Internal forces in Continuous Beams and Slabs

Dr. Nasreollah H A
3.4.3 Uniformly-loaded continuous beams with approximately equal spans: moments and shears

Table 3.5 may be used to calculate the design ultimate bending moments and shear forces, subject to the following provisions:

a) characteristic imposed load $Q_k$ may not exceed characteristic dead load $G_k$;

b) loads should be substantially uniformly distributed over three or more spans;

c) variations in span length should not exceed 15% of longest.

**Table 3.5 — Design ultimate bending moments and shear forces**

<table>
<thead>
<tr>
<th></th>
<th>At outer support</th>
<th>Near middle of end span</th>
<th>At first interior support</th>
<th>At middle of interior spans</th>
<th>At interior supports</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment</td>
<td>0</td>
<td>0.09$Fl$</td>
<td>-0.11$F_l$</td>
<td>0.07$F_l$</td>
<td>-0.08$F_l$</td>
</tr>
<tr>
<td>Shear</td>
<td>0.45$F$</td>
<td>—</td>
<td>0.6$F$</td>
<td>—</td>
<td>0.55$F$</td>
</tr>
</tbody>
</table>

**NOTE**: $l$ is the effective span;

$F$ is the total design ultimate load $(1.4G_k + 1.6Q_k)$.

No redistribution of the moments calculated from this table should be made.
3.5.2.4 *One-way spanning slabs of approximately equal span: uniformly distributed loads*

Where the conditions of 3.5.2.3 are met, the moments and shears in continuous one-way spanning slabs may be calculated using the coefficients given in Table 3.12. Allowance has been made in these coefficients for the 20% redistribution mentioned above.

The curtailment of reinforcement designed in accordance with Table 3.12 may be carried out in accordance with the provisions of 3.12.10.

**Table 3.12 — Ultimate bending moment and shear forces in one-way spanning slabs**

<table>
<thead>
<tr>
<th></th>
<th>End support/slab connection</th>
<th>At first interior support</th>
<th>Middle interior spans</th>
<th>Interior supports</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Simple</td>
<td>Continuous</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>At outer support</td>
<td>Near middle of end span</td>
<td>At outer support</td>
<td>Near middle of end span</td>
</tr>
<tr>
<td>Moment</td>
<td>0</td>
<td>0.086Fl</td>
<td>-0.04Fl</td>
<td>0.075Fl</td>
</tr>
<tr>
<td>Shear</td>
<td>0.4F</td>
<td>0.46F</td>
<td>0.6F</td>
<td>-</td>
</tr>
</tbody>
</table>

**NOTE**

- $F$ is the total design ultimate load $(1.4G_k + 1.6Q_k)$;
- $l$ is the effective span.
5.13.1 Introduction

When slabs are supported on all four sides, they effectively span in two directions provided that the longer side is no greater than twice the shorter side. It is often more economic to design slabs on this basis and to provide reinforcing steel in both directions to resist the orthogonal bending moments. The magnitude of the bending moment in each direction is dependent on the ratio of the two spans, and the support conditions. In square slabs the load is distributed equally in both directions and in rectangular slabs, the shorter, stiffer span resists a higher percentage of the load than the longer span.

There are two types of slab to be considered:

(i) Simply-supported slabs (Clause 3.5.3.3 and Table 3.13) and
(ii) Restrained slabs (Clause 3.5.3.4 and Table 3.14)

There are nine different types of support condition to be considered which relate to the particular support/restraint provided on each edge of individual slabs; these are illustrated in Figures 5.96 to 5.99.
<table>
<thead>
<tr>
<th>Type</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type 1</td>
<td><em>Interior panel</em></td>
</tr>
<tr>
<td>Type 2</td>
<td><em>One short edge discontinuous</em></td>
</tr>
<tr>
<td>Type 3</td>
<td><em>One long edge discontinuous</em></td>
</tr>
<tr>
<td>Type 4</td>
<td><em>Two adjacent edges discontinuous</em></td>
</tr>
<tr>
<td>Type 5</td>
<td><em>Two short edges discontinuous</em></td>
</tr>
<tr>
<td>Type 6</td>
<td><em>Two long edges discontinuous</em></td>
</tr>
<tr>
<td>Type 7</td>
<td><em>Three edges discontinuous (one long edge continuous)</em></td>
</tr>
<tr>
<td>Type 8</td>
<td><em>Three edges discontinuous (one short edge continuous)</em></td>
</tr>
<tr>
<td>Type 9</td>
<td><em>Four edges discontinuous</em></td>
</tr>
</tbody>
</table>
5.13.2 Simply-Supported Slabs

Simply-supported slabs do not have adequate provision to resist torsion at the corners. The maximum design bending moments permitted to prevent lifting and to resist the applied ultimate bending moment can be determined using Equations (10) and (11) in the code:

\[ m_{sx} = \alpha_{sx} n l_x^2 \]
\[ m_{sy} = \alpha_{sy} n l_x^2 \]

Equation (10)
Equation (11)

**Figure 5.100**

**Figure 5.101**
Table 3.13 — Bending moment coefficients for slabs spanning in two directions at right angles, simply-supported on four sides

<table>
<thead>
<tr>
<th>$l_y/l_x$</th>
<th>1.0</th>
<th>1.1</th>
<th>1.2</th>
<th>1.3</th>
<th>1.4</th>
<th>1.5</th>
<th>1.75</th>
<th>2.0</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha_{sx}$</td>
<td>0.062</td>
<td>0.074</td>
<td>0.084</td>
<td>0.093</td>
<td>0.099</td>
<td>0.104</td>
<td>0.113</td>
<td>0.118</td>
</tr>
<tr>
<td>$\alpha_{sy}$</td>
<td>0.062</td>
<td>0.061</td>
<td>0.059</td>
<td>0.055</td>
<td>0.051</td>
<td>0.046</td>
<td>0.037</td>
<td>0.029</td>
</tr>
</tbody>
</table>

where:

- $m_{sx}$ is the maximum design ultimate moment at mid-span on a strip of unit width and span $l_x$.
- $m_{sy}$ is the maximum design ultimate moment at mid-span on a strip of unit width and span $l_y$.
- $n$ is the total design ultimate load/unit area $= (1.4g_k + 1.6q_k)$.
- $l_x$ is the length of the shorter side.
- $l_y$ is the length of the longer side.
Table 3.14 — Bending moment coefficients for rectangular panels supported on four sides with provision for torsion at corners

<table>
<thead>
<tr>
<th>Type of panel and moments considered</th>
<th>Short span coefficients, $\beta_{sx}$</th>
<th>Long span coefficients, $\beta_{sy}$ for all values of $l_y/l_x$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Values of $l_y/l_x$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>1.0</td>
<td>1.1</td>
</tr>
<tr>
<td>Interior panels</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative moment at continuous edge</td>
<td>0.031</td>
<td>0.037</td>
</tr>
<tr>
<td>Positive moment at mid-span</td>
<td>0.024</td>
<td>0.028</td>
</tr>
<tr>
<td>One short edge discontinuous</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative moment at continuous edge</td>
<td>0.039</td>
<td>0.044</td>
</tr>
<tr>
<td>Positive moment at mid-span</td>
<td>0.029</td>
<td>0.033</td>
</tr>
<tr>
<td>One long edge discontinuous</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Negative moment at continuous edge</td>
<td>0.039</td>
<td>0.049</td>
</tr>
<tr>
<td>Positive moment at mid-span</td>
<td>0.030</td>
<td>0.036</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>----------------------------------------------------------------</td>
<td>-----------------</td>
<td>-----------------</td>
</tr>
<tr>
<td>Two adjacent edges discontinuous</td>
<td>0.047</td>
<td>0.056</td>
</tr>
<tr>
<td>Negative moment at continuous edge</td>
<td>0.036</td>
<td>0.042</td>
</tr>
<tr>
<td>Positive moment at mid-span</td>
<td>0.034</td>
<td>0.038</td>
</tr>
<tr>
<td>Two short edges discontinuous</td>
<td>0.046</td>
<td>0.050</td>
</tr>
<tr>
<td>Negative moment at continuous edge</td>
<td>0.034</td>
<td>0.038</td>
</tr>
<tr>
<td>Positive moment at mid-span</td>
<td>0.034</td>
<td>0.046</td>
</tr>
<tr>
<td>Two long edges discontinuous</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Negative moment at continuous edge</td>
<td>0.034</td>
<td>0.046</td>
</tr>
<tr>
<td>Positive moment at mid-span</td>
<td>0.034</td>
<td>0.046</td>
</tr>
<tr>
<td>Three edges discontinuous (one long edge continuous)</td>
<td>0.057</td>
<td>0.065</td>
</tr>
<tr>
<td>Negative moment at continuous edge</td>
<td>0.043</td>
<td>0.048</td>
</tr>
<tr>
<td>Positive moment at mid-span</td>
<td>0.043</td>
<td>0.054</td>
</tr>
<tr>
<td>Three edges discontinuous (one short edge continuous)</td>
<td>—</td>
<td>—</td>
</tr>
<tr>
<td>Negative moment at continuous edge</td>
<td>0.042</td>
<td>0.054</td>
</tr>
<tr>
<td>Positive moment at mid-span</td>
<td>0.055</td>
<td>0.065</td>
</tr>
</tbody>
</table>
5.13.7 Example 5.24: Simply Supported Two-Way Spanning Slab

A floor slab in an office building measures 5.0 m × 7.5 m and is simply supported at the edges with no provision to resist torsion at the corners or to hold the corners down. Using the design data given, determine suitable reinforcement.

Design Data:
Characteristic dead load due to finishes and services
Characteristic imposed load
Concrete grade
Characteristic strength of reinforcing steel
Self-weight of concrete
Exposure condition

\[
\begin{align*}
g_k &= 1.25 \text{ kN/m}^2 \\
q_k &= 2.5 \text{ kN/m}^2 \\
f_{cu} &= 40 \text{ N/mm}^2 \\
f_y &= 460 \text{ N/mm}^2 \\
\gamma_{concrete} &= 24 \text{ kN/m}^3 \text{ mild}
\end{align*}
\]

Figure 5.103
<table>
<thead>
<tr>
<th>References</th>
<th>Calculations</th>
<th>Output</th>
</tr>
</thead>
</table>
| Initial sizing of the section:  
The initial trial section depth can be based on deflection i.e.  
Table 3.9 value \[\frac{\text{basic span}}{\text{effective depth}} = 20\]  
(This will be modified using the Table 3.10 value as given in Clause 3.4.6.5 for the final deflection check) |
| Estimated effective depth required \[d \geq \frac{5000}{20} = 250 \text{ mm.}\]  
For C40 concrete with mild exposure steel cover \[\geq 20 \text{ mm}\]  
Assume 12 mm diameter bars:  
The required overall thickness \[h \geq (250 + 6 + 20) = 276 \text{ mm}\]  
Try a 275 mm thick slab.  
Actual effective depth for the short span:  
\[d = (275 - 20 - 6) = 249 \text{ mm}\]  
Actual effective depth for the long span:  
\[d = (275 - 20 - 12 - 6) = 237 \text{ mm}\]  
Design loading:  
Self-weight of slab \[(0.275 \times 24) = 6.6 \text{ kN/m}^2\]  
Finishes and services \[= 1.25 \text{ kN/m}^2\]  
Characteristic dead load \[g_k = 7.85 \text{ kN/m}^2\] |
<table>
<thead>
<tr>
<th>Table 3.13</th>
</tr>
</thead>
</table>
| Ultimate design load \[ [1.4 \times g_k] + (1.6 \times q_k) \]  
\[ = [(1.4 \times 7.85) + (1.6 \times 2.5)] \]  
\[ = 15.0 \text{ kN/m}^2 \]  
\[ l_y/l_x = 7.5/5.0 = 1.5 \]  
\[ \alpha_{sx} = 0.104 \]  
\[ \alpha_{sy} = 0.046 \]  
<table>
<thead>
<tr>
<th>Clause 3.5.3.3</th>
</tr>
</thead>
</table>
| Design moment in the direction of span \( l_x \)  
\[ m_{sx} = \alpha_{sx} n l_x^2 \]  
\[ = (0.104 \times 15.0 \times 5^2) \]  
\[ = 39.0 \text{ kNm/m width} \]  
| Design moment in the direction of span \( l_y \)  
\[ m_{sy} = \alpha_{sy} n l_x^2 \]  
\[ = (0.046 \times 15.0 \times 5^2) \]  
\[ = 17.25 \text{ kNm/m width} \]
<table>
<thead>
<tr>
<th>References</th>
<th>Calculations</th>
<th>Output</th>
</tr>
</thead>
</table>
| Clause 3.4.4.4 | Reinforcement: short span  
\[ K = \frac{M}{bd^2 f_{cu}} = \frac{39.0 \times 10^6}{1000 \times 249^2 \times 40} = 0.016 \quad < \quad 0.156 \]  
Section is singly reinforced  
\[ Z = d \left\{ 0.5 + \sqrt{\left( \frac{0.25 - K}{0.9} \right)} \right\} = d \left\{ 0.5 + \sqrt{\left( \frac{0.25 - 0.016}{0.9} \right)} \right\} = 0.98d \]  
The lever arm is limited to 0.95d  
For slabs of rectangular section and  
\[ f_y = 460 \text{ N/mm}^2 \]  
100\(A_s/A_c\) ≥ 0.13%  
Minimum \(A_s\) required = \(\frac{0.13 \times b \times h}{100}\) mm²  
\[ = \frac{0.13 \times 1000 \times 275}{100} = 358 \text{ mm}^2 \]  
< 452 mm²  
Minimum % reinforcement satisfied  
|               |                                                                                   |        |
| Clause 3.4.4.4 | Reinforcement: long span  
\[ K = \frac{M}{bd^2 f_{cu}} = \frac{17.25 \times 10^6}{1000 \times 237^2 \times 40} = 0.008 \quad < \quad 0.156 \]  
Section is singly reinforced  
|               |                                                                                   |        |
\[
Z = d \left\{ 0.5 + \sqrt{\left(0.25 - \frac{K}{0.9}\right)} \right\} = d \left\{ 0.5 + \sqrt{\left(0.25 - \frac{0.008}{0.9}\right)} \right\} = 0.99d
\]

The lever arm is limited to 0.95d

\[
A_s = \frac{M}{0.95 f_y Z} = \frac{17.25 \times 10^6}{0.95 \times 460 \times 0.95 \times 237} = 175 \text{ mm}^2/\text{m width}
\]

Minimum area of reinforcement required \( A_s = 358 \text{ mm}^2 \) (as before)

\[
< 377 \text{ mm}^2
\]

Minimum % reinforcement satisfied

<table>
<thead>
<tr>
<th>Table 3.25</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bottom Reinforcement</td>
</tr>
</tbody>
</table>

Select:
12 mm diameter bars @ 300 mm centres providing 377 mm\(^2\)/m at mid-span
Clause 3.12.11.2

Maximum spacing of reinforcement:

\[ \leq (3 \times d) = (3 \times 237) = 711 \text{ mm} \]
\[ \leq 750 \text{ mm} \]

Since there is less than 0.3% reinforcement, no further check is required on spacing.

Clause 3.12.10

There is no curtailment since the reinforcement would be less than 0.13%.

Clause 3.5.5

Shear Resistance

Clause 3.5.5.2

Shear stress \[ \nu = \frac{V}{bd} = \frac{37.5 \times 10^3}{1000 \times 237} = 0.16 \text{ N/mm}^2 \]

Maximum shear \[ \leq 0.8 \sqrt{f_{cu}} = (0.8 \times \sqrt{40}) \]
\[ = 5.05 \text{ N/mm}^2 \]
\[ \leq 5.0 \text{ N/mm}^2 \]

\[ \nu < \text{ maximum permitted value} \]

\[ \frac{100A_s}{bd} = \frac{100 \times 377}{1000 \times 237} = 0.16; \quad d = 237 \]

Table 3.8

\[ \nu_c \approx (0.38 \times 1.17) = 0.44 \text{ N/mm}^2 \]
(Note: This allows for using C40 concrete)

Table 3.16

\[ \nu < \nu_c \]

No links are required
Clause 3.5.7

Deflection:
As for beams in Clause 3.4.6.3 using values for shorter span
\[
\frac{M}{bd^2} \leq \text{Table 3.9 value} \times \text{Table 3.10 value}
\]

\[
\frac{M}{bd^2} = K \times f_{cu} = 0.016 \times 40 = 0.64
\]

Service stress \( f_s = \frac{2 \times f_y \times A_{s,\text{required}}}{3 \times A_{s,\text{provided}}} = \frac{2 \times 460 \times 377}{3 \times 452} = 256 \text{ N/mm}^2 \)

Interpolate between values given in Table 3.10

<table>
<thead>
<tr>
<th>service stress ( f_s )</th>
<th>( M/bd^2 )</th>
<th>0.50</th>
<th>0.75</th>
</tr>
</thead>
<tbody>
<tr>
<td>250</td>
<td></td>
<td>1.90</td>
<td>1.70</td>
</tr>
<tr>
<td>300</td>
<td></td>
<td>1.60</td>
<td>1.44</td>
</tr>
</tbody>
</table>

Use conservative estimate of modification factor \( \approx 1.44 \)
<table>
<thead>
<tr>
<th>References</th>
<th>Calculations</th>
<th>Output</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard 90° bend at support</td>
<td>Table 3.9 value $\times$ Table 3.10 value = $20 \times 1.44 = 28.8$</td>
<td>Adequate with respect to deflection</td>
</tr>
<tr>
<td></td>
<td>Actual $\frac{\text{span}}{\text{effective depth}} = \frac{5000}{249} = 20.1 &lt; 28.8$</td>
<td></td>
</tr>
</tbody>
</table>

![Diagram of a beam with 90° bends at supports, T12 @ 250 c/c and T12 @ 300 c/c, 5000 mm span.]
5.13.9 Example 5.25: Two-Way Spanning Restrained Slab

A part-floor plan for an office building is shown in Figure 5.104. The floor consists of restrained 180 mm thick slabs poured monolithically with edge beams. Using the design data given, design suitable reinforcement for a corner slab.

**Design Data:**
- Characteristic total dead load (including self-weight)
- Characteristic imposed load
- Concrete grade
- Characteristic strength of reinforcing steel
- Exposure

\[
\begin{align*}
g_k &= 6.2 \text{ kN/m}^2 \\
q_k &= 2.5 \text{ kN/m}^2 \\
f_{cu} &= 40 \text{ N/mm}^2 \\
f_y &= 460 \text{ N/mm}^2 \\
&\text{mild}
\end{align*}
\]

![Diagram of a part-floor plan for an office building showing a corner slab A with dimensions and load calculations.]
<table>
<thead>
<tr>
<th>References</th>
<th>Calculations</th>
<th>Output</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Design loading:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Characteristic dead load</td>
<td>( g_k = 6.2 \text{ kN/m}^2 )</td>
</tr>
<tr>
<td></td>
<td>Characteristic imposed load</td>
<td>( q_k = 2.5 \text{ kN/m}^2 )</td>
</tr>
<tr>
<td></td>
<td>Ultimate design load</td>
<td>[ (1.4 \times g_k) + (1.6 \times q_k) ]</td>
</tr>
<tr>
<td>Table 3.14</td>
<td>From Figure 5.96 of the text the corner slab is ‘type 4’</td>
<td></td>
</tr>
<tr>
<td>Clause 3.5.3.5</td>
<td>The slab is divided into middle and edge strips as indicated in Figure 3.9 of the code.</td>
<td></td>
</tr>
<tr>
<td>Figure 3.9</td>
<td>( \frac{l_y}{8} = \frac{l_x}{8} = \frac{6.0}{8} = 0.75 \text{ m} )</td>
<td></td>
</tr>
</tbody>
</table>

![Diagram](image)

(a) For span \( l_y \)

(b) For span \( l_x \)
Table 3.14

\[ l_x = l_y = 6.0 \text{ m} \quad \frac{l_y}{l_x} = 1 \]

Table 3.3

For C40 concrete with mild exposure steel cover \( \geq 20 \text{ mm} \). In most cases of this type, the cover requirements regarding 1 hour fire protection are less onerous than exposure requirements. Similarly, strength/deflection thickness requirements normally exceed those required for fire protection. The reader should refer to Table 3.4 and Figure 3.2 of the code.

Thickness of slab = 180 mm
Assume 10 mm diameter bars:
The effective depth for the outer layer \( d \) = (180 - 20 - 5) = 155 mm
the effective depth for the inner layer \( d \) = (180 - 20 - 10 - 5) = 145 mm
### Reinforcement for the middle strip of span $l_x$

$l_y/l_x = 1.0$

Figure 5.104  
Position 1  $\beta_{sx} = -0.047$  
Position 2  $\beta_{sx} = +0.036$

#### Position 1 (continuous edge)

Design moment at position 1:

Clause 3.5.3.4  
\[
m_{sx} = \beta_{sx} n l_x^2 = -(0.047 \times 12.68 \times 6^2) = -21.45 \text{ kNm/m width}
\]

Clause 3.4.4.4  
\[
K = \frac{M}{bd^2 f_{cu}} = \frac{21.45 \times 10^6}{1000 \times 155^2 \times 40} = 0.022 < 0.156
\]

Section is singly reinforced

\[
Z = d \left\{ 0.5 + \sqrt{\frac{0.25 - \frac{K}{0.9}}{0.9}} \right\}
\]

\[
= d \left\{ 0.5 + \sqrt{\frac{0.25 - 0.022}{0.9}} \right\} = 0.98d
\]

The lever arm is limited to $0.95d$

Select:

- Top Reinforcement
- 10 mm diameter bars @ 200 mm centres providing 393 mm$^2$/m over continuous edge.

### Table 3.14

<table>
<thead>
<tr>
<th>$A_s$</th>
<th>$M$</th>
<th>$0.95 f_y Z$</th>
<th>$0.95 \times 460 \times 0.95 \times 155$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$= \frac{M}{0.95 f_y Z}$</td>
<td>$= \frac{21.45 \times 10^6}{0.95 \times 460 \times 0.95 \times 155}$</td>
<td>$= 333 \text{ mm}^2/\text{m width}$</td>
<td></td>
</tr>
</tbody>
</table>

### Table 3.25

Actual area of reinforcement = $\frac{100 \times 393}{1000 \times 180} = 0.22\% > 0.13\%$
Table 3.14

Reinforcement for the middle strip of span $l_x$

\[ \frac{l_y}{l_x} = 1.0 \]

Figure 5.104

Position 1 $\beta_{sx} = -0.047$

Position 2 $\beta_{sx} = +0.036$

Position 1 (continuous edge)
Design moment at position 1:

Clause 3.5.3.4

\[ m_{sx} = \beta_{sx} n l_x^2 = -(0.047 \times 12.68 \times 6^2) \]
\[ = -21.45 \text{ kNm/m width} \]

Clause 3.4.4.4

\[ K = \frac{M}{bd^2 f_{cu}} = \frac{21.45 \times 10^6}{1000 \times 155^2 \times 40} = 0.022 < 0.156 \]

Section is singly reinforced

\[ Z = d \left\{ 0.5 + \sqrt{0.25 - \frac{K}{0.9}} \right\} = d \left\{ 0.5 + \sqrt{0.25 - \frac{0.022}{0.9}} \right\} \]
\[ = 0.98d \]
The lever arm is limited to 0.95d

Top Reinforcement

Select:
10 mm diameter bars @ 200 mm centres
providing 393 mm$^2$/m over continuous edge.

Table 3.25

Actual area of reinforcement = \[ \frac{100 \times 393}{1000 \times 180} = 0.22\% > 0.13\% \]
| Table 3.25 | Actual area of reinforcement $= \frac{100 \times 314}{1000 \times 180} = 0.17\% > 0.13\%$

**Position 3 (discontinuous edge)**

At least 50% of bottom steel should be provided in the top at the support and not less than the minimum area required.

$A_s \text{ required} = (0.5 \times 255) = 128 \text{ mm}^2/\text{m width}$

Minimum area of reinforcement $= \frac{0.13 \times 1000 \times 180}{100} = 234 \text{ mm}^2/\text{m}$

<table>
<thead>
<tr>
<th>Top Reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Select: 8 mm diameter bars @ 200 mm centres providing 252 mm$^2$/m at mid-span</td>
</tr>
</tbody>
</table>
Clause 3.5.5
Table 3.15
\[ l_y / l_x = 1.0 \]

Figure 5.104  position 1  \( \beta_{xy} = 0.4 \)
position 3  \( \beta_{xy} = 0.26 \)

**Position 1 (continuous edge)**
Design shear at position 1:

Clause 3.5.3.7
\[ \nu_{sx} = \beta_{xy} n l_x = (0.4 \times 12.68 \times 6.0) = 30.43 \text{ kN/m} \]

Clause 3.5.5
Shear resistance

Clause 3.5.5.2
Shear stress \( \nu = \frac{V}{bd} = \frac{30.43 \times 10^3}{1000 \times 155} = 0.196 \text{ N/mm}^2 \)

Maximum shear \( \leq 0.8 \sqrt{f_{cu}} = (0.8 \times \sqrt{40}) \)
\( = 5.05 \text{ N/mm}^2 \)
\( \leq 5.0 \text{ N/mm}^2 \)

\( \nu < \) maximum permitted value

\[ \frac{100A_s}{bd} = \frac{100 \times 393}{1000 \times 155} = 0.25; \quad d = 155 \]

Table 3.8
\( \nu_c \approx (0.5 \times 1.17) = 0.58 \text{ N/mm}^2 \)
(Note: 1.17 – allows for using C40 concrete)

Table 3.16
\( \nu < \nu_c \)

No links are required