2	Edge
11	1.281	217.66	-4.54	-19.36	189	2184.2	Edge
12	1.493	111.53	-9.45	27.23	189	1267.2	Corner

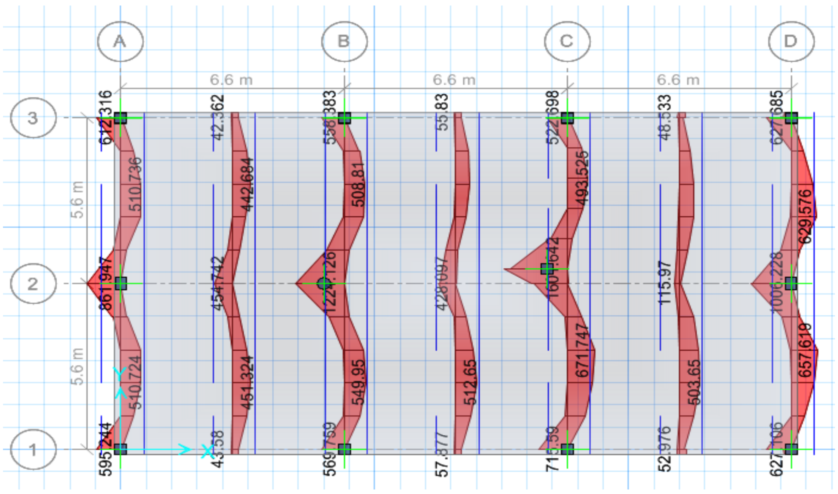


Figure 5.27: Along shorter side reinforcement(SAFE:BS ,with staggered column)

5.3.2 Without Staggered Column

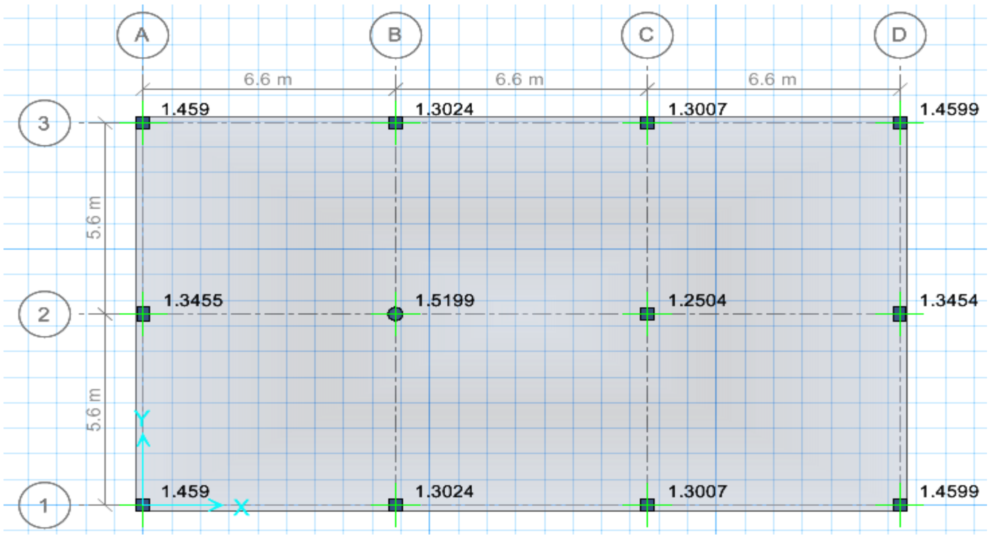


Figure 5.28: Punching shear ratio(SAFE:BS ,without staggered column)

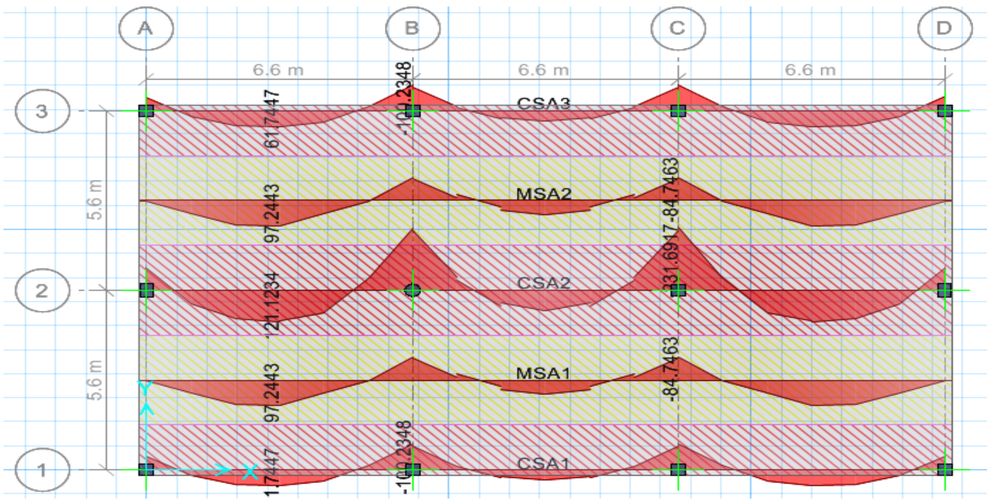


Figure 5.29: C.S & M.S along longer direction(SAFE:BS ,without staggered column)

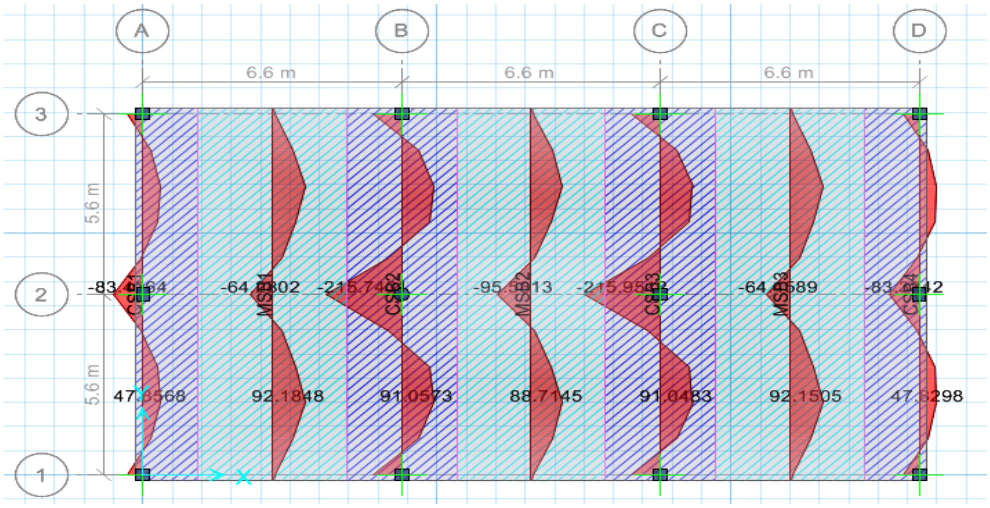


Figure 5.30: C.S & M.S along shorter direction(SAFE:BS ,without staggered column)

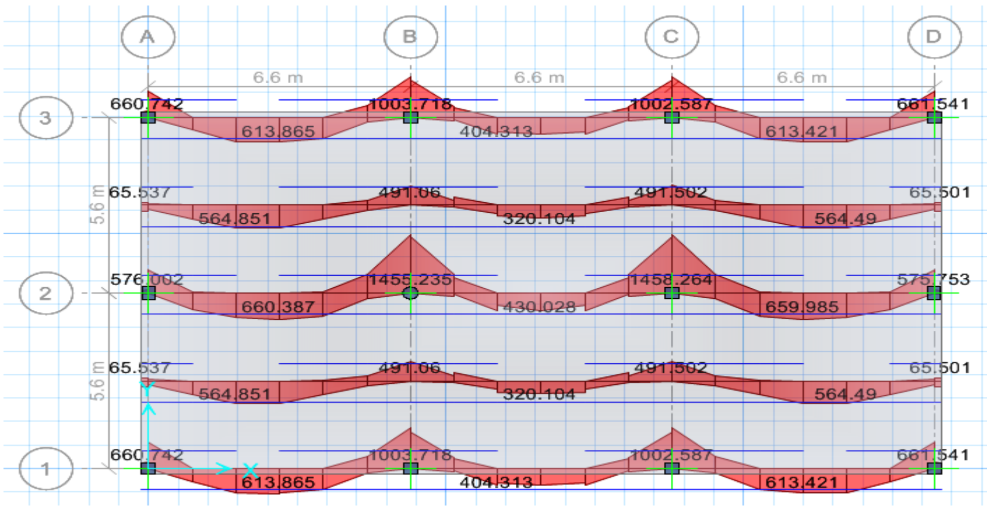


Figure 5.31: Along longer side reinforcement(SAFE:BS ,without staggered column)

Table 5.6: Punching shear result(SAFE:BS ,with staggered column)

column number	ratio	V_u	M_{ux}	M_{uy}	depth	perimeter	location
1	1.4590	111.09	7.83	-24.71	189	1267.2	Corner
2	1.3024	236.67	6.63	12.72	189	2184.2	Edge
3	1.3007	236.53	6.58	-12.55	189	2184.2	Edge
4	1.4599	111.12	7.81	24.77	189	1267.2	Corner
5	1.3455	249.09	0.00	-12.39	189	2184.2	Edge
6	1.5199	612.91	0.00	16.24	189	3004.927	Interior
7	1.2504	613.22	0.00	-17.22	189	3668	Interior
8	1.3454	248.98	0.00	12.44	189	2184.2	Edge
9	1.4590	111.09	-7.83	-24.71	189	1267.2	Corner
10	1.3024	236.67	-6.63	12.72	189	2184.2	Edge
11	1.3007	236.53	-6.58	-12.55	189	2184.2	Edge
12	1.4599	111.12	-7.81	24.77	189	1267.2	Corner

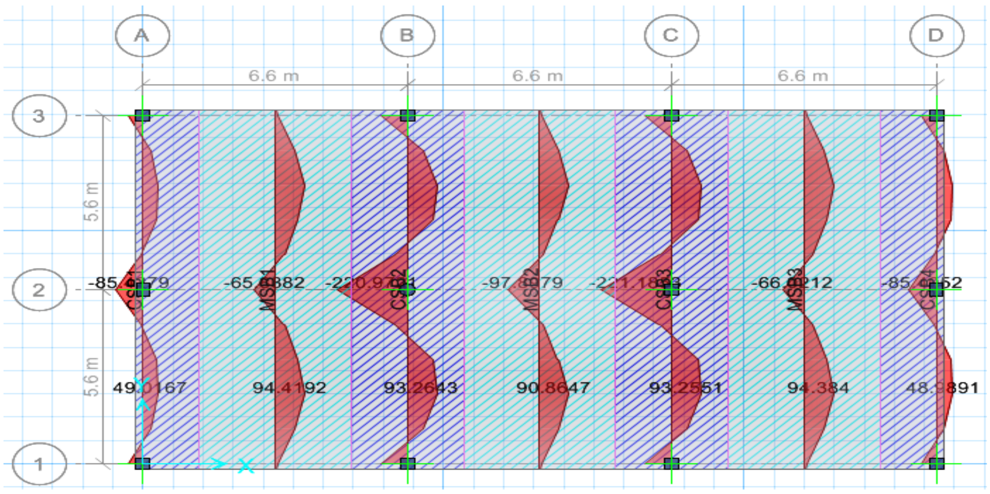


Figure 5.32: Along shorter side reinforcement(SAFE:BS ,without staggered column)

5.4 Analysis of Flat Slab as per EuroCode2:part1-2004

5.4.1 With Staggered Column

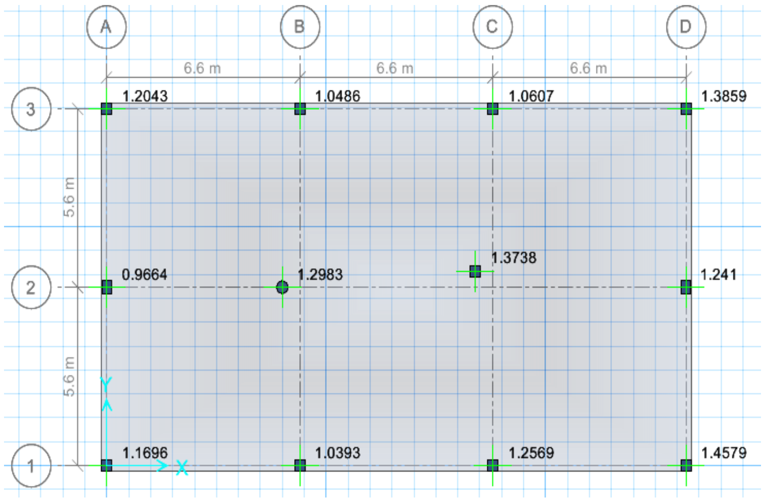


Figure 5.33: Punching shear ratio(SAFE:EC2 ,with staggered column)

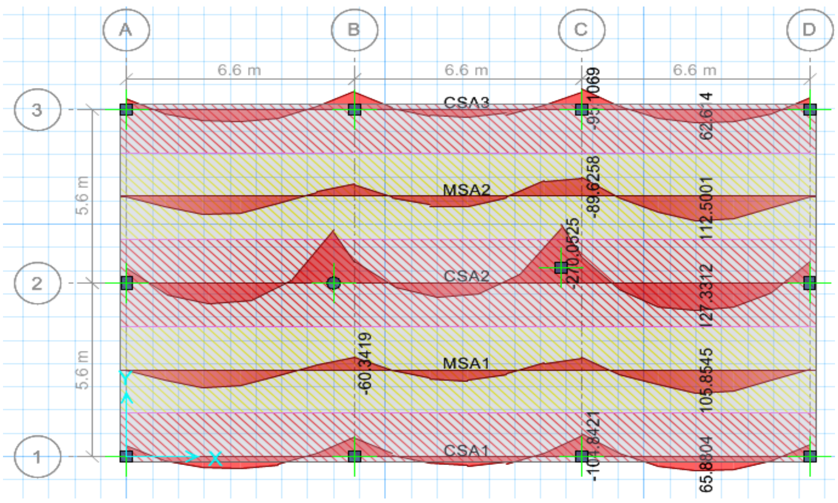


Figure 5.34: C.S & M.S along longer direction(SAFE:EC2 ,with staggered column)

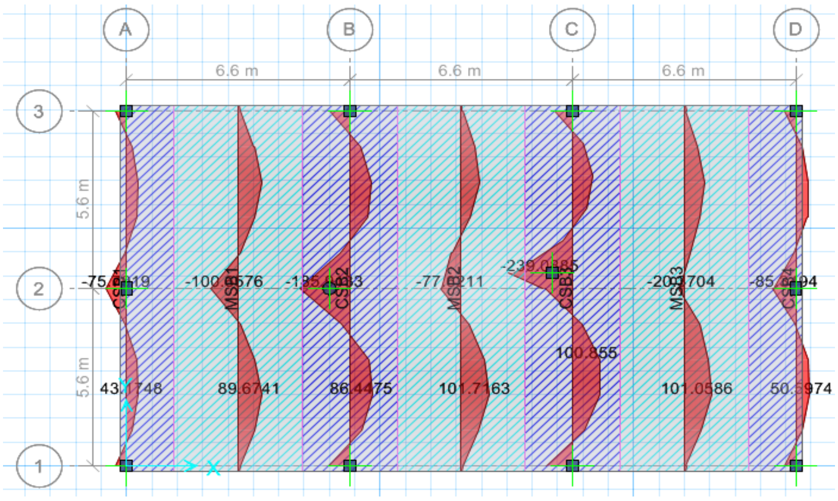


Figure 5.35: C.S & M.S along shorter direction(SAFE:EC2 ,with staggered column)

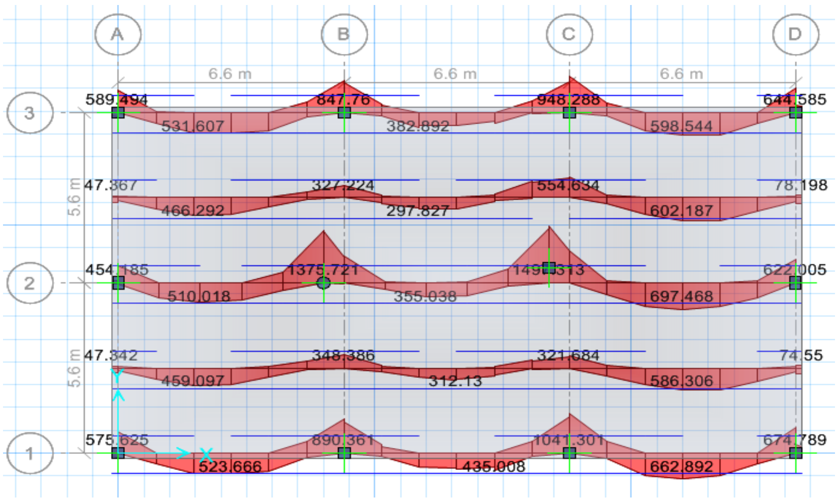


Figure 5.36: Along longer side reinforcement(SAFE:EC2 ,with staggered column)

Table 5.7: Punching shear result(SAFE:EC2 ,with staggered column)

column number	ratio	V_u	M_{ux}	M_{uy}	depth	perimeter	location
1	1.170	103.79	0.70	-9.84	189	1293.65	Corner
2	1.039	223.86	-0.78	4.34	189	2237.099	Edge
3	1.257	257.06	4.15	-10.10	189	2237.099	Edge
4	1.458	107.51	3.37	14.27	189	1293.65	Corner
5	0.966	216.15	-0.62	0.79	189	2237.099	Edge
6	1.298	559.34	-1.95	3.00	189	3598.377	Interior
7	1.374	611.06	-8.42	-15.05	189	3773.798	Interior
8	1.241	258.84	-2.37	4.91	189	2237.099	Edge
9	1.204	105.23	-1.05	-10.14	189	1293.65	Corner
10	1.049	216.16	1.58	6.77	189	2237.099	Edge
11	1.061	207.89	1.18	-12.05	189	2237.099	Edge
12	1.386	106.52	-2.85	13.03	189	1293.65	Corner

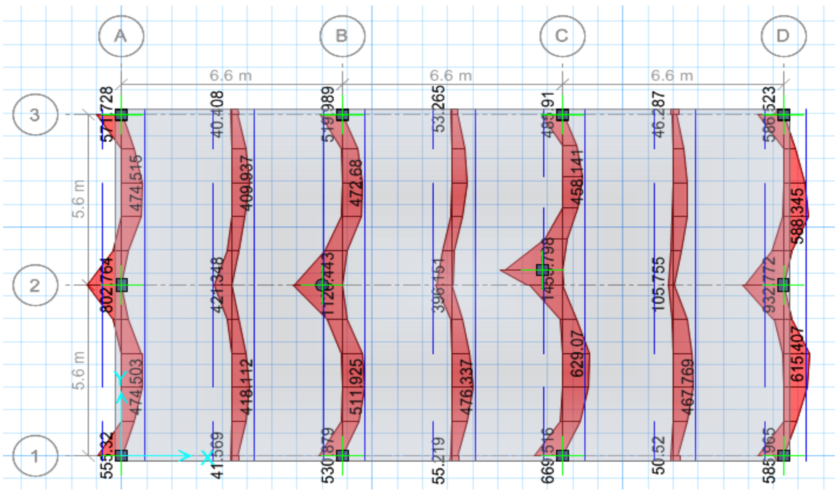


Figure 5.37: Along shorter side reinforcement(SAFE:EC2 ,with staggered column)

5.4.2 Without Staggered Column

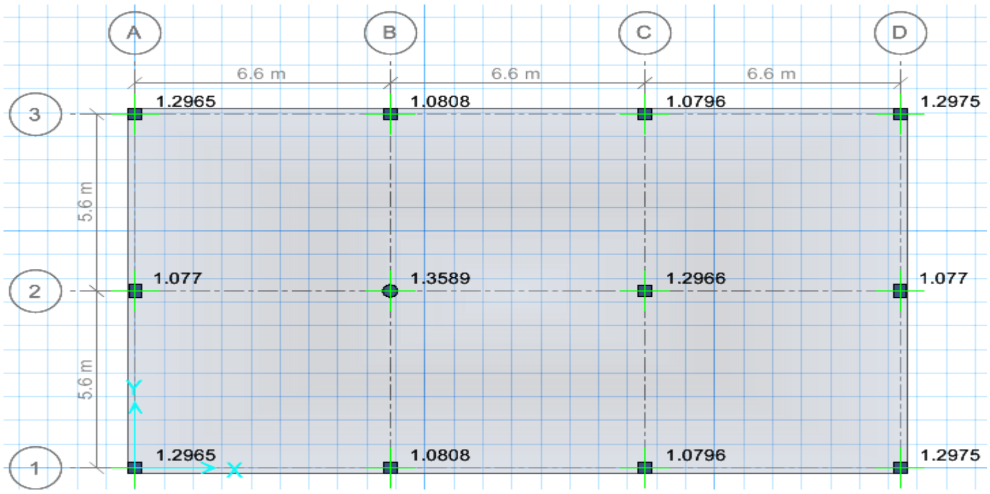


Figure 5.38: Punching shear ratio(SAFE:EC2 ,without staggered column)

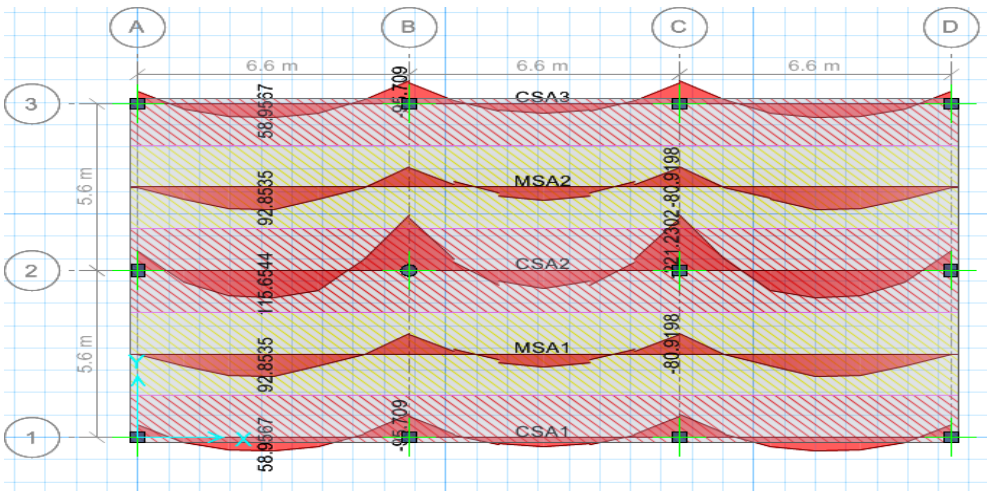


Figure 5.39: C.S & M.S along longer direction(SAFE:EC2 ,without staggered column)

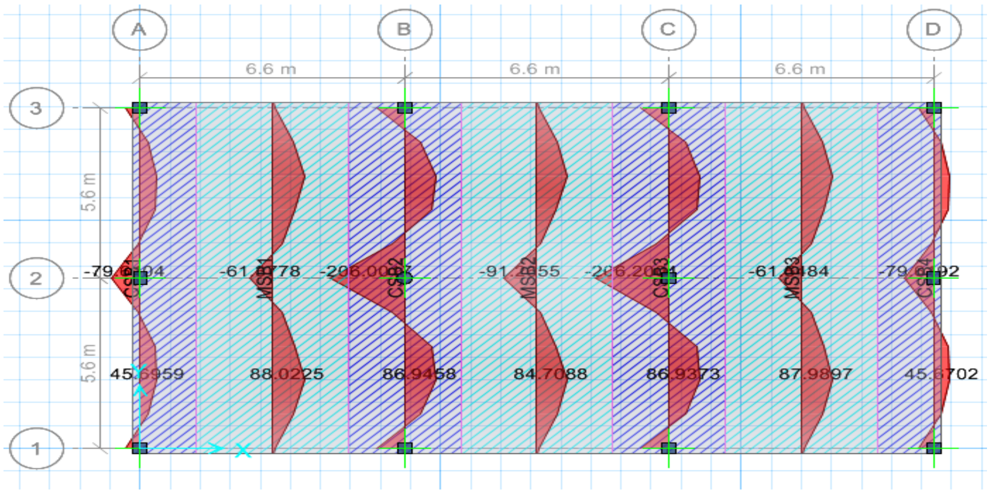


Figure 5.40: C.S& M.S along shorter direction(SAFE:EC2 ,without staggered column)

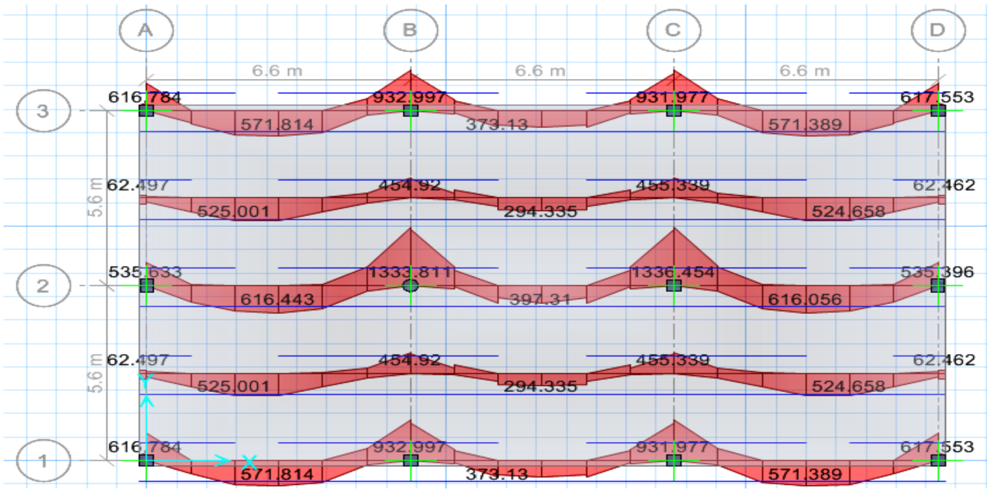


Figure 5.41: Along longer side reinforcement(SAFE:EC2 ,without staggered column)

Table 5.8: Punching shear result(SAFE:EC2 ,with staggered column)

column number	ratio	V_u	M_{ux}	M_{uy}	depth	perimeter	location
1	1.296	106.10	1.93	-11.60	189	1293.65	Corner
2	1.081	226.03	-0.48	7.92	189	2237.099	Edge
3	1.080	225.91	-0.50	-7.81	189	2237.099	Edge
4	1.298	106.13	1.91	11.63	189	1293.65	Corner
5	1.077	237.90	0.00	-1.80	189	2237.099	Edge
6	1.359	585.33	0.00	9.30	189	3598.377	Interior
7	1.297	585.62	0.00	-9.86	189	3773.798	Interior
8	1.077	237.79	0.00	1.83	189	2237.099	Edge
9	1.296	106.10	-1.93	-11.60	189	1293.65	Corner
10	1.081	226.03	0.48	7.92	189	2237.099	Edge
11	1.080	225.91	0.50	-7.81	189	2237.099	Edge
12	1.298	106.13	-1.91	11.63	189	1293.65	Corner

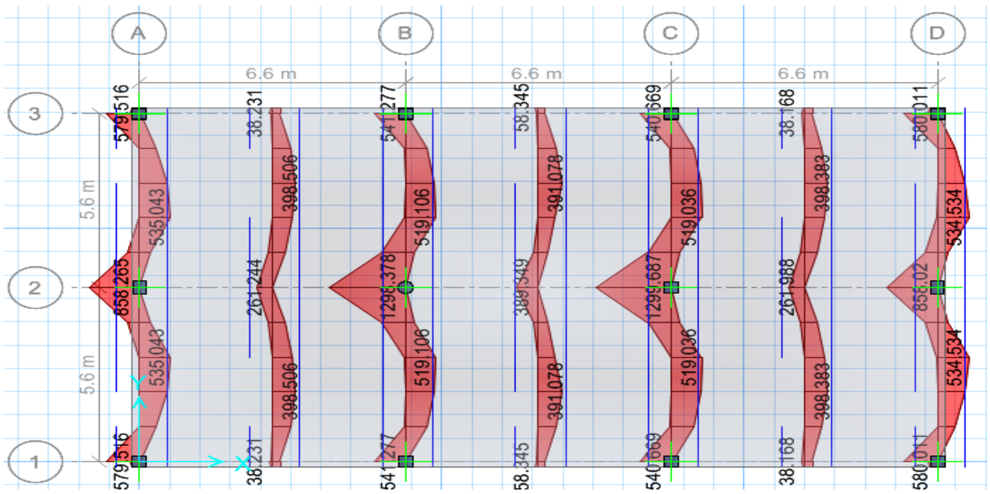


Figure 5.42: Along shorter side reinforcement(SAFE:EC2 ,without staggered column)

Final results :

With Staggered Column

Table 5.9: Along longer side frame(C.S moment)

frame	Code	Exterior span A-B			Interior span B-C			Exterior span C-D		
		M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
1	IS	-53.9	57.8	-97.1	-91.6	45.7	-100.2	-112.5	70.7	-58.9
	ACI	-47.76	51.23	-86.04	-81.17	40.48	-88.78	-99.64	62.61	-52.15
	BS	-52.63	56.45	-94.81	-89.45	44.61	-97.83	-109.80	69.00	-57.47
	Euro	-50.26	53.90	-90.53	-85.41	42.59	-93.42	-104.84	65.88	-54.87
2	IS	-77.1	103.5	-262.2	-250.3	68.6	-271.5	-289.7	136.6	-107.2
	ACI	-68.33	91.72	-232.29	-221.75	60.80	-240.58	-256.66	121.01	-94.99
	BS	-75.30	101.07	-255.98	-244.36	67.00	-265.11	-282.82	133.35	-104.68
	Euro	-71.90	96.51	-244.42	-233.32	63.97	-253.14	-270.05	127.33	-99.95
3	IS	-55.1	58.8	-93.2	-85.2	37.2	-86.6	-102.0	67.2	-57.1
	ACI	-48.80	52.13	-82.57	-75.45	32.97	-76.68	-90.39	59.51	-50.60
	BS	-53.78	57.45	-90.99	-83.14	36.34	-84.50	-99.60	65.57	-55.76
	Euro	-51.35	54.85	-86.88	-79.39	34.69	-80.69	-95.11	62.61	-53.24

Table 5.10: Along shorter side frame(C.S moment)

FRAME	Code	Exterior span A-B			Interior span B-C		
		M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
4	IS	-42.8	46.3	-81.3	-80.3	46.0	-44.2
	ACI	-37.96	41.03	-72.03	-71.12	40.73	-39.16
	BS	-41.83	45.22	-79.38	-78.37	44.88	-43.15
	Euro	-39.94	43.17	-75.79	-74.83	42.86	-41.20
5	IS	-82.4	92.7	-199.4	-196.9	85.1	-79.7
	ACI	-73.00	82.16	-176.69	-174.43	75.37	-70.59
	BS	-80.45	90.54	-194.70	-192.21	83.06	-77.79
	Euro	-76.81	86.45	-185.91	-183.54	79.31	-74.27
6	IS	-105.5	108.2	-107.9	-120.0	80.6	-74.1
	ACI	-93.49	95.85	-95.60	-106.34	71.42	-65.69
	BS	-103.02	105.62	-105.34	-117.18	78.70	-72.39
	Eeuro	-98.37	100.86	-100.59	-111.89	75.15	-69.12
7	IS	-45.7	54.3	-91.6	-87.7	50.0	-45.9
	ACI	-40.53	48.09	-81.12	-77.68	44.34	-40.69
	BS	-44.66	52.99	-89.39	-85.60	48.86	-44.83
	Euro	-42.64	50.60	-85.35	-81.73	46.65	-42.81

Table 5.11: Along longer side frame(M.S moment)

frame	Code	Exterior span A-B			Interior span B-C			Exterior span C-D		
		${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$
1	IS	2.6	86.2	-64.5	-64.7	57.1	-58.2	-62.4	113.5	-0.4
	ACI	2.31	76.35	-57.17	-57.35	50.61	-51.53	-55.24	100.60	-0.38
	BS	2.54	84.14	-62.99	-63.20	55.76	-56.78	-60.88	110.86	-0.42
	Euro	2.43	80.34	-60.15	-60.34	53.25	-54.22	-58.13	105.85	-0.40
2	IS	2.6	87.3	-59.9	-56.4	53.8	-90.3	-96.1	120.7	1.5
	ACI	2.26	77.38	-53.06	-49.99	47.63	-80.03	-85.18	106.92	1.33
	BS	2.49	85.27	-58.47	-55.09	52.49	-88.19	-93.86	117.82	1.46
	Euro	2.38	81.42	-55.83	-52.60	50.12	-84.21	-89.63	112.50	1.40

Table 5.12: Along shorter side frame(C.S moment)

FRAME	Code	Exterior span A-B			Interior span B-C		
		${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$	${M_{ul}}^-$	${M_u}^+$	${M_{ur}}^-$
3	IS	10.5	96.2	-108.0	-106.9	94.2	10.3
	ACI	9.34	85.23	-95.66	-94.68	83.49	9.12
	BS	10.29	93.91	-105.42	-104.33	92.00	10.05
	Euro	9.83	89.67	-100.66	-99.62	87.85	9.60
4	IS	6.9	109.1	-83.3	-78.9	73.7	7.2
	ACI	6.09	96.67	-73.77	-69.92	65.30	6.40
	BS	6.71	106.53	-81.29	-77.04	71.96	7.05
	Euro	6.41	101.72	-77.62	-73.56	68.71	6.73
5	IS	6.1	108.4	-21.9	-19.3	80.8	5.4
	ACI	5.37	96.05	-19.36	-17.13	71.59	4.78
	BS	5.92	105.84	-21.33	-18.88	78.88	5.27
	Euro	5.65	101.06	-20.37	-18.03	75.32	5.03

Without Staggered Column

Table 5.13: Along longer side frame(C.S moment)

frame	Code	Exterior span A-B			Interior span B-C			Exterior span C-D		
		M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
1	IS	-56.1	63.2	-102.7	-93.0	41.6	-93.0	-102.6	63.2	-56.2
	ACI	-49.70	56.03	-90.96	-82.38	36.89	-82.39	-90.88	56.00	-49.76
	BS	-54.76	61.74	-100.23	-90.78	40.65	-90.79	-100.14	61.71	-54.83
	Euro	-52.29	58.96	-95.71	-86.68	38.81	-86.69	-95.62	58.92	-52.36
2	IS	-91.9	124.1	-250.1	-237.5	81.0	-237.3	-250.7	124.0	-91.9
	ACI	-81.41	109.92	-221.59	-210.39	71.80	-210.26	-222.09	109.83	-81.40
	BS	-89.71	121.12	-244.18	-231.84	79.12	-231.69	-244.73	121.03	-89.69
	Euro	-85.66	115.65	-233.16	-221.37	75.55	-221.23	-233.68	115.56	-85.64
3	IS	-56.1	63.2	-102.7	-93.0	41.6	-93.0	-102.6	63.2	-56.2
	ACI	-49.70	56.03	-90.96	-82.38	36.89	-82.39	-90.88	56.00	-49.76
	BS	-54.76	61.74	-100.23	-90.78	40.65	-90.79	-100.14	61.71	-54.83
	Euro	-52.29	58.96	-95.71	-86.68	38.81	-86.69	-95.62	58.92	-52.36

Table 5.14: Along shorter side frame(C.S moment)

FRAME	Code	Exterior span A-B			Interior span B-C		
		M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
4	IS	-45.0	49.0	-85.4	-85.4	49.0	-45.0
	ACI	-39.84	43.43	-75.69	-75.69	43.43	-39.84
	BS	-43.90	47.86	-83.41	-83.41	47.86	-43.90
	Euro	-41.92	45.70	-79.64	-79.64	45.70	-41.92
5	IS	-84.0	93.3	-221.0	-221.0	93.3	-84.0
	ACI	-74.40	82.63	-195.78	-195.78	82.63	-74.40
	BS	-81.99	91.06	-215.74	-215.74	91.06	-81.99
	Euro	-78.28	86.95	-206.00	-206.00	86.95	-78.28
6	IS	-83.9	93.3	-221.2	-221.2	93.3	-83.9
	ACI	-74.31	82.62	-195.97	-195.97	82.62	-74.31
	BS	-81.89	91.05	-215.95	-215.95	91.05	-81.89
	Euro	-78.19	86.94	-206.20	-206.20	86.94	-78.19
7	IS	-45.0	49.0	-85.4	-85.4	49.0	-45.0
	ACI	-39.84	43.40	-75.67	-75.67	43.40	-39.84
	BS	-43.90	47.83	-83.38	-83.38	47.83	-43.90
	Euro	-41.92	45.67	-79.62	-79.62	45.67	-41.92

Table 5.15: Along longer side frame(M.S moment)

frame	Code	Exterior span A-B			Interior span B-C			Exterior span C-D		
		M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
1	IS	1.8	99.6	-92.1	-86.8	57.7	-86.8	-92.2	99.5	1.7
	ACI	1.56	88.25	-81.56	-76.88	51.14	-76.91	-81.66	88.16	1.55
	BS	1.72	97.24	-89.88	-84.72	56.36	-84.75	-89.98	97.15	1.70
	Euro	1.65	92.85	-85.82	-80.89	53.81	-80.92	-85.92	92.76	1.63
2	IS	1.8	99.6	-92.1	-86.8	57.7	-86.8	-92.2	99.5	1.7
	ACI	1.56	88.25	-81.56	-76.88	51.14	-76.91	-81.66	88.16	1.55
	BS	1.72	97.24	-89.88	-84.72	56.36	-84.75	-89.98	97.15	1.70
	Euro	1.65	92.85	-85.82	-80.89	53.81	-80.92	-85.92	92.76	1.63

Table 5.16: Along shorter side frame(C.S moment)

FRAME	Code	Exterior span A-B			Interior span B-C		
		M_{ul}^-	M_u^+	M_{ur}^-	M_{ul}^-	M_u^+	M_{ur}^-
3	IS	8.3	94.4	-65.8	-65.8	94.4	8.3
	ACI	7.33	83.66	-58.33	-58.33	83.66	7.33
	BS	8.07	92.18	-64.28	-64.28	92.18	8.07
	Euro	7.71	88.02	-61.38	-61.38	88.02	7.71
4	Is	7.1	90.9	-97.9	-97.9	90.9	7.1
	ACI	6.32	80.51	-86.74	-86.74	80.51	6.32
	BS	6.96	88.71	-95.58	-95.58	88.71	6.96
	Euro	6.65	84.71	-91.27	-91.27	84.71	6.65
5	IS	8.3	94.4	-66.0	-66.0	94.4	8.3
	ACI	7.34	83.62	-58.50	-58.50	83.62	7.34
	BS	8.09	92.15	-64.46	-64.46	92.15	8.09
	Euro	7.72	87.99	-61.55	-61.55	87.99	7.72

5.5 summary

In this chapter describe design of flat slab (with & without staggered column) using SAFE software. Also comparison of moments and punching shear is carried out.

Chapter 6

Conclusion and Future Scope

6.1 Conclusion

- The positive mid-span moment is increasing and negative moment is decreasing when we analyze the slab with Equivalent Frame Method.

- The negative moment's section shall be designed to resist the larger of the two interior negative design moments for the span framing into common supports.

- Negative & Positive moments at exterior support is increases for IS 456-2000 as compared to (ACI 318-08, BS 8110-1997, EC2:Part1-2004) for Equivalent Frame Method. Also Negative moment at interior support is increases for BS 8110-1997 as compared to other codes.

- In the Exterior support, the total design moments (Mo) are distributed as 100% in column strip and 0% in middle strip in both the case IS 456-2000 and ACI 318-08.Also In the Interior support,the total design moments (Mo) are distributed as 75% in column strip and 25% in middle strip and Positive moments are distributed as 60% in column strip and 40% in middle strip.

- In the Exterior support, the total design moments (Mo) are distributed as 75% in column strip and 25% in middle strip in both the case BS 8110-1197 and EC2-Part1-2004.Also In the Interior support,the total design moments (Mo) are distributed as 75% in column strip and 25% in middle strip and Positive moments are distributed as 55% in column strip and 45% in middle strip.

- In flat slab (with & without staggered column) in both cases the punching shear criteria is satisfy except Interior columns as per IS 456-2004 & ACI 318-08 as compared to other code.

- SAFE takes straight line for calculating of column & middle strip for both cases(with & without staggered column) as shown in Fig.5.39. In manual design takes the typical frame for calculating of column & middle strip as shown Fig.4.7.Also SAFE takes average moment for converting FEM moment to strip moment.

- In flat slab (with & without staggered column) in both cases the flat slab are satisfy against deflection as shown in Table.6.1 as per all codes.

Table 6.1: Deflection results

	IS 456-2000	ACI 318-08	BS 8110-1997	EC2:Part1-2004
Allowable deflection	L/250	L/240	L/250	L/250
Allowable deflection	26.4 mm	27.5 mm	26.4 mm	26.4 mm
Actual deflection	25.51 mm	20 mm	22.027 mm	22.84 mm

6.2 Future Scope

- Analysis and design of flat slab with opening.

- Analysis and design of flat slab due to lateral load

- Brittle failure of flat slab is observed during earthquakes hence the pushover analysis of flat slab can be carried out to study the actual behavior of flat slab during earthquakes.

- Study for flat slab is done only for flat slab with drop and with column capital and Flat slab without drop and without column head hence it can be done for flat slab with only column head and also for flat slab with only drop.

Appendix A

List of Paper Communicated

- Jecky Patel and Shri Himat Solanki, "Analysis and Design of Flat Slab With Staggered Column", 3^{rd} International Conference (NUiCONE, Nirma University, Ahmedabad, 6-8 December 2012. (Abstract Communicated)

References

[1] D A Gasparini,“Contribution of C.A.P turner to Devlopment of Reinforced concrete Flat Slab 1905-1909”, *Journal of Structural engineering* , Vol.128, (2002).

[2] P.C. Varghese *“Advanced Reinforced Concrete Design”*, prentice hall of India limited, New Delhi, EEE (2002).

[3] Dong-Guen Lee, Hyun-Su Kim, Hye-Sook Kim and Seung-Jae Lee “An Efficient Analysis of Flat Slab Structures Using Super-Elements ”, *The Ninth East Asia-Pacific Conference on Structural Engineering and Con struction.*

[4] Bureau of Indian Standards, New Delhi, “*IS 456:2000,Plain and Reinforced Concrete - Code of Practice”*, Fourth Revision, July (2000).

[5] American Concrete Institute, “*ACI 318-08, Building Code Requirements for Structural Concrete and Commentary”*, January (2008).

[6] British Standard, “*BS 8110-1:1997,Code of practice for design and construction”*, March (1997).

[7] European Standard, “*Eurocode 2:Part1-2004,Design of concrete structures - Part 1-1: General 2rules and rules for buildings”*, December (2004).

[8] M.Anitha ,B.Q.Rahman and JJ.Vijay, “Analysis and Design of Flat Slabs Using Various Codes”,*International Institute of Information Technology, Hyderabad*, April (2007).

[9] K.Baskaran "Irregular Flat Slabs Designed According to Structural Membrane Approach",*Morley Symposium on Concrete Plasticity and its Application, University Cambridge*, July (2007).

[10] H.-S. Kim, D.-G. Lee "Efficient analysis of flat slab structures subjected to lateral loads", Sungkyunkwan University, Republic of Korea, Engineering Structures 27 (2005), pp251-263.

[11] Gowda N Bharath, Gowda S. B. Ravishankar, A.V Chandrashekar "Review and Design of Flat Plate/Slabs Construction in India", *undated paper.*

[12] Murray K.A.,Cleland D.J. and Gilbert S.G "The Development of a Prediction Method for the Ultimate Load Capacity of Reinforced Concrete Flat Slabs in the Vicinity of Edge Columns", *Proceedings of the* 2^{nd} *International Congress , Italy*,June (2006).

[13] W.Gene Corley, James O. Jirsa "Equivalent Frame Analysis for Flat Slab Design", *ACI Journal*, November (1970).

[14] Sidney H. Symmonds, Janko Misic "Design factors for the Equivalent Frame Method", *ACI Journal*, November (1971).

[15] Ibrahim. S. Vepari "Study on Economical Aspects of Long Span Slabs", *National Conference on Recent Trends in Engineering & Technology*, May (2011).

[16] Maurice P. van Buren "Staggered Columns in Flat Plates",*Proceedings of the American Society of Civil Engineers, Journal of the Structural Division*, June (1971).

[17] A.S. Santhi, J. Prasad and A.K. Ahuja "Effects of Creep and Shrinkage on the Deflection of RCC Two Way Flat Plates", *Asian Journal of Civil Engineering (Building and Housing)* VOL. 8, NO. 3 (2007), pp267-282.

[18] S Unnikrishna Pillai, Devdas Menon *"Reinforced Concrete Design"*, Tata McGraw-Hill Publishing Compny Limited, Eighth reprint (2007).

[19] Prab Bhatt, Thomas J.MacGinley and Ban Seng Choo *"Reinforced Concrete Design theory and examples"*, 3^{rd} edition, (2006).

[20] The Institution of Structural Engineers *"Manual for the Design of Concrete Building Structures to Eurocode 2"*, September (2006).

[21] A. Malhas, Adeeb Rahman "A Comparative Investigation of the Provisions of the ACI and EC2 for Flextural Deflection", *undated paper.*

[22] ESE Soedarsono HS *"Design and Detailing of Flat Slab"*, FEBRUARY (2002).

[23] British Cement Association *"Deflections in Concrete Slabs and Beams"*, Concrete Society Technical Report No. 58, (2005).

[24] R Moss, O Brooker *"How to Design Concrete Structures Using Eurocode2"*, The Concrete Centre, September (2009).

[25] H.J.Shah *"Reinforced Concrete"*, Charotar Publications, Fifth edition (2008).

[26] S.N. Sinha *"Reinforced Concrete Design"*, Second edition, Tata Mc-graw hill.(2002).