ANALYSIS AND DESIGN OF FLAT SLABS USING VARIOUS CODES

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Under the guidance of
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HYDERABAD
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CERTIFICATE

This is to certify that the project entitled “ANALYSIS AND DESIGN OF FLAT SLABS USING VARIOUS CODES” submitted as partial fulfillment for the award of Masters of Technology in Computer Aided Structural Engineering, IIIT-Hyderabad is a bonafied work done by M.Anitha, B.Q.Rahman, JJ.VIJAY

First year second semester students during the year 2006-2007.

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ACKNOWLEDGEMENT

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ABSTRACT

Flat slabs system of construction is one in which the beams used in the conventional methods of constructions are done away with. The slab directly rests on the column and load from the slab is directly transferred to the columns and then to the foundation. To support heavy loads the thickness of slab near the support with the column is increased and these are called drops, or columns are generally provided with enlarged heads called column heads or capitals.

Absence of beam gives a plain ceiling, thus giving better architectural appearance and also less vulnerability in case of fire than in usual cases where beams are used.

Plain ceiling diffuses light better, easier to construct and requires cheaper form work.

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.

The aim of this project is to try and illustrate the methods used for flat slab design using ACI-318, NZ-3101, and Eurocode2 and IS: 456 design codes.

For carrying out this project an interior panel of a flat slab with dimensions 6.6 x 5.6 m and super imposed load 7.75 KN/m² was designed using the codes given above.
**Introduction**

**Basic definition of flat slab:** In general normal frame construction utilizes columns, slabs & Beams. However it may be possible to undertake construction without providing beams, in such a case the frame system would consist of slab and column without beams. These types of slabs are called flat slab, since their behavior resembles the bending of flat plates.

**Components of flat slabs:**

**Drops:** To resist the punching shear which is predominant at the contact of slab and column support, the drop dimension should not be less than one-third of panel length in that direction.

**Column heads:**

Certain amount of negative moment is transferred from the slab to the column at the support. To resist this negative moment the area at the support needs to be increased. This is facilitated by providing column capital/heads.

![Flat slab with drop panel & column head](image)
Design of flat slabs by IS: 456

The term flat slab means a reinforced concrete slab with or without drops, supported generally without beams, by columns with or without flared column heads (see Fig. 12). A flat slab may be solid slab or may have recesses formed on the soffit so that the soffit comprises a series of ribs in two directions. The recesses may be formed by removable or permanent filler blocks.

Components of flat slab design:

a) Column strip :
Column strip means a design strip having a width of 0.25 $l_1$, but not greater than 0.25 $l_1$, on each side of the column centre-line, where $l_1$ is the span in the direction moments are being determined, measured centre to centre of supports and $l_2$ is the-span transverse to $l_1$, measured centre to centre of supports.

b) Middle strip :
Middle strip means a design strip bounded on each of its opposite sides by the column strip.

c) Panel:
Panel means that part of a slab bounded on each of its four sides by the centre-line of a Column or centre-lines of adjacent-spans.

Division into column and middle strip along:

<table>
<thead>
<tr>
<th>Longer span</th>
<th>Shorter span</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_1 = 6.6$ m, $L_2 = 5.6$ m</td>
<td>$L_1 = 5.6$ m, $L_2 = 6.6$ m</td>
</tr>
<tr>
<td>(i) column strip</td>
<td>(i) column strip</td>
</tr>
<tr>
<td>$= 0.25 L_2 = 1.4$ m</td>
<td>$= 0.25 L_2 = 1.65$ m</td>
</tr>
<tr>
<td>But not greater than $0.25 L_1 = 1.65$ m</td>
<td>But not greater than $0.25 L_1 = 1.4$ m</td>
</tr>
<tr>
<td>(ii) Middle strip</td>
<td>(ii) Middle strip</td>
</tr>
<tr>
<td>$= 5.6 - (1.4 + 1.4) = 2.8$ m</td>
<td>$= 6.6 - (1.4 + 1.4) = 3.8$ m</td>
</tr>
</tbody>
</table>
The 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, the width of drops at right angles to the non-continuous edge and measured from the centre-line of the columns shall be equal to one-half the width of drop for interior panels.

Since the span is large it is desirable to provide drop.

Drop dimensions along:

<table>
<thead>
<tr>
<th>Longer span</th>
<th>Shorter span</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_1 = 6.6,\text{m}$, $L_2 = 5.6,\text{m}$</td>
<td>$L_1 = 5.6,\text{m}$, $L_2 = 6.6,\text{m}$</td>
</tr>
<tr>
<td>Not less than $L_1/3 = 2.2,\text{m}$</td>
<td>Not less than $L_1/3 = 1.866,\text{m}$</td>
</tr>
</tbody>
</table>

Hence provide a drop of size $2.2 \times 2.2\,\text{m}$ i.e. in column strip width.

e) column head:

Where column heads are provided, that portion of a column head which lies with in the largest right circular cone or pyramid that has a vertex angle of $90^\circ$ and can be included entirely within the outlines of the column and the column head, shall be considered for design purposes (see Fig. 2).
Fig 2:

Column head dimension along:

<table>
<thead>
<tr>
<th>Longer span</th>
<th>Shorter span</th>
</tr>
</thead>
<tbody>
<tr>
<td>$L_1 = 6.6 \text{ m}$ , $L_2 = 5.6 \text{ m}$</td>
<td>$L_1 = 5.6 \text{ m}$ , $L_2 = 6.6 \text{ m}$</td>
</tr>
<tr>
<td>Not greater than $L_i/4 = 1.65 \text{ m}$</td>
<td>Not greater than $L_i/4 = 1.4 \text{ m}$</td>
</tr>
</tbody>
</table>

Adopting the diameter of column head = 1.30 m = 1300 mm

f) Depth of flat slab:

The thickness of the flat slab up to spans of 10 m shall be generally controlled by considerations of span ($L$) to effective depth ($d$) ratios given as below:

Cantilever 7; simply supported 20; Continuous 26

For slabs with drops, span to effective depth ratios given above shall be applied directly; otherwise the span to effective depth ratios in accordance with above shall be multiplied by 0.9. For this purpose, the longer span of the panel shall be considered. The minimum thickness of slab shall be 125 mm.
Depth of flat slab:

Considering the flat slab as a continuous slab over a span not exceeding 10 m

\[ \frac{L}{d} = 26 \Rightarrow d = \frac{L}{26} \]

Depth considering along:

<table>
<thead>
<tr>
<th>Longer span</th>
<th>Shorter span</th>
</tr>
</thead>
<tbody>
<tr>
<td>( L_1 = 6.6 \text{ m} ), ( L_2 = 5.6 \text{ m} )</td>
<td>( L_1 = 5.6 \text{ m} ), ( L_2 = 6.6 \text{ m} )</td>
</tr>
<tr>
<td>( d = \frac{L}{26} = \frac{6600}{26} = 253.8 \text{ mm} )</td>
<td>( d = \frac{L}{26} = \frac{5600}{26} = 215.3 \text{ mm} )</td>
</tr>
<tr>
<td>Say 260 mm</td>
<td>Say 220 mm</td>
</tr>
</tbody>
</table>

Taking effective depth of 25mm
Overall depth \( D = 260 + 25 = 285 \text{ mm} \) > 125 mm (minimum slab thickness as per IS: 456)

\( \therefore \) It is safe to provide depth of 285 mm.

g) Estimation of load acting on the slab:

Dead load acting on the slab = 0.285 x 25 = 6.25 \( KN/m^2 = W_{d1} \)

Floor finishes etc. load on slab = 1.45 \( KN/m^2 = W_{d2} \)

Live load on slab = 7.75 \( KN/m^2 = W_l \)

Total dead load = \( W_{d1} + W_{d2} = 7.7 \) \( KN/m^2 = W_d \)
The design live load shall not exceed three times the design dead load.

\[
\frac{w_l}{w_d} = \frac{7.75}{7.7} = 1.006 < 3 \therefore \text{OK}
\]

Total design load = \(w_d + w_l = 15.45 \, \text{KN/m}^2\)

h) Total Design Moment for a Span

The absolute sum of the positive and average and is given by negative bending moments in each direction shall be taken as:

\[
M_0 = \frac{W l_n}{8}
\]

\(M_0\) = total moment.
\(W\) = design load on an area \(l_1l_2\)
\(l_n\) = clear span extending from face to face of columns, capitals, brackets or walls, but not less than 0.65 \(l_1\)
\(l_1\) = length of span in the direction of \(M_0\).
\(l_2\) = length of span transverse to \(l_1\).

Circular supports shall be treated as square supports having the same area.
Equivalent side of the column head having the same area:

\[
a = \frac{\pi}{4} d^2 = \frac{\pi}{4} (1.3)^2 = 1.152m
\]

Clear span along long span = \(l_n = 6.6 - \frac{1}{2}(1.152) - \frac{1}{2}(1.152) = 5.448 \, m > 4.29\)
(Should not be less than 0.65 \(l_1\) ) \(\therefore\text{OK}\)

Clear span along long span = \(l_n = 5.6 - \frac{1}{2}(1.152) - \frac{1}{2}(1.152) = 4.44 \, m > 3.64 \, m\)
(Should not be less than 0.65 \(l_1\) ) \(\therefore\text{OK}\)
Total design load along:

<table>
<thead>
<tr>
<th>Longer span</th>
<th>Shorter span</th>
</tr>
</thead>
<tbody>
<tr>
<td>$l_1 = 5.448 \text{ m}, l_2 = 5.6 \text{ m}$</td>
<td>$l_1 = 4.44 \text{ m}, l_2 = 6.6 \text{ m}$</td>
</tr>
<tr>
<td>$W = w l_2 l_n$</td>
<td>$W = w l_2 l_n$</td>
</tr>
<tr>
<td>$W = 15.45 \times 5.6 \times 5.448 = 471.36 \text{ KN}$</td>
<td>$W = 15.45 \times 6.6 \times 4.44 = 452.74 \text{ KN}$</td>
</tr>
</tbody>
</table>

The absolute sum of $-ve$ and $+ve$ moment in a panel along:

<table>
<thead>
<tr>
<th>Longer span</th>
<th>Shorter span</th>
</tr>
</thead>
<tbody>
<tr>
<td>$l_1 = 5.448 \text{ m}, l_2 = 5.6 \text{ m}$</td>
<td>$l_1 = 4.44 \text{ m}, l_2 = 6.6 \text{ m}$</td>
</tr>
<tr>
<td>$M_0 = \frac{W l_2}{8} = \frac{471.36 \times 5.448}{8}$</td>
<td>$M_0 = \frac{W l_2}{8} = \frac{452.74 \times 4.44}{8}$</td>
</tr>
<tr>
<td>$M_0 = 320.99 \text{ KN m}$</td>
<td>$M_0 = 251.2 \text{ KN m}$</td>
</tr>
</tbody>
</table>

(i) Negative and Positive Design Moments:

The negative design moment shall be at the face of rectangular supports, circular supports being treated as square supports having the same 31.4.5.1 Columns built integrally with the slab system area. Shall be designed to resist moments arising from loads.

In an interior span, the total design moment $M_0$ shall be distributed in the following proportions:

Negative design moment 0.65
Positive design moment 0.35

In an end span, the total design moment $M_0$ shall be distributed in the following proportions:

Interior negative design moment: $0.75 \frac{0.10}{1 + \frac{1}{\alpha_c}}$

Positive design moment: $0.63 \frac{0.28}{1 + \frac{1}{\alpha_c}}$
Exterior negative design moment: \( \frac{0.65}{1 + \frac{1}{\alpha_c}} \)

\( \alpha_c \) 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 and is given by:

\[
\alpha_c = \frac{\sum K_c}{K_s}
\]

\( K_c \) = sum of the flexural stiffness of the columns meeting at the joint.

\( K_s \) = flexural stiffness of the slab, expressed as 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 \( M_0 \) for the panel in the direction considered is not less than that required by:

\[
M_0 = \frac{W l_s}{8}
\]

The negative moment section shall be designed to resist the larger of the two interior negative design moments determined for the spans framing into a common support unless an analysis is made to distribute the unbalanced moment in accordance with the stiffness of the adjoining parts.

Column strip:
Negative moment at an interior support: At an interior support, the column strip shall be designed to resist 75 percent of the total negative moment in the panel at that support.

Negative moment at an exterior support:

a) At an exterior support, the column strip shall be designed to resist the total negative moment in the panel at that support.

b) Where the exterior support consists of a column or a wall extending for a distance equal to or greater than three-quarters of the value of \( l_2 \). The length of span transverse to the direction moments are being determined, the exterior negative moment shall be considered to be uniformly distributed across the length \( l_2 \).

Positive moment for each span: For each span, the column strip shall be designed to resist 60 percent of the total positive moment in the panel.

Moments in the middle strip:

a) That portion of the design moment not resisted by the column strip shall be assigned to the adjacent middle strips.
b) Each middle strip shall be proportioned to resist the sum of the moments assigned to its two half middle strips. c) The middle strip adjacent and parallel to an edge supported by a wall shall be proportioned, to resist twice the moment assigned to half the middle strip corresponding to the first row of interior columns.

Stiffness calculation:

let the height of the floor = 4.0 m

clear height of the column = height of floor – depth of drop – thickness of slab – thickness of head.

= 4000 – 140 – 285 – 300 = 3275 mm

Effective height of column = 0.8 x 3275 = 2620 mm

(Assuming one end hinged and other end fixed)

stiffness coefficient

\[
\alpha_c = \frac{\sum K_c}{K_s} = \frac{\text{sum of flexural stiffness of column acting at the joint}}{\text{flexural stiffness of the slab}}
\]

\[
\text{Longer span}
\]

\[
K_c = \left(\frac{4EI}{L}\right)_{\text{BOTTOM}} + \left(\frac{4EI}{L}\right)_{\text{TOP}} = 2 \times \left(\frac{4EI}{L}\right) = 2 \times \left(\frac{4E}{L}\right) \times \left(\frac{50^4}{12}\right) = \frac{2 \times 4E \times (520 \times 10^3)}{327.5}
\]

\[
K_s = \frac{4E \times (660 \times 28.5^3)}{12 \times 560}
\]

\[
\therefore \alpha_c = \frac{\sum K_c}{K_s} = \frac{2 \times 4E \times 1587.73}{4E \times 2273.5} = 1.39
\]

From table 17 of IS: 456-2000

\[
\frac{L_2}{L_1} = 0.848 \quad \& \quad \frac{W}{W_D} = 1.00
\]

\[
\therefore \alpha_{c_{, \text{min}}} = 0.7
\]

\[
\alpha_c > \alpha_{c_{, \text{min}}}
\]

Hence correction for pattern of loading in the direction of longer span is not required.
Shorter span

\[ K_s = \frac{2 \times (50)^4}{12 \times 262} = 3975.8 \]

\[ K_s = \frac{560 \times 28.5^3}{12 \times 760} = 1421.4 \]

\[ \alpha_c = \frac{3975.8}{1421.4} = 2.79 \]

From table 17 of IS: 456-2000 for

\[ \frac{L_2}{L_1} = 1.17 \quad \& \quad \frac{W_L}{W_D} = 1.00 \]

\[ \alpha_{c,\text{min}} \approx 0.75 \]

\[ \alpha_c > \alpha_{c,\text{min}} : \text{ok} \]

Hence the correction for pattern loading in the direction of short span is not required.

---

<table>
<thead>
<tr>
<th>Imposed load/dead load</th>
<th>Ratio ( \frac{l_2}{l_1} )</th>
<th>Value of ( \alpha_{c,\text{min}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5 to 2.0</td>
<td>(1)</td>
<td>(2)</td>
</tr>
<tr>
<td>0.5</td>
<td>0.5</td>
<td>0.6</td>
</tr>
<tr>
<td>0.8</td>
<td>1.0</td>
<td>0.7</td>
</tr>
<tr>
<td>1.0</td>
<td>0.8</td>
<td>0.7</td>
</tr>
<tr>
<td>2.0</td>
<td>1.0</td>
<td>0.8</td>
</tr>
<tr>
<td>2.0</td>
<td>1.25</td>
<td>1.2</td>
</tr>
<tr>
<td>1.0</td>
<td>2.0</td>
<td>1.3</td>
</tr>
<tr>
<td>1.0</td>
<td>2.0</td>
<td>1.6</td>
</tr>
<tr>
<td>2.0</td>
<td>0.5</td>
<td>1.3</td>
</tr>
<tr>
<td>2.0</td>
<td>0.8</td>
<td>1.5</td>
</tr>
<tr>
<td>2.0</td>
<td>1.0</td>
<td>1.6</td>
</tr>
<tr>
<td>2.0</td>
<td>1.25</td>
<td>1.9</td>
</tr>
<tr>
<td>2.0</td>
<td>2.0</td>
<td>4.9</td>
</tr>
<tr>
<td>0.5</td>
<td>0.8</td>
<td>1.8</td>
</tr>
<tr>
<td>0.5</td>
<td>2.3</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>1.25</td>
<td>2.8</td>
</tr>
<tr>
<td>2.0</td>
<td>2.0</td>
<td>13.0</td>
</tr>
</tbody>
</table>
Distribution of bending moment across the panel width

It is an exterior panel.

**Longer span**

**column strip**

-ve B.M at exterior support = \[ \frac{-0.65 M_o}{1 + \frac{1}{\alpha_c}} \times 1.0 = \frac{-0.65 \times 320.99}{1 + \frac{1}{1.39}} \times 1.0 = -121.34 \text{ KN m} \]

+ve span BM = \[ \frac{0.63 - 0.28}{1 + \frac{1}{\alpha_c}} \times M_o \times 0.60 = \frac{0.63 - 0.28}{1 + \frac{1}{1.39}} \times 320.99 \times 0.60 = 90 \text{ KNm} \]

-ve span BM at interior support = \[ - \left( \frac{0.75 - 0.10}{1 + \frac{1}{\alpha_c}} \right) \times M_o \times 0.75 = - \left( \frac{0.75 - 0.10}{1 + \frac{1}{1.39}} \right) \times 320.99 \times 0.75 = -166.50 \text{ KN m} \]

**Middle strip**

-ve BM at exterior support = \[ \frac{-0.65 M_o}{1 + \frac{1}{\alpha_c}} \times 0.0 = 0.0 \text{ KN m} \]

+ve span BM = \[ \frac{0.63 - 0.28}{1 + \frac{1}{\alpha_c}} \times M_o \times 0.40 = \frac{0.63 - 0.28}{1 + \frac{1}{1.39}} \times 320.99 \times 0.40 = 59.96 \text{ KNm} \]

-ve BM at interior support = \[ - \left( \frac{0.75 - 0.10}{1 + \frac{1}{\alpha_c}} \right) \times M_o \times 0.75 = - \left( \frac{0.75 - 0.10}{1 + \frac{1}{1.39}} \right) \times 320.99 \times 0.25 = -55.50 \text{ KN m} \]
Short span

column strip

-ve moment at exterior support = \[
\left(\frac{-0.65 M_o}{1 + \frac{1}{\alpha_c}}\right) \times 1.0 = \left(\frac{-0.65 \times 251.2}{1 + \frac{1}{2.79}}\right) \times 1.0 = -120.19 \text{ KNm}
\]

+ve moment \[0.63 - \frac{0.28}{1 + \frac{1}{2.79}}\] \times 251.2 \times 0.60 = 63.88 \text{ KNm}

-ve moment at exterior support = \[
- \left[0.75 - \frac{0.10}{1 + \frac{1}{\alpha_c}}\right] \times M_o \times 0.75 = - \left[0.75 - \frac{0.10}{1 + \frac{1}{2.79}}\right] \times 251.2 \times 0.75 = -127.43 \text{ KNm}
\]

Middle strip

-ve moment at exterior support = \[
\left(\frac{-0.65 M_o}{1 + \frac{1}{\alpha_c}}\right) \times 0.0 = 0.0 \text{ KNm}
\]

+ve mid-span moment = \[
0.63 - \frac{0.28}{1 + \frac{1}{2.79}}\] \times M_o \times 0.40 = \left[0.63 - \frac{0.28}{1 + \frac{1}{2.79}}\right] \times 251.2 \times 0.40 = 42.59 \text{ Knm}

-ve moment at interior span = \[
- \left[0.75 - \frac{0.10}{1 + \frac{1}{\alpha_c}}\right] \times M_o \times 0.75 = - \left[0.75 - \frac{0.10}{1 + \frac{1}{2.79}}\right] \times 251.2 \times 0.25 = -42.44 \text{ KNm}
\]

j) Effective depth of the slab

Thickness of the slab, from consideration of maximum positive moment any where in the slab.

Maximum +ve BM occurs in the column strip (long span) = 90.91 KNm

\[\therefore\] factored moment = 1.50 \times 90.91 = 136.36 KNm
\[ M_o = 0.138 f_{ck} b d^2 (b = 2800 \text{ mm}) \]
\[ d = \frac{136.36 \times 10^6}{\sqrt{0.138 \times 20 \times 2800}} \quad (\text{M-20 grade concrete}) \]
\[ d = 132.83 \text{ mm} \approx 140 \text{ mm} \]

Using 12 mm \( \varphi \) (diameter) main bars.

Overall thickness of slab = 140 + 15 + \( \frac{12}{2} \) = 161 mm \( \cup \) 170 mm

\[ \therefore \text{Depth (along longitudinal direction)} = 170 - 15 - \frac{12}{2} = 150 \text{ mm} \]

\[ \therefore \text{Depth (along longitudinal direction)} = 150 - 12 = 138 \text{ mm} \]

k) Thickness of drop from maximum –ve moment consideration

Thickness of drop from consideration of maximum –ve moment any where in the panel.

Max –ve BM occurs in the column strip = 166.6 KNm
\[ M_u = 0.138 f_{ck} b d^2 \]
\[ 1.5 \times 166.6 \times 10^6 = 0.138 \times 20 \times 1400 \times d^2 \]
\[ d = 254.3 \text{ mm} \]
Say 260 mm. Use 12 mm \( \varphi \) bars

Over all thickness of flat slab: \( D = 260 + 15 + \frac{12}{2} = 281 \text{ mm} \)
1) Shear in Flat Slab

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 (Fig. 2). The shape in plan is geometrically similar to the support immediately below the slab.

check for shear stress developed in slab

The critical section for shear for the slab will be at a distance \(d/2\) from the face of drop.

Perimeter of critical section = \(4 \times 2340 = 9340\) mm

\[
V_0 = 1.5 \times 15.45 \times [L_1 \times L_2 - (2.34)(2.34)]
\]

Total factored shear force: 
\[
= 1.5 \times 15.45 \times [6.6 \times 5.6 - (5.47)] 
= 729.78\) KN

Nominal shear stress = \(\tau_v = \frac{V}{bd} = \frac{729.78 \times 10^3}{9340 \times 140} = 0.55\) N/mm\(^2\)

Shear strength of concrete = \(\tau_c = 0.25\sqrt{f_{ck}} = 0.25\sqrt{20} = 1.11\) N/mm\(^2\)

Permissible shear stress = \(\tau_v \neq k_s\tau_c\)

\(k_s = (0.5 + \beta_c), \beta_c = 0.848\)

\(k_s = (0.5 + 0.848)\)

\(k_s = 1.348 < 1 \Rightarrow 1\)

\(= 1 \times 1.11\)

\(= 1.11\) N/mm\(^2\)

\(\tau_v < \tau_c \Rightarrow\) safe design ok

if \(\tau_v > 1.5\tau_c\) then the slab should be re-designed

m) check for shear in drop

\(b_0 = \pi (D + d_0) = \pi (1.3 + 0.26) = 4.89\) m

\(V = 1.5 \times 15.45 [5.6 \times 6.6 - \frac{\pi}{4}(1.3 + 0.26)^2]\)

\(V = 812.27\) KN

\(\tau_v = \frac{812.27 \times 10^3}{4890 \times 260} = 0.683\) N/mm\(^2\)

Nominal shear stress: \(\tau_c = 0.25\sqrt{f_{ck}} = 1.11\) N/mm\(^2\)

\(\tau_v < \tau_c\) [safe in shear]
n) Reinforcement details

**Longer span**

-ve exterior reinforcement:

\[ M_u = 0.87 f_y A_{st} [d - 0.42 x_u] \]

\[ 1.5 \times 121.34 \times 10^6 = 0.87 \times 415 \times A_{st}[150 - 0.42 \times 0.48 \times 150] \]

\[ A_{st} = 4209 \text{ mm}^2 \]

Use 12 mm \( \phi \) bars = \[ \frac{4209}{113} = 38 \text{ No.s} \]

\[ \therefore \text{c/c spacing is} = \frac{1.4 \times 1000}{38} = 36 \text{ mm c/c} \]

+ve steel:

\[ 1.5 \times 90 \times 10^6 = 43239.3 \times A_{st} \]

\[ A_{st} = 3122 \text{ mm}^2 \]

Use 12 mm \( \phi \) bars = \[ \frac{3122}{113} = 28 \text{ No.s} \]

\[ \therefore \text{c/c spacing} = \frac{3.8 \times 1000}{28} = 135 \text{ mm c/c} \]

Reinforcement along shorter span:

**Column strip:**

\[ M_u = 0.87 f_y A_{st} [d - 0.42 x_u] \]

\[ 1.5 \times 127.5 \times 10^6 = 0.87 \times 415 \times A_{st}[140 - 0.42 \times 0.48 \times 140] \]

\[ A_{st} = 3768.9 \text{ mm}^2 \]

Use 12 mm \( \phi \) bars = \[ \frac{3768.9}{\pi (12)^2} = 33 \text{ No.s} \]

\[ \therefore \text{c/c spacing is} = \frac{1.4 \times 1000}{33} = 42 \text{ mm c/c} \]

**Middle strip:**

\[ M_u = 0.87 f_y A_{st} [d - 0.42 x_u] \]

\[ 1.5 \times 63.88 \times 10^6 = 0.87 \times 415 \times A_{st}[281 - 0.42 \times 0.48 \times 281] \]

\[ A_{st} = 1182 \text{ mm}^2 \]

Use 12 mm \( \phi \) bars = \[ \frac{1182}{\pi (12)^2} = 10 \text{ No.s} \]

\[ \therefore \text{c/c spacing is} = \frac{2.8 \times 1000}{10} = 280 \text{ mm c/c} \]
Design of flat slabs as per NZS: 3101

DEFINITIONS:

- A flat slab is reinforced concrete slab directly supporting on column (without any support of beams).
- Flat slabs is divided into column strips & middle strips.
  - Column strips is a design strip with a width on each side of a column centre line equal to 0.25L1 or 0.25L2, whichever is less.
  - A middle strip is a design strip bounded by 2 column strips.
  - A panel is bounded by column, beams, or wall centre lines on all sides.

DESIGN METHOD:

- There must a minimum 3 continuous spans in each directions.
- Panels shall be rectangular with a ratio of longer to shorter spans, centre to centre of supports, not greater than 2.
- Successive span lengths, centre-to-centre of supports, in each direction shall not differ by more than 1/3 of the longer spans.
- Columns may be offsets a maximum of 10% of the span (in direction of offset) from either axis between centre lines of successive columns.
- All loads shall be due to gravity only and uniformly distributed over entire panels. The live loads shall not exceeds 2 times the dead load.

DESIGN PROCEDURE:

- First analysis the column strips & middle strips using 0.25L1/0.25L2.
- Drop panel is used to reduce the amount of negative moment reinforcement over the column of the flat slab, the size of drop panel shall be 1/6 of the span length measured from centre-to-centre of support in that direction.
• Estimate the depth of flat slabs from clauses 14.2.5 & 3.3.2.2.(b)
  Assume fy=300MPA.

<table>
<thead>
<tr>
<th>Fy (MPA)</th>
<th>Exteriors panels</th>
<th>Interior panels</th>
</tr>
</thead>
<tbody>
<tr>
<td>300</td>
<td>Ln/36</td>
<td>Ln/40</td>
</tr>
<tr>
<td>400</td>
<td>Ln/32</td>
<td>Ln/35</td>
</tr>
</tbody>
</table>

• The absolute sum for the span shall be determined in a strip bounded laterally by
  the center line of the panel on each side of centre of the supports.

• The absolute sum of positive and average negative moments in each direction at
  the ultimate limit state shall be not less than:

  \[ Mo = WuL^2Ln^3/8; \]

  **Negative & positive design moments:**

  In an interior spans

  ➢ Negative moments—0.65
  ➢ Positive moments---0.35

  In end spans

<table>
<thead>
<tr>
<th></th>
<th>Exterior edge unrestrained</th>
<th>Slab with beams between all supports</th>
<th>Slabs without beams between interior supports</th>
<th>Exterior edge fully restrained</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Without edge beams</td>
<td>With edge beams</td>
<td></td>
</tr>
<tr>
<td>Interior – ve moments</td>
<td>0.75</td>
<td>0.70</td>
<td>0.70</td>
<td>0.65</td>
</tr>
<tr>
<td>Positive moments</td>
<td>0.63</td>
<td>0.57</td>
<td>0.52</td>
<td>0.50</td>
</tr>
<tr>
<td>Exterior – ve moments</td>
<td>0</td>
<td>0.16</td>
<td>0.26</td>
<td>0.30</td>
</tr>
</tbody>
</table>

22
SHEAR STRENGTH

- Design of cross section of member subjected to shear shall be based on

$$v' \leq \varphi V_n.$$ 

Where $$v'$$=shear force at that section .

$$V_n$$=nominal shear strength of the section.

$$\varphi$$ =strength reduction factor.

- The nominal shear stress $$V_n$$ shall not exceed 0.2$$f_c$$, 1.1$$\sqrt{f_c}$$ or 9MPA.

- Spacing limits for shear reinforcements shall be:
  - 0.5d in non-prestressed member
  - 0.75 h in prestressed member
  - 600mm.

- Design of slab for two way action shall be based on

$$V_n = V_n/b_{od}$$

Where $$v_n$$ shall not be greater than $$V_c$$

$$V_c = 0.17(1+2\beta_c)\sqrt{f_c}$$

$$\beta_c$$=shorter side/long side of the concentrated load

- Design the interior panel of flat slabs 6.6 x 5.6 m in size for a super imposed load of 7.75 KN/m², provide two way reinforcement.
Design steps:

<table>
<thead>
<tr>
<th>LONGER SPAN</th>
<th>SHORTER SPAN</th>
</tr>
</thead>
<tbody>
<tr>
<td>L1=6.6m, L2=5.6m</td>
<td>L2=6.6m, L1=5.6m</td>
</tr>
<tr>
<td>• Column strip</td>
<td>• Column strip</td>
</tr>
<tr>
<td>0.25L2=1.4 m</td>
<td>0.25L1=1.4 m</td>
</tr>
<tr>
<td>≤0.25L1=1.65m</td>
<td>Adopt 1.4m</td>
</tr>
<tr>
<td>• Middle strip</td>
<td>• Middle strip</td>
</tr>
<tr>
<td>5.6-(1.4+1.4)=2.8m</td>
<td>6.6-(1.4+1.4)=3.8m</td>
</tr>
</tbody>
</table>

Drop dimensions:

<table>
<thead>
<tr>
<th>Longer span</th>
<th>Shorter span</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shall not be less than L/3=6.6/3</td>
<td>Shall not be less than L/3=5.6/3</td>
</tr>
<tr>
<td>2.2M</td>
<td>1.86M</td>
</tr>
</tbody>
</table>

Hence provide a drop size of 2.2x2.2m

Estimate the depth of flat slabs:

From clauses 14.2.5 & 3.3.2.2(b)

<table>
<thead>
<tr>
<th>Fy(MPA)</th>
<th>Exteriors panels</th>
<th>Interior panels</th>
</tr>
</thead>
<tbody>
<tr>
<td>300</td>
<td>Ln/36</td>
<td>Ln/40</td>
</tr>
<tr>
<td>400</td>
<td>Ln/32</td>
<td>Ln/35</td>
</tr>
</tbody>
</table>
Let's adopt $f_y = 300$ MPa

d = $\frac{6600}{36} = 183.3$ mm for exterior

d = $\frac{6600}{40} = 165$ mm

taking effective depth 25 mm

overall depth $D = 185 + 25 = 210$ mm

Load calculations:

nominal density of concrete ($\rho = 2400$ kg/m$^3$): clauses 3.3.2.3

(Wd) dead load on slab $0.210 \times 24 = 5.04$ kN/m$^2$

(WL)live load on slab $= 7.75$ kN/m$^2$

\[ 12.79 \text{kN/m}^2 \]

Check \( W_l/W_d < 2 \)

\[ \frac{7.75}{5.04} = 1.53 < 2 \quad \text{O.K} \]

Total static moments for the spans:

\[ M_0 = W_u l^2 Ln^2/8 \]

Longer span

\[ M_0 = 389.99 \text{KN-M} \]

Shorter span

\[ M_0 = 330 \text{KN-M} \]
Distribution of bending moments across the panel width:

Interior span
-ve moment = 0.65
+ve moment = 0.35

Column strip
-ve B.M at exterior span = 0.75xMo = 271.4 KN-M
+ve B.M at interior span = 0.63xMo = 245.64 KN-M
-ve B.M at interior span = 0.65xMo = 253.4 KN-M

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.65</td>
<td>+0.63</td>
</tr>
<tr>
<td></td>
<td>245.3kN-m</td>
</tr>
</tbody>
</table>

Middle strip
-ve B.M at exterior support = 0 KN-M
+ve span BM = 0.63*Mo = 245.64 KN-M
-ve span BM at interior support
= 0.75xMo = 292.40 KN-M

Column strip
-ve B.M at exterior support = 0.70xMo KN-M
= 231 KN-M
+ve span BM interior support = 0.52*Mo = 171.6 KN-M
-ve span BM at exterior support
= 0.26xMo = 85.8 KN-M
Middle strip

-ve B.M at exterior support = 0.65xMo KN-M
  = 214.5 KN-M
+ve span BM mid span = 0.35*Mo = 115 KN-M

-ve span BM at interior support
  = 0.70xMo = 231 KN-M

Moments in column strips:

Interior negative moments

<table>
<thead>
<tr>
<th>L2/L1</th>
<th>0.5</th>
<th>1.0</th>
<th>2.0</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(α L2/L1) = 0</td>
<td>75</td>
<td>75</td>
<td>75</td>
</tr>
<tr>
<td>(α L2/L1) &gt; 0</td>
<td>90</td>
<td>75</td>
<td>45</td>
</tr>
</tbody>
</table>

Positive moments

<table>
<thead>
<tr>
<th>L2/L1</th>
<th>0.5</th>
<th>1.0</th>
<th>2.0</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(α L2/L1) = 0</td>
<td>75</td>
<td>75</td>
<td>75</td>
</tr>
<tr>
<td>(α L2/L1) &gt; 0</td>
<td>90</td>
<td>75</td>
<td>45</td>
</tr>
</tbody>
</table>

Longer span:

Column strip:
- ve BM at exterior span = 292.14 KN-M
  + ve BM at mid span  = 147.37 KN-M
- ve BM at interior span = 189.8 KN-M

Middle strip:

- ve BM at exterior span = 0 KN-M
  + ve BM at mid span  = 147.37 KN-M
- ve BM at interior span = 219 KN-M
Shorter span:-

Column strip:-
-ve BM at exterior span=231.14KN-M
+ve BM at mid span =102.97KN-M
-ve BM at interior span =64KN-M

Middle strip:-
-ve BM at exterior span=214.5 KN-M
+ve BM at mid span =69.3KN-M
-ve BM at interior span =173.25KN-M

Check for shear develop in slab

\[ \nu' \leq \varepsilon V_n. \]

Design of slab for two way action under clauses 9.3.15.2

\[ V^* = \frac{V_n}{b_o * d} \]

\[ V_n = \text{nominal shear stress} \]

\[ V_n = 1.5 * 12.79 * [5.6 * 6.6 - (2.30)(2.30)] \]

\[ V_n = 607.\text{KN} \]

\[ V_n^* = 607.5 * 10^3 / 9200 * 165 \]

\[ V_n^* = 0.399 \text{ N/mm}^2 \]

\[ V_c = 0.17(1 + \alpha d / (2 * b_o)) \sqrt{f_c} \]

\[ V_c = 0.17(1 + 2 \beta c) \sqrt{f_c} \]

\[ B_c = \text{shorter side/long side} \]

\[ V_c = 2.51 \text{ N/mm}^2 \]

\[ V_n \text{ is not greater than } V_c \text{ (safe)} \]

Reinforcement:-

Longer span
-ve exterior reinforcement

\[ \text{Mu} = \text{As} \times f_y (d - 0.59 \times (\text{Ast} \times F_y / F_c \times b)) \]

Reinforcement ratio \( \rho = \sqrt{F_c / (4 \times F_y)} \)

\[ P = 0.0045 \]

\[ P = \text{As} / b \times d \]

\[ \text{As} = 9477.6 \text{ mm}^2 \]

Use 12 mm dia bars =83 nos

c/c spacing 17 mm

+ve steel

\[ \text{As} = 3946 \text{ mm}^2 \]

Use 12 mm dia bars 34 nos

c/c spacing 111 mm

Shorter span

Column strip

\[ \text{Mu} = \text{As} \times f_y (d - 0.59 \times (\text{Ast} \times F_y / F_c \times b)) \]

\[ \text{As} = 6798 \text{ mm}^2 \]

Use 12 mm 60 nos

23 mm c/c spacing

Middle strip

\[ \text{As} = 2648 \text{ mm}^2 \]

c/c spacing 121 mm
EUROCODE

Introduction

This Eurocode gives all structural design irrespective of the material of construction. It establishes principles and requirements for safety, serviceability and durability of structures. The Eurocode uses a statistical approach to determine realistic values for actions that occur in combination with each other. Partial factors for actions are given in this Eurocode, whilst partial factors for materials are prescribed in their relevant Eurocode. It is again divided into different codes based on the materials. In this Eurocode, 2 gives the design of concrete structures.

EUROCODE 2

1. Eurocode 2 is generally laid out to give advice on the basis of phenomena (e.g. bending, shear etc) rather than by member types as in BS 8110 (e.g. beams, slabs, columns etc).

2. Design is based on characteristic cylinder strengths not cube strengths.

3. The Eurocode does not provide derived formulae (e.g. for bending, only the details of the stress block are expressed). This is the traditional European approach, where the application of a Eurocode expected to be provided in a textbook or similar publication.

4. Units for stress are mega pascals, MPa (1 MPa = 1 N/m²).

5. Higher strengths of concrete are covered by Eurocode 2, up to class C90/105. However, because the characteristics of higher strength concrete are different, some Expressions in the Eurocode are adjusted for classes above C50/60.

6. The partial factor for steel reinforcement is 1.15. However, the characteristic yield strength of steel that meets the requirements of BS 4449 will be 500 MPa; so overall the effect is negligible. Eurocode 2 is applicable for ribbed reinforcement with characteristic yield strengths of 400 to 600 MPa. There is no guidance on plain bar or mild steel reinforcement in the Eurocode, but guidance is given in the background paper to the UK National Annex 10.

7. Minimum concrete cover is related to bond strength, durability and fire resistance. In addition to the minimum cover an allowance for deviations due to variations in execution (Construction) should be included. Eurocode 2 recommends that, for concrete cast against formwork, this is taken as 10 mm, unless the construction is subject to a quality assurance systemic which case it could be reduced to 5 mm or even 0 mm whereon -conforming members are rejected (e.g. in a precast yard).

8. The punching shear checks are carried at 2d from the face of the column and for a rectangular column, the perimeter is rounded at the corners.
Design of flat slabs as per EUROCODE 2

A procedure for carrying out the detailed design of flat slabs is given below.

1. Determine design life
2. Assess actions on the slab
3. Determine which combinations of actions apply
4. Determine loading arrangements
5. Assess durability requirements and determine concrete strength
6. Check cover requirements for appropriate fire resistance period
7. Calculate min. cover for durability, fire and bond requirements
8. Analyse structure to obtain critical moments and shear forces
9. Design flexural reinforcement
10. Check for deflection
11. Check punching shear capacity
12. Check spacing of bars

Determine design life

Based on structural design and their usage the values are given in table

<table>
<thead>
<tr>
<th>Design life(years)</th>
<th>Examples</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>Temporary structures</td>
</tr>
<tr>
<td>10-30</td>
<td>Replaceable structural parts</td>
</tr>
<tr>
<td>15-25</td>
<td>Agricultural and similar structures</td>
</tr>
<tr>
<td>50</td>
<td>Buildings and other common structures</td>
</tr>
<tr>
<td>120</td>
<td>Monumental buildings, bridges and other civil engineering structures</td>
</tr>
</tbody>
</table>

Assess actions on the slab

The load arrangements for flat slabs met the following requirements

1. The ratio of the variable actions (Qk) to the permanent actions (Gk) does not exceed 1.25.
2. The magnitude of the variable actions excluding partitions does not exceed 5 kN/m².
Procedure for determining flexural reinforcement

Carry out analysis of slab to determine design moments ($M$)
(Where appropriate use coefficients from the below Table).

<table>
<thead>
<tr>
<th>End support/slab connection</th>
<th>First interior support</th>
<th>Interior spans</th>
<th>Interior supports</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pinned</td>
<td>continuous</td>
<td></td>
<td></td>
</tr>
<tr>
<td>End support</td>
<td>End span</td>
<td>End Support</td>
<td>End span</td>
</tr>
<tr>
<td>Moment</td>
<td>0</td>
<td>0.086Fl</td>
<td>-0.04Fl</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.075Fl</td>
<td>-0.086Fl</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.063Fl</td>
<td>-0.063Fl</td>
</tr>
</tbody>
</table>

Where $F$ is the total design ultimate load, $l$ is the effective span

This analysis is only for concrete class $C50/60$ only.

Determine $K$ from the equation $K = M/bd^2f_{ck}$

Determine $K'$ from the given Table or $K' = 0.60\delta - 0.182\delta^2 - 0.21$ where $\delta \leq 1.0$

<table>
<thead>
<tr>
<th>% redistribution</th>
<th>$\delta$ (redistribution ratio)</th>
<th>$K'$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.00</td>
<td>0.205</td>
</tr>
<tr>
<td>5</td>
<td>0.95</td>
<td>0.193</td>
</tr>
<tr>
<td>10</td>
<td>0.90</td>
<td>0.180</td>
</tr>
<tr>
<td>15</td>
<td>0.85</td>
<td>0.166</td>
</tr>
<tr>
<td>20</td>
<td>0.80</td>
<td>0.151</td>
</tr>
<tr>
<td>25</td>
<td>0.75</td>
<td>0.136</td>
</tr>
</tbody>
</table>

If $K < K'$, Provide compression reinforcement. Otherwise No compression reinforcement

Obtain lever arm $z$ from the equation $z = d/2[1-3.53K] \leq 0.95d$

Calculate tension reinforcement required from $A_s = M/fydz$;

Check minimum reinforcement requirements $A_{s, \min} = 0.26\sqrt{f_{ctm}\text{bt}} d/fyk$

where $fyk \geq 25$

Check maximum reinforcement requirements. $A_{s, \max} = 0.04A_c$ for tension or compression reinforcement outside lap locations.
Check for deflection

Eurocode 2 has two alternative methods of designing for deflection; either by limiting span-to-depth ratio or by assessing the theoretical deflection using the Expressions given in the Eurocode. In this we have to find using span to depth ratio.

Procedure for finding deflection

1. Determine basic l/d from below fig

2. Determine Factor 1 (F1)
   For ribbed or waffle slabs
   \[ F_1 = 1 - 0.1 \left( \frac{b_f}{b_w} - 1 \right) \geq 0.8 \]
   Where \( b_f \) = flange breadth and \( b_w \) = rib breadth
   Otherwise \( F_1 = 1.0 \)

3. Determine Factor 2 (F2)
   Where the slab span exceeds 7 m and it supports brittle partitions, \( F_2 = \frac{7}{l_{eff}} \)
   Otherwise \( F_2 = 1.0 \)

4. Determine Factor 3 (F3)
   \( F_3 = \frac{310}{s_s} \)
   Where \( s_s \) = Stress in reinforcement at serviceability limit state or \( s_s \) may be assumed to be 310 MPa (i.e. \( F_3 = 1.0 \))

Check \( A_{s,prov} \leq 1.5 \times A_{s,req'd} \)

Is basic \( l/d \times F_1 \times F_2 \times F_3 \geq \)Actual \( l/d \) if this condition is satisfied it is safe from deflection otherwise we have to increase \( A_{s,prov} \).
Punching shear

The design value of the punching shear force, $V_{Ed}$, will usually be the support reaction at the ultimate limit state.

1. The maximum value of shear at the column face is not limited to 5 MPa, and depends on the concrete strength used.
2. The control perimeters for rectangular columns in this have rounded corners.
3. Where shear reinforcement is required the procedure is simpler; the point at which no shear reinforcement is required can be calculated directly and then used to determine the extent of the area over which shear reinforcement is required.
4. It is assumed that the reinforcement will be in a radial arrangement. However, the reinforcement can be laid on a grid provided the spacing rules are followed.

Procedure for determining the punching shear

1. Determine value of factor $\beta$ from the below fig

![Fig showing corner, edge, and internal columns with $\beta$ values]

2. Determine value of $v_{Ed,max}$ design shear stress at face of column from

$$v_{Ed,max} = \beta \frac{V_{Ed}}{u_i d_{eff}}$$

where $u_i$ is perimeter of column
deff $= (d_y + d_z)/2$ (dy and dz are the effective depths in orthogonal directions)

Determine value of $v_{Rd,max}$ from Table 1

Check $v_{Ed,max} \leq v_{Rd,max}$ if not redesign the slab.

Determine value of $v_{Ed}$, (design shear stress)

$$v_{Ed,max} = \beta \frac{V_{Ed}}{u_1 d_{eff}}$$

where $u_1$ is length of control perimeter

Determine concrete punching shear capacity (without shear reinforcement), $v_{RD,c}$ from where $r_l = (r_{ly} r_{lz})^{0.5}$

(rly, rlz are the reinforcement ratios in two orthogonal directions for fully bonded tension steel, taken over a width equal to column width plus 3 $d$ each side.)

Is $v_{Ed} > v_{Rd,c}$ if it satisfies Punching shear reinforcement not required otherwise
Determine area of punching shear reinforcement per perimeter from:
\[ A_{sw} = (v E_d - 0.75 v R_d, c) s r u_1 / (1.5 f_{ywd, ef}) \]
where \( s_r \) is the radial spacing of shear reinforcement

\[ f_{ywd, ef} = 250 + 0.25 d_{eff} \leq f_{ywd} \]

Determine the length of the outer perimeter where shear reinforcement not required from:
\[ u_{out, ef} = b V E_d / (v R_d, c, d) \]

Check spacing of bars

Min area or reinforcement

1. The minimum area of longitudinal reinforcement in the main direction is \( A_{s, min} = 0.26 f_{ctm} b t d / f_{yk} \) but not less than 0.0013 \( b d \).
2. The minimum area of a link leg for vertical punching shear reinforcement is \( 1.5 A_{sw, min} / (s_r st) \geq 0.08 f_{ck}^{1/2} f_{yk} \).

which can be rearranged as \( A_{sw, min} \geq (s_r st) / F \)

where \( s_r \) = the spacing of the links in the radial direction
\( s_t \) = the spacing of the links in the tangential direction
\( F \) can be obtained from Table 10

Max area of reinforcement

Outside lap locations, the maximum area of tension or compress reinforcement should not exceed \( A_{s, max} = 0.4 A_c \)

Minimum spacing of reinforcement

The minimum spacing of bars should be the greater of:
- Bar diameter
- Aggregate size plus 5 mm
- 20 mm

Max spacing of main reinforcement

For slabs less than 200 mm thick the following maximum spacing rules apply:
1. for the principal reinforcement
   - 3\( h \) but not more than 400 mm
2. for the secondary reinforcement:
   - 3.5\( h \) but not more than 450 mm

The exception is in areas with concentrated loads or areas of maximum moment where the following applies:
1. for the principal reinforcement
   - 2\( h \) but not more than 250 mm
2. for the secondary reinforcement
   - 3\( h \) but not more than 400 mm

Where \( h \) is the depth of the slab.
For slabs 200 mm thick or greater reference should be made to Section 7.3.3 of the Eurocode.

Spacing of punching shear reinforcement

Where punching shear reinforcement is required the following rules should be observed.
1. It should be provided between the face of the column and \( kd \) inside the outer perimeter where shear reinforcement is no longer required. \( k \) is 1.5, unless the perimeter at which reinforcement is no longer required is less than 3 \( d \) from the face of the column. In this case the reinforcement should be placed in the zone 0.3 \( d \) to 1.5 \( d \) from the face of the column.
2. There should be at least two perimeters of shear links.
3. The radial spacing of the links should not exceed 0.75 \( d \)
4. The tangential spacing of the links should not exceed 1.5 \( d \) within 2\( d \) of the column face.
5. The tangential spacing of the links should not exceed 2\( d \) for any other perimeter.
6. The distance between the face of the column and the nearest shear reinforcement should be less than 0.5\( d \)
**Numerical example:**

Longer span = 6.6 m  
Shorter span = 5.6 m  
Live load = 7.75 kN/m²  
Assume grade of concrete as C20/25 i.e $f_{ck} = 20$ MPa  
Where C20/25 the cylinder strength as 25 MPa, whereas C20/25 the cube strength as 20 MPa,

Depth of the slab from deflection criteria = span/21  
(this is based on longer span)  
Effective depth = 314 mm  
This depth also satisfies the fire resistance according to euro code (REI 120).

Total depth = 314+15 = 350 mm (Based on the axis distance from code)  
D = 350 mm

Load calculations

Dead load acting on the slab = 0.35 x 25 = 8.75 KN/m² = $W_d$

Live load on slab = 7.75 KN/m² = $W_l$

The design live load shall not exceed 1.25 times the design dead load.

Check: $wl/wd = 0.0885 < 1.25$ (safe)

Total design load = $W_d + W_l = 15.45$ KN/m²

Values of secant modulus of elasticity for C20/25 = 29 KN/mm²

Moments calculations

For longer span  
Calculate $M = 503.118$ KN-m  
From this calculate $K$, $K = M/bd^2f_{ck}$  
$= 0.0129$

$K' = 0.60\delta - 0.182\delta^2 - 0.21$ where $\delta \leq 1.0$

$= 0.1975 < K$  
(ok) safe  
No compression reinforcement required

Calculation of Z  
$Z = d/2[1-3.53K]$  
= 298  
$\leq 0.95$  
OK (safe)  
Punching shear calculations  
For internal columns take $\beta = 1.15$

$vEd_{max} = \beta VEd/(ui deff)$
where \( u_i \) is perimeter of column = 2000mm

column size is 500x500 mm

\[ v_{Ed,\max} = \frac{(1.15 \times 896)}{(2000 \times 314)} = 1.64 \text{ KN/mm}^2 \]

\[ v_{Rd,\max} = 3.31 \text{ (from code)} \]

\[ v_{Ed,\max} \leq v_{Rd,\max} \]

OK (safe)

\[ v_{Ed,\max} = \beta V_{Ed} / (u_i d_{eff}) \]

\[ v_{Ed} = \frac{1.16 \times 896 \times 10^3}{1200 \times 314} = 2.73 \]

\[ v_{Rd,c} = 0.75 \text{ from code} \]

\[ v_{Ed} > v_{Rd,c} \]

ok

Area of punching shear reinforcement

\[ A_{sw} = (v_{Ed} - 0.75 v_{Rd,c}) s_r u_1 / (1.5 f_y w_d, e) \]

\[ = 2334.4 \text{ mm}^2 \]

Min area or reinforcement

\[ A_s, \text{ min} = 0.26 f_{ctm} b t d / f_y k \]

\[ = 408.2 \text{ mm}^2 \]

\[ < 0.0013 \times 1000 \times 314 \]

\[ = 424 \text{ mm}^2 \]

Ok

Max area of reinforcement

\[ A_s, \text{ max} = 0.4 A_c \]

\[ = 2415.5 \text{ mm}^2 \]

Minimum spacing of reinforcement

The minimum spacing of bars should be the greater of:

Bar diameter = 12 mm
Aggregate size plus 5 mm = 9.75 mm
20 mm
Min spacing = 20 mm

Max spacing of main reinforcement
Use 12 mm φ bars = \( \frac{4209}{113} \) = 38 No.s

\[ \therefore \text{c/c spacing is} = \frac{1.4 \times 1000}{38} = 36 \text{ mm c/c} \]

Max spacing = 36 mm

In this no punching shear reinforcement so no spacing for that.
Design of flat slabs using ACI-318:

Drop of flat slabs:

Where a drop panel is used to reduce amount of negative moment reinforcement over the column of a flat slab, size of drop panel shall be in accordance with the following:

Drop panel shall extend in each direction from centerline of support a distance not less than one-sixth the span length measured from center-to-center of supports in that direction.

Projection of drop panel below the slab shall be at least one-quarter the slab thickness beyond the drop.

In computing required slab reinforcement, thickness of drop panel below the slab shall not be assumed greater than one-quarter the distance from edge of drop panel to edge of column or column capital.

Thickness of the slab:

For slabs without interior beams spanning between the supports and having a ratio of long to short span not greater than 2, the minimum thickness shall be in accordance with the provisions of Table below and shall not be less than the following values:

(a) Slabs without drop panels as ......................... 5 in.
(b) Slabs with drop panels as defined ................. 4 in.

<table>
<thead>
<tr>
<th>MINIMUM THICKNESS OF SLABS WITHOUT INTERIOR BEAMS</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Yield strength, $f_y$, psi</th>
<th>Without drop panels$^\dagger$</th>
<th>With drop panels$^\dagger$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Exterior panels</td>
<td>Interior panels</td>
</tr>
<tr>
<td></td>
<td>Without edge beams</td>
<td>With edge beams$^\ddagger$</td>
</tr>
<tr>
<td>40,000</td>
<td>33</td>
<td>36</td>
</tr>
<tr>
<td>60,000</td>
<td>30</td>
<td>33</td>
</tr>
<tr>
<td>75,000</td>
<td>28</td>
<td>31</td>
</tr>
</tbody>
</table>
Design strips

Column strip is a design strip with a width on each side of a column centerline equal to 0.25 \( l_2 \) or 0.25 \( l_1 \), whichever is less.

Middle strip is a design strip bounded by two column strips.

A panel is bounded by column, beam, or wall centerlines on all sides.

Column head

The upper supporting part of a column is enlarged to form the column head. The diameter or the column head is made 0.20 to 0.25 of the span length.

Total factored static moment for a span

Total factored static moment for a span shall be determined in a strip bounded laterally by centerline of panel on each side of centerline of supports.

Absolute sum of positive and average negative factored moments in each direction shall not be less than.

\[
M_0 = \frac{w_u l_2 l_3}{8}
\]

\( w_u \) = load per unit area acting on the slab panel

\( l_u \) = Clear span \( l_u \) shall extend from face to face of columns, capitals, brackets, or walls.

Value of \( l_u \) shall not be less than 0.65 \( l_1 \). Circular or regular polygon shaped supports shall be treated as square supports with the same area.

\( l_2 \) = When the span adjacent and parallel to an edge is being considered, the distance from edge to panel centerline shall be substituted for \( l_2 \).

In an interior span, total static moment \( M_0 \) shall be distributed as follows:

Negative factored moment .................................0.65

Positive factored moment .................................0.35
In an end span, total factored static moment $M_o$ shall be distributed as follows:

<table>
<thead>
<tr>
<th></th>
<th>(1)</th>
<th>(2)</th>
<th>(3)</th>
<th>(4)</th>
<th>(5)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Exterior</td>
<td>Slab with</td>
<td>Slab without</td>
<td>Exterior</td>
<td></td>
</tr>
<tr>
<td></td>
<td>edge unre-</td>
<td>beams</td>
<td>beams</td>
<td>edge fully</td>
<td></td>
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<td></td>
<td>strained</td>
<td>between all</td>
<td>between</td>
<td>restrained</td>
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<td>supports</td>
<td>interior</td>
<td></td>
<td></td>
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<tr>
<td>Interior</td>
<td>0.75</td>
<td>0.70</td>
<td>0.70</td>
<td>0.65</td>
<td></td>
</tr>
<tr>
<td>negative</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>factored</td>
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<tr>
<td>moment</td>
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<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Positive</td>
<td>0.63</td>
<td>0.57</td>
<td>0.52</td>
<td>0.35</td>
<td></td>
</tr>
<tr>
<td>factored</td>
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<tr>
<td>moment</td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Exterior</td>
<td>0</td>
<td>0.16</td>
<td>0.26</td>
<td>0.30</td>
<td>0.65</td>
</tr>
<tr>
<td>negative</td>
<td></td>
<td></td>
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</tbody>
</table>

Negative moment sections shall be designed to resist the larger of the two interior negative factored moments determined for spans framing into a common support unless an analysis is made to distribute the unbalanced moment in accordance with stiffness of adjoining elements.

Edge beams or edges of slab shall be proportioned to resist in torsion their share of exterior negative factored moments.

**Factored moments in middle strips:**

That portion of negative and positive factored moments not resisted by column strips shall be proportionately assigned to corresponding half middle strips.

Each middle strip shall be proportioned to resist the sum of the moments assigned to its two half middle strips.

A middle strip adjacent to and parallel with an edge supported by a wall shall be proportioned to resist twice the moment assigned to the half middle strip corresponding to the first row of interior supports.
Factored moments in column strips:

Column strips shall be proportioned to resist the following portions in percent of exterior negative factored moments:

<table>
<thead>
<tr>
<th>$\frac{h}{h'}$</th>
<th>0.5</th>
<th>1.0</th>
<th>2.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>$(\alpha_{1/2}/\alpha_{1/4}) = 0$</td>
<td>75</td>
<td>75</td>
<td>75</td>
</tr>
<tr>
<td>$(\alpha_{1/2}/\alpha_{1/4}) \geq 1.0$</td>
<td>90</td>
<td>75</td>
<td>45</td>
</tr>
</tbody>
</table>

Column strips shall be proportioned to resist the following portions in percent of exterior negative factored moments:

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<th>2.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>$(\alpha_{1/2}/\alpha_{1/4}) = 0$</td>
<td>$\beta_t = 0$</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>$\beta_t \geq 2.5$</td>
<td>75</td>
<td>75</td>
<td>75</td>
</tr>
<tr>
<td>$(\alpha_{1/2}/\alpha_{1/4}) \geq 1.0$</td>
<td>$\beta_t = 0$</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>$\beta_t \geq 2.5$</td>
<td>90</td>
<td>75</td>
<td>45</td>
</tr>
</tbody>
</table>

Modification of factored moment:

Modification of negative and positive factored moments by 10 percent shall be permitted provided the total static moment for a panel in the direction considered is not less than that required by

$$M_0 = \frac{w_i b_i^2 l_n^2}{8}$$

Shear provision (punching shear):

Two-way action where each of the critical sections to be investigated shall be located so that its perimeter $b_o$ is a minimum but need not approach closer than $d/2$ to

(a) Edges or corners of columns, concentrated loads, or reaction areas, or
(b) Changes in slab thickness such as edges of capitals or drop panels.

**Nominal shear strength of concrete:**

for flat slabs $V_c =$nominal shear strength of concrete

$V_c$ Shall be smallest of the following:

[Where $\beta_c$ is the ratio of long side to short side of the column, concentrated load or reaction area
and where $\alpha_c$ is 40 for interior columns, 30 for edge columns,20 for corner columns]

(a) $V_c = \left(2 + \frac{4}{\beta_c}\right) \sqrt{f_{c'}b_0d}$

(b) $V_c = \left(\frac{\alpha_c d}{b_0} + 2\right) \sqrt{f_{c'}b_0d}$

(c) $V_c = 4 \sqrt{f_{c'}b_0d}$

**Numerical example:**

consider the slab to be designed with drop’s

Depth of the slab from deflection criteria $= \frac{l_n}{36}$

(for yield stress $f_{yi} = 60,000$ psi $\approx 415$ N/mm$^2$)

$\Rightarrow$ Minimum depth of slab

$= \max \left(\frac{16.76 \times 12}{36}, \frac{14.22 \times 12}{36}\right)$

$= \max (5.58 \text{ in}, 4.74 \text{ in})$

$= 5.58$ in $\uparrow$ $6$ in

6 in $> 4$ in (for slabs with drop panels)

Providing a slab of thickness 6 in or 152.4 mm.

Density of concrete = 150 lb/ft$^3$

Dead load on the slab $= \frac{6}{12} \times (150) = 75$ psf $= 3.6$ KN/m$^2$

Live load on the slab $= 161.80$ psf $= 7.75$ KN/m$^2$

Design load on the slab $= (1.2 \times 7.5 + 1.6 \times 161.80)$

$= 348.88 \approx 350$ psf

$= 16.765$ KN/m$^2$
For short span direction, the total static design moment:

\[ M_0 = \frac{1}{8} \times \frac{350}{1000} \times 16.76 \times 14.22^2 = 148.26 \text{ ft-kips} = 201.04 \text{ KNm} \]

This is distributed as follows:

Negative design moment = 148.06 x 0.65 = 96.24 ft-kips = 130.50 KNm

Positive design moment = 148.06 x 0.35 = 51.891 ft-kips = 70.36 KNm

The column strip has a width of \( 2 \times \frac{14.22}{4} = 7.11 \text{ ft} = 180.59 \text{ mm} \)

With \( \frac{l_2}{l_1} = \frac{16.76}{14.22} = 1.17 ; \alpha_1 = 0 \) (\( \because \) no beams)

**Bending moment for column strip**:

Negative moment for column strip = 75% of total negative moment in the panel
= 0.75 x 96.24 = 72.18 ft-kips = 97.88 KNm

Positive moment for column strip = 60% of total positive moment in the panel.
= 0.60 x 51.891 = 31.135 ft-kips = 42.21 KNm

Static moment along longer direction

\[ M_0 = \frac{1}{8} \times \frac{350}{1000} \times 16.76 \times 14.22 = 174.75 \text{ ft-kips} = 237 \text{ KNm} \]

This is distributed as follows:

Negative design moment = 237 x 0.65 = 154 ft-kips = 208.89 KNm

Positive design moment = 237 x 0.35 = 83.00 ft-kips = 113.22 KNm

The column strip has a width of \( 2 \times \frac{16.76}{4} = 8.38 \text{ ft} = 212.85 \text{ mm} \)

With \( \frac{l_2}{l_1} = \frac{14.22}{16.76} = 0.8484 \)

**Bending moment for column strip**

Negative moment for column strip = 75% of total negative moment in the panel
= 0.75 x 154.00 = 115.50 ft-kips = 157.66 KNm

Positive moment for column strip = 60% of total positive moment in the panel.
= 0.60 x 83.00 = 49.8 ft-kips = 67.977 KNm
Bending moment for middle strip along shorter span

Negative moment for middle strip = 0.25 x 96.24
= 24.06 ft-kips
= 32.84 KNm

Positive moment for middle strip = 0.40 x 51.891
= 20.7564 ft-kips
= 28.33 KNm

Bending moment for middle strip along longer span

Negative moment for middle strip = 0.25 x 154 = 38.5 ft-kip
= 52.55 KNm

Positive moment for middle strip = 0.40 x 83.00 = 33.2 ft-kips
= 45.318 KNm

Max moment (+ve or –ve) along shorter span = 72.18 ft-kips
Max moment (+ve or –ve) along longer span = 115.50 ft-kips

\( \rho_{\text{max}} = \) maximum permitted reinforcement ratio

\[ M_u = \varphi \rho f_y f_d (1 - 0.59 \frac{f_y}{f_d}) \]

\[ M_u = [0.90 \times 0.0206 \times 60,000 \times 14.22 (1 - 0.59 \times 0.0206 \times \frac{60}{4})] \]

\[ d_1^2 = \frac{M_{u,1}}{12193.65} = \frac{72.18}{12193.65} \times 1000 = (2.43)^2 \]

\( \Rightarrow d_1 = 2.43 \text{ in} \approx 61.72 \text{ mm} \)

\[ d_2 = \sqrt{\frac{M_{u,2}}{12193.65}} = \sqrt{\frac{115.50 \times 1000}{12193.65}} = 3.07 \text{ in} = 77.79 \text{ mm} \approx 78 \text{ mm} \]

provide a slab of thickness 6 in.

Drop in flat slabs:

Span of panel in longer direction = 16.76 ft

Length of drop panel
\[ = \frac{1}{6} \times 16.76 \times 2 \]
\[ = 5.58 \text{ ft} \approx 5.60 \text{ ft} \approx 1.71 \text{ m} \]

with half width on either side of the centre line of support = 0.85 m

Thickness of drop = \( \frac{1}{4} \times 6 \) = 1.5 in = 38.1 mm
Check for punching shear:

\( V_u \) = factored shear, acting at distance \( d/2 \) from face of the support.

(assuming column of size 400 mm by 400 mm)

\[
V_u = 350[(16.76 \times 14.22) - (1.31 + 0.5)(1.31 + 0.5)] \\
= 350[238.32 - 1.81^2] = 82265.365 \text{ lb} = 365.91 \text{ KN}
\]

\[
\sqrt{f'_c b_0 d} = \sqrt{4000 \times (4 \times 21.72) \times 6} = 32968.64 \text{ lb}
\]

\( \beta_c = 1.17 \)

The nominal stress of concrete will be smallest of the following:

(a)

\[
V_c = \left( 2 + \frac{4}{\beta_c} \right) \sqrt{f'_c b_0 d} \\
= \left( 2 + \frac{4}{1.17} \right) \times 32968.64 = 178650.57 \text{ lb}
\]

(b)

\[
V_c = \left( \frac{\alpha_x d}{b_0} + 2 \right) \sqrt{f'_c b_0 d} \\
= \left( \frac{40 \times 6}{4 \times 21.72} + 2 \right) \times 32968.64 = 157010.87 \text{ lb}
\]

(c)

\[
V_c = 4 \sqrt{f'_c b_0 d} \\
= 4 \times 32968.64 = 131874.56 \text{ lb}
\]

\[ \Rightarrow V_c = 131874.56 > V_u \]

section safe in punching shear \( \therefore \) safe.
Reinforcement

Depth = 6 ft, Width = 16.76 ft

Minimum area of steel required = 0.0018 x gross area of concrete (for control of temperature & shrinkage cracking)

\[ = 0.0018 \times 6 \times 16.76 = \frac{2.17}{12} = 0.1808 \text{ in}^2 \]

In 14.22 ft direction, \( \rho_{\text{min}} = \frac{0.1808}{6 \times 14.22} = 0.00211 \)

In 16.76 ft direction, \( \rho_{\text{min}} = \frac{0.1808}{6 \times 16.76} = 0.0017 \)

\[ R = \rho f_y \left( 1 - 0.588 \frac{f'_c}{f_y} \right) \text{ psi or } R = \frac{M_u}{\phi bd^2} = \frac{M_u}{0.90 \times 6^2 \times b} = \frac{M_u}{b(324)} \]

Calculation of area of steel: Along shorter span:

For negative moment in column strip:

\[ R = \frac{M_u}{b(324)} = \frac{72.18 \times 10^3}{14.76 \times 32.4} = 150.933 \]

Reinforcement ratio = 0.0040

Area of reinforcement = 0.0040 x 14.76 x 6 x 12 = 4.250 \( \text{in}^2 / \text{ft} \)

Provide Bar No.10, at a spacing of 3.5 in, 7 in number

For positive moment in column strip:

\[ R = \frac{M_u}{b(324)} = \frac{31.135 \times 10^3}{14.76 \times 32.4} = 65 \]

Reinforcement ratio = 0.0017

Area of reinforcement = 0.0017 x 14.76 x 6 x 12 = 1.8066 \( \text{in}^2 / \text{ft} \)

Provide Bar No. 8, at a spacing of 5 in, 4 in number
For negative moment in middle strip:

\[ R = \frac{M_u}{b(324)} = \frac{24.6 \times 10^3}{14.76 \times 32.4} = 50.311 \]

Reinforcement ratio = 0.0013

Area of reinforcement = 0.0013 x 14.76 x 6 x 12 = 1.38 \text{ in}^2 / \text{ft}

Provide Bar No. 6, at a spacing of 4 in, 3 in number

For positive moment in middle strip:

\[ R = \frac{M_u}{b(324)} = \frac{20.75 \times 10^3}{14.76 \times 32.4} = 43.40 \]

Reinforcement ratio = 0.00075

Area of reinforcement = 0.00075 x 14.76 x 6 x 12 = 0.79 \text{ in}^2 / \text{ft}

Provide Bar No. 4, at a spacing of 3 in, 4 in number

Calculation of area of steel: Along longer span:

For negative moment in column strip:

\[ R = \frac{M_u}{b(324)} = \frac{115.50 \times 10^3}{16.22 \times 32.4} = 219.77 \]

Reinforcement ratio = 0.00375

Area of reinforcement = 0.00375 x 16.22 x 6 x 12 = 4.38 \text{ in}^2 / \text{ft}

Provide Bar No. 11, at a spacing of 4 in, 8 in number

For positive moment in column strip:

\[ R = \frac{M_u}{b(324)} = \frac{49.8 \times 10^3}{16.22 \times 32.4} = 94.76 \]

Reinforcement ratio = 0.00175

Area of reinforcement = 0.00175 x 16.22 x 6 x 12 = 2.04 \text{ in}^2 / \text{ft}
Provide Bar No. 7, at a spacing of 3.5 in, 3 in number

For negative moment in middle strip:

\[ R = \frac{M_u}{b(324)} = \frac{38.50 \times 10^3}{16.22 \times 32.4} = 73.25 \]

Reinforcement ratio = 0.00125

Area of reinforcement = 0.00125 \times 16.22 \times 6 \times 12 = 1.4598 \text{ in}^2 / \text{ft}

Provide Bar No. 6, at a spacing of 4 in, 3 in number

For positive moment in middle strip:

\[ R = \frac{M_u}{b(324)} = \frac{33.20 \times 10^3}{16.22 \times 32.4} = 63.17 \]

Reinforcement ratio = 0.00115

Area of reinforcement = 0.00115 \times 16.22 \times 6 \times 12 = 1.34 \text{ in}^2 / \text{ft}

Provide Bar No. 7, at a spacing of 5 in, 7 in number
**Result:** - codal comparisons (ACI,NZS,IS)

<table>
<thead>
<tr>
<th>CODE</th>
<th>IS-456</th>
<th>ACI-318</th>
<th>NZS 3101</th>
<th>Euro code</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shape of test specimen for concrete strength (mm)</td>
<td>Cube 150x150x150</td>
<td>Cylinder 152.4x304.8</td>
<td>Cylinder 152.4x304.8</td>
<td>Cylinder 152.4x304.8</td>
</tr>
<tr>
<td>Grade of concrete (N/mm²)</td>
<td>20</td>
<td>20</td>
<td>30</td>
<td>20</td>
</tr>
<tr>
<td>Grade of steel (N/mm²)</td>
<td>415</td>
<td>413.7</td>
<td>420</td>
<td>500</td>
</tr>
<tr>
<td>Negative moment (KN-m)</td>
<td>188.5</td>
<td>208.9</td>
<td>292.14</td>
<td>192.6</td>
</tr>
<tr>
<td>Positive moments (KN-m)</td>
<td>90</td>
<td>113.22</td>
<td>147.37</td>
<td>135.5</td>
</tr>
<tr>
<td>Area of reinforcement (mm²)</td>
<td>4209</td>
<td>2829</td>
<td>2817</td>
<td>2415.5</td>
</tr>
<tr>
<td>Thickness of slab for Serviceability criteria (mm)</td>
<td>170</td>
<td>150</td>
<td>210</td>
<td>315</td>
</tr>
<tr>
<td>Punching shear</td>
<td>Safe</td>
<td>Safe</td>
<td>Safe</td>
<td>Safe</td>
</tr>
</tbody>
</table>
Conclusions:

- By comparing with different codes we concluded that ACI 318, NZS 3101 & euro codes are most effective in designing of flat slabs.

- As per Indian code we are using cube strength but in international standards cylindered are used which gives higher strength than cube.

- Drops are important criteria in increasing the shear strength of the slab.

- Enhance resistance to punching failure at the junct ion of concrete slab & column.

- By incorporating heads in slab, we are increasing rigidity of slab.

- In the interior span, the total design moments (Mo) are same for IS, NZS, ACI.

- 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.

- According to Indian standard (IS 456) for RCC code has recommended characteristic strength of concrete as 20, 25, and 30 and above 30 for high strength concrete. For design purpose strength of concrete is taken as 2/3 of actual strength this is to compensate the difference between cube strength and actual strength of concrete in structure. After that we apply factor of safety of 1.5. So in practice Indian standard actually uses 46% of total concrete characteristic strength. While in International practice is to take 85% of total strength achieved by test and then apply factor of safety which is same as Indian standard so in actual they use 57% of total strength.

- Pre fabricate sections to be integrated into the design for ease of construction.
References:-

1. Indian standards 456,875.

2. ASI-318

3. NZS:3101

4. Euro code

5. Dr.pradeep kumar ramancharala

6. Reinforced concrete design –S.unnikrishna pillai, Devdas menon

7. R.C.C design ----- S.Ramamrutham