## Amendments to the Code of Practice for Structural Use of Concrete 2013

<table>
<thead>
<tr>
<th>Item</th>
<th>Clause/Annex</th>
<th>Current Version</th>
<th>Amendments</th>
<th>Remarks</th>
</tr>
</thead>
</table>
| 1    | Clause 1.1   | The following are outside the scope of this Code of Practice:  
(a) particular aspects of special types of buildings and civil engineering works, such as membrane, shell and composite structures, viaducts, dams, pressure vessels, and reservoirs  
(b) no fines concrete, aerated concrete, glass fibre reinforced concrete, and concrete containing lightweight or heavy aggregate or structural steel sections. | The following are outside the scope of this Code of Practice  
(a) particular aspects of special types of buildings and civil engineering works, such as membrane, composite structures, viaducts, dams, pressure vessels, and reservoirs  
(b) no fines concrete, aerated concrete, glass fibre reinforced concrete, and concrete containing lightweight or heavy aggregate or structural steel sections. | The word “shell” is deleted as clause 5.2.1.1 covers the design of shell structures. |
| 2    | Clause 1.4.1 | **General terms**  
acceptable standards standards acceptable to the Building Authority (BA) as given in Annex A  
cantilever projecting structure a structural element that cantilevers from the main structure for example, canopies, balconies, bay windows, air conditioning platforms etc.  
design working life the period of time during which a structure that has undergone normal maintenance is unlikely to require major repairs | **General terms**  
acceptable standards standards acceptable to the Building Authority (BA) as given in Annex A  
cantilever projecting structure a structural element that cantilevers from the main structure for example, canopies, balconies, bay windows, air conditioning platforms etc.  
cementitious content the combined mass of cement, silica fume and either pulverised fuel ash or ground granulated blast furnace slag per cubic metre of compacted concrete. For silica fume, the dry mass shall be used  
free water/cement ratio the ratio between the mass of the free water in the concrete mix and the cementitious content  
design working life the period of time during which a structure that has undergone normal maintenance is unlikely to require major repairs | Definitions of “cementitious content” and “free water/cement ratio” are given. |
| 3    | Clause 1.5   | \( f_{cu} \) Characteristic compressive strength of concrete  
\( f_{p0} \) design tensile stress in the tendons  
\( f_{pe} \) design effective prestress in the tendons after all losses  
\( f_{pu} \) estimated design service stress in the tension reinforcement  
\( f_y \) characteristic yield strength of reinforcement  
\( f_y \), \( f_y \) characteristic yield strength of the shear reinforcement  
\( f_y \), \( f_y \) characteristic yield strength of the shear reinforcement  
\( f_y \) characteristic dead load  
\( h \) depth of cross section measured in the plane under consideration, or thickness of wall  
\( b_{agg} \) maximum size of coarse aggregate  
\( t_{bf} \) thickness of a beam flange  
\( t_b \) effective span of a beam or slab  
\( l_{pr} \) basic anchorage length for reinforcement  
\( l_{e} \) effective height of a column or wall in the plane of bending considered  
\( M \) design ultimate moment at the section considered  
\( N \) design ultimate axial force  
\( n_b \) number of bars in a reinforcement bundle  
\( R_b \) characteristic imposed load  
\( R_{ts} \) tensile strength | \( f_{cu} \) Characteristic compressive strength of concrete  
\( f_{p0} \) design tensile stress in the tendons  
\( f_{pe} \) design effective prestress in the tendons after all losses  
\( f_{pu} \) estimated design service stress in the tension reinforcement  
\( f_y \) specified characteristic yield strength  
\( f_y \) characteristic yield strength of the shear reinforcement  
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\( f_y \) characteristic dead load  
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\( n_b \) number of bars in a reinforcement bundle  
\( R_b \) characteristic imposed load  
\( R_{ts} \) tensile strength | Definition of symbols \( f_y \) and \( R_b \) is unified and amended to “specified characteristic yield strength” and “tensile strength” respectively. |
Legend: revision/addition

3 Clause 1.5

Clause 1.5

Clause 1.5

Clause 2.2.3.2

Clause 2.2.3.2

Clause 3.2.3 & Table 3.3

Clause 3.2.3 & Table 3.3

Legend: revision/addition

Definition of symbols $s_f$, $v_f$, $v_x$, $v_y$, $\beta_x$, $\beta_y$, $\Delta_x$, and $\Delta_F$ are added.

Check of structural integrity

Check of structural integrity

A note to specify the circumstances where fire limit state checking is added.

Strength classes

Strength classes

Table 3.3 - Strength of reinforcement

Table 3.3 - Strength of reinforcement

Legend: revision/addition

Definition of symbol $f_y$ is unified and amended to "specified characteristic yield strength".

Table 3.3 - Strength of reinforcement

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Clause 1.5

Clause 1.5

Clause 2.2.3.2

Clause 2.2.3.2

Clause 3.2.3 & Table 3.3

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Legend: revision/addition

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Legend: revision/addition

Definition of symbol $f_y$ is unified and amended to "specified characteristic yield strength".
<table>
<thead>
<tr>
<th></th>
<th>Clause  3.2.8.4</th>
<th>Performance of type 2 mechanical couplers</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Type 2 mechanical coupler should satisfy the following criteria:</td>
<td>(a) The splicing assemblies shall be tested to establish that they comply with the requirements given in clause 3.2.8.3.</td>
</tr>
<tr>
<td></td>
<td>(b) Static tension test. The splicing assemblies must develop in tension the greater of 100 percent of the specified tensile strength, ( f_y ), of the bar. and 125 percent of the specified yield strength, ( f_y ), of the bar.</td>
<td>(b) Static tension test. The splicing assemblies must develop in tension the greater of 100 percent of the specified tensile strength, ( f_y ), of the bar, and 125 percent of the specified characteristic yield strength, ( f_y ), of the bar.</td>
</tr>
<tr>
<td></td>
<td>(c) Static compression test. The splicing assemblies must develop in compression 125 percent of the specified yield strength, ( f_y ), of the bar.</td>
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</tr>
<tr>
<td></td>
<td>(d) Cyclic tension-compression test. The splicing assemblies shall be tested in four stages as given in Table 3.4, and must sustain Stages 1 through 3 without failure.</td>
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</tr>
<tr>
<td></td>
<td>If the conditions of acceptance for the static tension test are complied with in Stage 4, the static tension test may be omitted.</td>
<td></td>
</tr>
</tbody>
</table>

**Definition of symbols** \( f_y \) and \( R_m \) is unified and amended to “specified characteristic yield strength” and “tensile strength” respectively.

---

<table>
<thead>
<tr>
<th></th>
<th>Clause  4.2.1</th>
<th>Performance of type 2 mechanical couplers</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>The original term “cement content” is replaced by “cementitious content”. The definition for “cementitious content” is given in Clause 1.4.1 in item 2 above.</td>
<td>(c) the environment (clause 4.2.3);</td>
</tr>
<tr>
<td></td>
<td>The definition for “cementitious content” is given in Clause 1.4.1 in item 2 above.</td>
<td>(d) the type of cementitious material(s) (clauses 4.2.5 and 4.2.7);</td>
</tr>
<tr>
<td></td>
<td>Consequential amendments (see remarks for Clause 4.2.1 in item 7 above).</td>
<td>(e) the type of aggregate (clauses 4.2.5 and 4.2.7);</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(f) the cementitious content and water/cement ratio of the concrete (clause 4.2.6);</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(g) workmanship, to obtain full compaction and efficient curing (clauses 10.3.5 and 10.3.6).</td>
</tr>
</tbody>
</table>

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</tr>
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<tr>
<td></td>
<td>Where the minimum dimension of the concrete to be placed in one continuous operation is greater than 600 mm, and especially where the cement content is 400 kg/m³ or more, measures to reduce the temperature such as using material with a slower release of heat of hydration should be considered.</td>
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<td>Consequential amendments (see remarks for Clause 4.2.1 in item 7 above).</td>
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</table>
### Table 4.2 - Nominal cover to all reinforcement (including links) and minimum concrete grade to meet durability requirements for reinforced and prestressed concrete

<table>
<thead>
<tr>
<th>Conditions of exposure (see clause 4.2.3)</th>
<th>Nominal cover (mm)</th>
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</tr>
</thead>
<tbody>
<tr>
<td>Lowest grade of concrete</td>
<td></td>
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<td></td>
</tr>
<tr>
<td>Condition 1 - slabs only</td>
<td>25</td>
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<td>25</td>
</tr>
<tr>
<td>- other members</td>
<td>25</td>
<td>- other members</td>
<td>25</td>
</tr>
<tr>
<td>Condition 2</td>
<td>35</td>
<td>Condition 2</td>
<td>35</td>
</tr>
<tr>
<td>Condition 3</td>
<td>45</td>
<td>Condition 3</td>
<td>45</td>
</tr>
<tr>
<td>Condition 4</td>
<td>50</td>
<td>Condition 4</td>
<td>50</td>
</tr>
<tr>
<td>Condition 5 (see note 3)</td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Maximum free water/cement ratio</th>
<th>Minimum cement content (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.65</td>
<td>290</td>
</tr>
<tr>
<td>0.65</td>
<td>290</td>
</tr>
<tr>
<td>0.60</td>
<td>300</td>
</tr>
<tr>
<td>0.55</td>
<td>340</td>
</tr>
<tr>
<td>0.45</td>
<td>380</td>
</tr>
<tr>
<td>0.40</td>
<td>380</td>
</tr>
</tbody>
</table>

### Notes:
1. This table relates to normal-weight aggregates of 20 mm nominal size. Adjustments to minimum cement contents for aggregates of nominal sizes other than 20 are given in clause 4.2.5.4.
2. Cover not less than the nominal cover corresponding to the environmental exposure condition plus any allowance for loss of cover due to abrasion.
3. Consideration should also be given to cover requirements for fire protection (see clause 4.3.7) and the safe transmission of bond forces (see clause 6.7).
4. For prestressed concrete, grade C30 or lower should not be used and the minimum cement content should be 300 kg/m³.

### Table 4.2 - Nominal cover to all reinforcement (including links) and minimum concrete grade to meet durability requirements for reinforced and prestressed concrete

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### Notes:
1. This table relates to normal-weight aggregates of 20 mm nominal size. Adjustments to minimum cementitious contents for aggregates of nominal sizes other than 20 are given in clause 4.2.5.4.
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---

**Legend:**
- Revision/addition
- Consequential amendments (see remarks for Clause 4.2.1 in item 7 above).
### Consequential amendments (see remarks for Clause 4.2.1 in item 7 above).

**Clause 4.2.5.4 & Table 4.3**

**Adjustment to cement contents for different sized aggregates**

The minimum cement contents given in Tables 4.2 relate to 20 mm nominal maximum size of aggregate. For other sizes of aggregate the values should be modified as given in Table 4.3 subject to the condition that the cement content should not be less than 240 kg/m³ for the exposure conditions covered by Table 4.2.

<table>
<thead>
<tr>
<th>Nominal maximum aggregate size (mm)</th>
<th>Adjustment to minimum cement contents (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>40</td>
</tr>
<tr>
<td>14</td>
<td>20</td>
</tr>
<tr>
<td>20</td>
<td>0</td>
</tr>
<tr>
<td>40</td>
<td>-30</td>
</tr>
</tbody>
</table>

**Table 4.3 - Adjustments to minimum cement contents for aggregates other than 20 mm nominal maximum size**

**Legend:** revision/addition

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**Clause 4.2.6.1**

**Consequential amendments (see remarks for Clause 4.2.1 in item 7 above).**

The free water/cement ratio is an important factor in the durability of concrete and should always be the lowest value compatible with producing fully compacted concrete without segregation or bleeding. Appropriate values for the maximum free water/cement ratio are given in Tables 4.2 and 4.4 for particular exposure conditions.

A minimum cement content is required to ensure a long service life under particular exposure conditions, and appropriate values are given in Tables 4.2 and 4.4. However, the cement content required for a particular water/ cement ratio can vary significantly for different mix constituents. Where adequate workability is difficult to obtain at the maximum free water/cement ratio allowed, an increased cement content, the use of fly ash or ground-granulated blast furnace slag, or the use of water-reducing admixtures should be considered.

For normal strength concrete, i.e. $f_{cm} \leq 60$ N/mm², a total cementitious content including cement and fly ash or ground-granulated blast furnace slag in excess of 600 kg/m³ should not be used unless special consideration has been given in design to the increased risk of cracking due to drying shrinkage in thin sections or to thermal stresses in thicker sections. For high strength concrete ($f_{cm} > 60$ N/mm²), total cementitious contents should be controlled to avoid large heat of hydration as well as large shrinkage and creep strains. Under normal circumstances, the cement content should be limited to not more than 400 kg/m³.

For concrete made with normal-weight aggregate and used in foundations to low rise structures in non-aggressive soil conditions, a minimum grade of C25 may be used where the minimum cement content is not less than 200 kg/m³.

For high strength concrete, reference should also be made to requirements in clause 4.3.
Clause 4.2.6.2 & Table 4.4
Table 4.4 gives recommended values for the maximum free water/cement ratio, the minimum cement content and the lowest grade of concrete to ensure long service life under the appropriate conditions of exposure.

<table>
<thead>
<tr>
<th>Condition of exposure (see clause 4.2.3.2)</th>
<th>Concrete not containing embedded metal</th>
<th>Concrete containing embedded metal</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Maximum free water/cement ratio</td>
<td>Minimum cement content (kg/m³)</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.65</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>0.65</td>
</tr>
</tbody>
</table>

Notes:
1. See clause 4.2.6.3 for adjustments to the mix proportions.
2. See clause 4.2.5.2 for permitted reduction in concrete grade.
3. See clause 4.2.6.1 for concrete used in foundations to low rise structures in non-aggressive soil conditions.

Table 4.4 - Durability of unconfined concrete made with normal-weight aggregates of 20 mm nominal maximum size

Clause 4.2.6.3
Mix adjustments
The cement contents given in Table 4.4 apply to 20 mm nominal maximum size aggregate. For other sizes of aggregate they should be changed as given in Table 4.3. Different aggregates require different water contents to produce concrete of the same workability and therefore at a given cement content, different water/cement ratios are obtained. In order to achieve a satisfactory workability at the specified maximum free water/cement ratio, it may be necessary to modify the mix as described in clause 4.2.6.1. When pfa or ggbs is used, the total content of cement plus pfa or ggbs should be at least as great as the values given in tables 4.2 and 4.4. In these conditions the word 'cement' in cement content and 'water/cement' ratio means the total content of cement plus pfa or ggbs. Good curing is essential with concrete made from these materials (see clause 10.3.6).

Consequential amendments (see remarks for Clause 4.2.1 in item 7 above).

Clause 4.3.1.2(b)
Methods to reduce risk of concrete spalling
At least one of the following methods should be provided.
(a) Method A: A reinforcement mesh with a nominal cover of 15mm. This mesh shall have wires with a diameter ≤ 2mm with a pitch ≤ 50 x 50mm. The nominal cover to the main reinforcement shall be ≤ 40mm, or
(b) Method B: Include in the concrete mix not less than 1.5 kg/m³ of monofilament propylene fibres. The fibres shall be 6 – 12 mm long and 18 – 32 μm in diameter, and shall have a melting point less than 180°C, or
(c) Method C: Protective layers for which it is demonstrated by local experience or fire testing that no spalling of concrete occurs under fire exposure, or
(d) Method D: A design concrete mix for which it has been demonstrated by local experience or fire testing that no spalling of concrete occurs under fire exposure.

Methods to reduce risk of concrete spalling
At least one of the following methods should be provided.
(a) Method A: A reinforcement mesh with a nominal cover of 15mm. This mesh shall have wires with a diameter ≤ 2mm with a pitch ≤ 50 x 50mm. The nominal cover to the main reinforcement shall be ≤ 40mm, or
(b) Method B: Include in the concrete mix not less than 1.5 kg/m³ of monofilament propylene fibres. The fibres shall be 6 – 12 mm long and 18 – 32 μm in diameter, and shall have a melting point less than 180°C, or
(c) Method C: Protective layers for which it is demonstrated by local experience or fire testing that no spalling of concrete occurs under fire exposure, or
(d) Method D: A design concrete mix for which it has been demonstrated by local experience or fire testing that no spalling of concrete occurs under fire exposure.

A note is added to require that post-fire investigation should include an assessment on the type and extent of remedial works that are required to restore the effectiveness of the adopted method for reducing the risk of concrete spalling.
Clause 5.2.1.2(a)

Note 1: Unless $h_{fl}$ is taken as $\leq 1/4$, the shear stress between the web and flange should be checked and provided with transverse reinforcement.

![Figure 5.2 - Effective flange width parameters](image)

For structural analysis, where a great accuracy is not required, a constant width may be assumed over the whole span. The value applicable to the span section should be adopted.

Method for designing flange reinforcements in flange beams is added.

Legend: revision/addition

(b) The longitudinal shear stress, $\tau_{s}$, at the interface between one side of a flange and the web should be taken as:

$$\tau_{s} = \frac{A_{p}}{I_{p}}$$  \hspace{1cm} (5.3a)

where,

- $A_{p}$ is the thickness of the beam flange
- $I_{p}$ is the longitudinal length of the flange beam under consideration (see Figure 5.2b) of which the maximum value may be assumed to be half the distance between the section where the moment is 0 and the section where the moment is maximum. Where point loads are applied, this length should not exceed the distance between the point loads
- $A_{p}$ is the change of compressive force in the flange over the length $A_{p}$
Method for designing flange reinforcements in flange beams is added.

Legend

Revision/addition

1.8

(Cont’d)

Clause 5.2.1.2(a)

A - compressive struts

B - longitudinal bar anchored beyond this projected point

(see Note 1(e))

Figure 5.2b - Notations for the connection between flange and web

(b) Transverse reinforcements per unit length A/u is should be determined by assuming the flange to behave as a braced framework consisting of concrete struts and ties formed by tensile reinforcement and using the following equation:

\[ 0.871 \frac{A_u}{u} = A_t \cdot \cot \theta \]  \hspace{1cm} (6.3b)

where:

- \( A_t \) is the area of flange transverse reinforcement
- \( u \) is the spacing of the flange transverse reinforcement

For the purpose of avoiding failure of the compression struts in the flange, the following condition should be satisfied:

\[ u \leq \frac{(0.68 f_{yw}) \sin \theta \cdot \cos \theta}{A_t} \]  \hspace{1cm} (6.3c)

In the absence of more rigorous calculation, the following recommended values for \( \cot \theta \) can be used:

- \( 1.0 \leq \cot \theta \leq 2.0 \) for compression flanges (45° ≤ \( \theta \) ≤ 26.5°)
- \( 2.0 \leq \cot \theta \leq 1.25 \) for tension flanges (26.5° ≤ \( \theta \) ≤ 38.0°)

(a) In case of combined shear between the flange and the web, and transverse bending, the area of steel should be the greater of that determined by Equation 5.3b or half that determined by Equation 5.3b plus that required for transverse bending.

(b) Minimum longitudinal flange reinforcement should be provided in accordance with Clause 9.3.1. Longitudinal tension reinforcement in the flange should be anchored beyond the steel required to transmit the force back to the web at the section where the reinforcement is required (see Figure 5.2b).

(c) For structural analysis, where a great accuracy is not required, a constant width may be assumed over the whole span. The value applicable to the span section should be adopted.
<table>
<thead>
<tr>
<th>Clause 6.1.3.3(g)</th>
<th>Loads on supporting beams</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>6.1.3.3(g)</strong></td>
<td><strong>Clause 6.1.3.3(g)</strong></td>
</tr>
<tr>
<td><strong>Loads on supporting beams</strong></td>
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</tr>
<tr>
<td>The design loads on beams supporting solid slabs spanning in two directions at right angles and supporting uniformly distributed loads may be assessed from the following equations:</td>
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</tr>
<tr>
<td>$v_{sx} = \beta_{sx} n_{lx}$</td>
<td>$v_{sy} = \beta_{sy} n_{ly}$</td>
</tr>
<tr>
<td>$v_{sx} = \beta_{sx} n_{lx}$</td>
<td>$v_{sy} = \beta_{sy} n_{ly}$</td>
</tr>
<tr>
<td>where:</td>
<td>where:</td>
</tr>
<tr>
<td>$v_{sx}$ is the design end shear on strips of unit width and span $l_x$ and considered to act over the middle three-quarters of the edge,</td>
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</tr>
<tr>
<td>$v_{sy}$ is the design end shear on strips of unit width and span $l_y$ and considered to act over the middle three-quarters of the edge,</td>
<td>$v_{sy}$ is the design end shear on strips of unit width and span $l_y$ and considered to act over the middle three-quarters of the edge,</td>
</tr>
<tr>
<td>$\beta_{sx}$ and $\beta_{sy}$ are the shear force coefficients shown in Table 6.7.</td>
<td>$\beta_{sx}$ and $\beta_{sy}$ are the shear force coefficients shown in Table 6.7.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Clause 6.1.3.5 – Table 6.8 – Note 1</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Clause 6.1.3.5 – Table 6.8 – Note 1</strong></td>
<td><strong>Notes</strong></td>
</tr>
<tr>
<td><strong>Notes</strong>:</td>
<td><strong>Notes</strong>:</td>
</tr>
<tr>
<td>1. $v = 0.4 \text{ for } f_{c2} \leq 40 \text{ N/mm}^2$ or $0.4 f_{c2}/40$ for $f_{c2} &gt; 40 \text{ N/mm}^2$ with the value of $f_{c2}$ not to be taken as greater than $80 \text{ N/mm}^2$</td>
<td>1. $v = 0.4 \text{ for } f_{c2} \leq 40 \text{ N/mm}^2$ or $0.4 f_{c2}/40$ for $f_{c2} &gt; 40 \text{ N/mm}^2$ with the value of $f_{c2}$ not to be taken as greater than $80 \text{ N/mm}^2$</td>
</tr>
</tbody>
</table>

| Clarification of the definition of $v_{sx}$ and $v_{sy}$ by amending the word “edge” to “supporting beam” for clarity. | A typo is rectified. |
A new clause to provide guidelines for the design of shear reinforcement in circular columns is added.

Table 6.4: Values of the coefficient \( \beta \)

<table>
<thead>
<tr>
<th>( \beta )</th>
<th>0.01</th>
<th>0.02</th>
<th>0.03</th>
<th>0.04</th>
<th>0.05</th>
<th>0.06</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Design shear stress at the column base should be checked in accordance with clause 6.2.5.

 Clause 6.2.1(e)

The design shear strength of columns may be checked in accordance with clause 6.2.5.
Figure 6.18a - Geometry of the Circular Section

(2) For Table 6.2, the term \( \psi \) should be replaced by \( \nu \), such that nominal links should be provided when \( 0.5 \psi < \nu < \psi \), and shear reinforcement should be provided when \( \psi < \nu < 0.8 \psi \) or \( \psi > 0.2 \psi \).

(3) For Table 6.3, calculation of \( \nu_c \) should be taken as half the total area of longitudinal steel and \( A_u \) should be determined as equation 6.58a;

(iv) Provision of shear reinforcement

Shear reinforcement can be either fixed with circular links (see equation 6.58b) or spiral links (see equation 6.58c). The spacing of links is in the direction of the height of the circular columns. The shear design for the cross section should be analysed from the following equations:

\[
\begin{align*}
A_{x1} & = \frac{2 \sigma_s (V - V_c)}{0.87 f_n} \quad 6.58b \\
A_{x2} & = \frac{\pi s_s (V - V_c)}{0.87 f_n (1 - 0.222 s_s / c)} \quad 6.58c
\end{align*}
\]

where:

- \( s_s \) is the spacing of circular links along the member for equation 6.58b or the plan spacing of spiral links along the member for equation 6.58c.

Note: Since each link is cut twice by the shear plane, \( A_{x2} \) is twice the cross sectional area of the link.
22 Clause 6.8.1.2 Design forces
The design forces acting on a beam-column joint shall be evaluated from the maximum internal forces in all members meeting at the joint under the most adverse load combinations at ultimate limit state as specified in table 2.1, with the joint in equilibrium.

For lateral load resisting frames where critical zones may be located at beam ends adjacent to the beam-column joint, the design forces should be calculated by taking the provided amount of longitudinal beam reinforcement in the critical zones at yield, i.e. \( f_y \). With gravity load dominated frames where reversal moments will not occur at the beam end, yielding of bottom beam reinforcement need not be considered.

Design forces
The design forces acting on a beam-column joint shall be evaluated from the maximum internal forces in all members meeting at the joint under the most adverse load combinations at ultimate limit state as specified in Table 2.1, with the joint in equilibrium.

The design forces for beam-column joint of lateral load resisting frames, where critical zones may be located at beam ends adjacent to the column, should be calculated by taking the most adverse combined net moments and forces at the joint under the load combinations at ultimate limit state as specified in Table 2.1, with the joint in equilibrium. Where reversal of beam end moment occurs, the beam-column joint design should also be calculated by taking the required amount of longitudinal beam reinforcement entering the beam-column joint at yield, i.e. \( f_y \). With gravity load dominated frames where reversal moments will not occur at the beam end, yielding of bottom beam reinforcement need not be considered.

At columns of two-way frames, where beams frame into the joint from two directions, these forces need only be considered in each direction independently.

23 Clause 6.8.1.3 – equation 6.71 Joint shear stress
The horizontal joint shear stress computed with equation 6.71 shall not exceed \( 0.2 f_y \).

\[ \nu_{\text{th}} = \frac{V_{\text{th}}}{b \cdot h_t} \] 6.71

where

- \( V_{\text{th}} \) is the total horizontal design joint shear force in the direction being considered, i.e. either \( V_{\text{th}} \) or \( V_{\text{th}}' \) as appropriate.

Joint shear stress
The horizontal joint shear stress computed with equation 6.71 shall not exceed \( 0.2 f_y \).

\[ \nu_{\text{th}} = \frac{V_{\text{th}}}{b \cdot h_t} \] 6.71

where:

- \( V_{\text{th}} \) is the total horizontal design joint shear force in the direction being considered, i.e. either \( V_{\text{th}} \) or \( V_{\text{th}}' \) as appropriate.

The magnitude of the horizontal design shear force shall be evaluated from a rational analysis taking into account the effect of all forces on the joint including the beneficial column shear forces.

24 Clause 7.2.1 – Table 7.1 The terms “Quasi-permanent load combination” and “Frequent load combination” are deleted.

Table 7.1 - Limitations of maximum estimated surface crack widths

<table>
<thead>
<tr>
<th>Exposure condition</th>
<th>Reinforced members and prestressed members with unbonded tendons</th>
<th>Prestressed members with bonded tendons</th>
</tr>
</thead>
<tbody>
<tr>
<td>1, 2, and 3</td>
<td>0.3 mm²</td>
<td>0.2 mm</td>
</tr>
<tr>
<td>4</td>
<td>0.2 mm</td>
<td></td>
</tr>
<tr>
<td>Water retaining structures²</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

Notes:
1. For exposure condition 1, crack width has no influence durability and this limit is set to guarantee acceptable appearance. In the absence of appearance this limit may be relaxed.
2. Water retaining structures referred to are water tanks and the like used in general building works and not meant to include large civil or water retaining structures.

The equation for the determination of average strain \( \varepsilon_m \) for a limiting crack width of 0.1 mm to facilitate the assessment of crack widths for structures with design crack widths limited to 0.1 mm is added.

25 Clause 7.2.3 – equation 7.2

\[ \varepsilon_m = \frac{h (h - z) (d - z)}{3 E_d A_d (d - z)} \] 7.2

\[ \varepsilon_m = \frac{h (h - z) (d - z)}{3 E_d A_d (d - z)} \] for a limiting design crack width of 0.2 mm.

\[ \varepsilon_m = \frac{h (h - z) (d - z)}{3 E_d A_d (d - z)} \] for a limiting design crack width of 0.1 mm.

26 Clause 7.3.2 Excessive response to wind loads
Excessive accelerations under wind loads that may cause discomfort or alarm to occupants should be avoided. A static or dynamic analysis could be employed taking into account the pertinent features of the structure and its surroundings. Limiting deflection at the top of a building is 5% when considering a static characteristic wind load should result in an acceptable environment for occupants in normal buildings.

Excessive response to wind loads
Excessive accelerations under wind loads that may cause discomfort or alarm to occupants should be avoided. A static or dynamic analysis could be employed taking into account the pertinent features of the structure and its surroundings. Limiting deflection at the top of a building is 5% when considering a static characteristic wind load should result in an acceptable environment for occupants in normal buildings.

The height \( H \) for determination of building deflection should be measured from the highest floor level excluding plant rooms / roof features and alike.
### Table 7.4 - Modification factor for tension reinforcement

<table>
<thead>
<tr>
<th>Clause 7.3.4.5 - Table 7.5</th>
<th>Factor</th>
<th>Clause 7.3.4.4 - Table 7.4 - Note 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Typos are rectified.</td>
</tr>
</tbody>
</table>

#### Notes:
1. The values in the table are derived from the following equation:
   \[
   f_{u}' = \frac{f_{k} \cdot A_{s, prov}}{b d} \cdot \frac{1}{f_{y}}
   \]
   where:
   - \( f_{k} \) is the design ultimate moment at the centre of the span or, for a cantilever, at the support.
   - \( A_{s, prov} \) is the area of reinforcement.

2. For a continuous beam, if the percentage of redistribution is not known but the design ultimate moment at mid-span is obviously the same as or greater than the elastic ultimate moment, the stress \( f_{u}' \) in this table may be taken as \( 2/3 f_{y} \).

<table>
<thead>
<tr>
<th>Service stress</th>
<th>0.50</th>
<th>0.75</th>
<th>1.00</th>
<th>1.50</th>
<th>2.00</th>
<th>3.00</th>
<th>4.00</th>
<th>5.00</th>
<th>6.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>2.00</td>
<td>2.00</td>
<td>2.00</td>
<td>1.85</td>
<td>1.83</td>
<td>1.86</td>
<td>1.98</td>
<td>1.99</td>
<td>1.97</td>
</tr>
<tr>
<td>150</td>
<td>2.00</td>
<td>2.00</td>
<td>1.98</td>
<td>1.60</td>
<td>1.49</td>
<td>1.25</td>
<td>1.15</td>
<td>1.09</td>
<td>1.09</td>
</tr>
<tr>
<td>167</td>
<td>2.00</td>
<td>2.00</td>
<td>1.91</td>
<td>1.63</td>
<td>1.44</td>
<td>1.21</td>
<td>1.08</td>
<td>0.99</td>
<td>0.92</td>
</tr>
<tr>
<td>200</td>
<td>2.00</td>
<td>1.95</td>
<td>1.76</td>
<td>1.51</td>
<td>1.35</td>
<td>1.14</td>
<td>1.02</td>
<td>0.94</td>
<td>0.88</td>
</tr>
<tr>
<td>250</td>
<td>1.90</td>
<td>1.70</td>
<td>1.54</td>
<td>1.30</td>
<td>1.04</td>
<td>0.84</td>
<td>0.67</td>
<td>0.65</td>
<td>0.62</td>
</tr>
<tr>
<td>300</td>
<td>1.60</td>
<td>1.44</td>
<td>1.33</td>
<td>1.10</td>
<td>1.06</td>
<td>0.93</td>
<td>0.85</td>
<td>0.80</td>
<td>0.76</td>
</tr>
<tr>
<td>( f_{k} = 500 )</td>
<td>333</td>
<td>1.41</td>
<td>1.28</td>
<td>1.18</td>
<td>1.05</td>
<td>0.96</td>
<td>0.86</td>
<td>0.79</td>
<td>0.75</td>
</tr>
</tbody>
</table>

#### Notes:
1. The values in the table are derived from the following equation:
   \[
   f_{u}' = \frac{f_{k} \cdot A_{s, prov}}{b d} \cdot \frac{1}{f_{y}}
   \]
   where:
   - \( f_{k} \) is the design ultimate moment at the centre of the span or, for a cantilever, at the support.

2. The design service stress in the tension reinforcement in a member may be estimated from the equation:
   \[
   f_{s} = \frac{2}{3} f_{k} \cdot \frac{1}{b d} \cdot \frac{1}{f_{y}}
   \]
   where:
   - \( f_{s} \) is the design service stress in the tension reinforcement in a member.

3. For a continuous beam, if the percentage of redistribution is not known but the design ultimate moment at mid-span is obviously the same as or greater than the elastic ultimate moment, the stress \( f_{s} \) in this table may be taken as \( 2/3 f_{y} \).

### Table 7.5 - Modification factor for compression reinforcement

<table>
<thead>
<tr>
<th>Clause 7.3.4.5 - Table 7.5</th>
<th>Factor</th>
<th>Clause 7.3.4.4 - Table 7.4 - Note 2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Typos are rectified.</td>
</tr>
</tbody>
</table>

#### Notes:
1. The values in this table are derived from the following equation:
   \[
   A_{c, prov} = \frac{100 \cdot A_{s, prov}}{b d} \cdot \left( 1 + \frac{100 f_{y}}{3.04} \right) \leq 1.5
   \]

2. The area of compression reinforcement \( A_{c, prov} \) used in this table may include all bars in the compression zone, even those not effectively tied with links.

<table>
<thead>
<tr>
<th>Service stress</th>
<th>100</th>
<th>150</th>
<th>200</th>
<th>250</th>
<th>300</th>
<th>350</th>
<th>400</th>
<th>450</th>
<th>500</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f_{k} = 500 )</td>
<td>100</td>
<td>150</td>
<td>200</td>
<td>250</td>
<td>300</td>
<td>350</td>
<td>400</td>
<td>450</td>
<td>500</td>
</tr>
</tbody>
</table>

#### Notes:
1. The values in this table are derived from the following equation:
   \[
   A_{c, prov} = \frac{100 A_{s, prov}}{b d} \cdot \left( 1 + \frac{100 f_{y}}{3.04} \right) \leq 1.5
   \]

2. The area of compression reinforcement \( A_{c, prov} \) used in this table may include all bars in the compression zone, even those not effectively tied with links.
Clause 7.3.6 – Table 7.7

Typos are rectified.

Table 7.7 - Values of $p_0$ for calculation of shrinkage curvatures

Clause 7.3.6 – Figure 7.2

The term “deflection” in the title is amended to “curvature” to tally with the figure.

Figure 7.2 - Loading history for serviceability limit state - deflection

Figure 7.2 - Loading history for serviceability limit state - curvature

Clause 8.4.3 – equation 8.2

A bracket is added to the denominator of equation 8.2 for clarity.

$F_0 = F_a / m_b$

where:

$F_a$ is the force in the bar or group of bars

$m_b$ is the effective bar size which, for a single bar is the bar size and for a group of bars in contact is equal to the diameter of a bar of equal total area.

$F = F_a / m_b$

where:

$F_a$ is the force in the bar or group of bars

$m_b$ is the effective bar size which, for a single bar is the bar size and for a group of bars in contact is equal to the diameter of a bar of equal total area.
### Clause 8.4.4

**Values for design ultimate anchorage bond stress**

Values for design ultimate anchorage bond stress, $f_{bu}$, may be obtained from the equation:

$$f_{bu} = \beta \sqrt{f_{cu}}$$

where $f_{cu}$ is the characteristic compressive cube strength of concrete, limited to 60 N/mm² for the purpose of calculating ultimate anchorage bond stress.

$\beta$ is the design ultimate anchorage bond stress, and $eta$ is a coefficient dependent on the bar type.

For bars in tension in slabs or in beams where minimum links have been provided in accordance with Table 6.2, the values of $\beta$ may be taken from Table 8.3. These values include a partial safety factor, $k_{ph}$, of 1.4.

<table>
<thead>
<tr>
<th>Bar type</th>
<th>Bars in tension</th>
<th>Bars in compression</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain bars</td>
<td>0.28</td>
<td>0.35</td>
</tr>
<tr>
<td>Ribbed bars</td>
<td>0.50</td>
<td>0.63</td>
</tr>
<tr>
<td>Fabric (see clause 8.4.6)</td>
<td>0.65</td>
<td>0.61</td>
</tr>
</tbody>
</table>

**Table 8.3 - Values of bond coefficient $\beta$**

In beams where minimum links in accordance with Table 6.3 have not been provided, the design anchorage bond stresses used should be those appropriate to plain bars irrespective of the type of bar used. This does not apply to slabs.

### Clause 8.4.5

**Equation 8.4**

**Minimum ultimate anchorage bond lengths**

The ultimate anchorage bond length, $l_b$, should be greater than or equal to the value calculated from:

$$l_b > \frac{V}{f_{bu}}$$

where $V$ is the design shear force.

Values for anchorage bond lengths are given in Table 8.4 as multiples of bar diameter.

### Clause 8.4.6

**Equation 8.4**

**Minimum ultimate anchorage bond lengths**

The ultimate anchorage bond length, $l_b$, should be greater than or equal to the value calculated from:

$$l_b > \frac{V}{f_{bu}}$$

where $V$ is the design shear force.

Values for anchorage bond lengths are given in Table 8.4 as multiples of bar diameter.

---

Legend: revision/addition

Typos are rectified. A bracket is added to the denominator of equation 8.4 for clarity.
### Clause 8.4.6 – Figure 8.1

The symbol “C” is amended to “c” in order to tally with the notation used in Equations 8.5 and 8.6.

![Figure 8.1 - Requirements of a bend anchorage](image1)

### Clause 8.4.8

An equation for calculating the minimum support width for anchor bars less than or equal to 12 mm in diameter is added to tally with the minimum bend radius for hooks and loops given in Table 8.2.

#### Minimum support widths

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Equation 8.5</th>
<th>Equation 8.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchored bars less than or equal to 12 mm in diameter</td>
<td>(2(a + c))</td>
<td>(2(a + c))</td>
</tr>
<tr>
<td>Anchored bars greater than 12 mm in diameter</td>
<td>(2(a + c))</td>
<td>(2(a + c))</td>
</tr>
</tbody>
</table>

Where:
- \(a\) = Anchor bar diameter
- \(c\) = Minimum bend radius

See Figure 8.1 for notation.

### Clause 8.5 – Figure 8.2

Additional details of anchorage of links for hooks with 150° & above is added. Original detail of 180° hook is covered by the new details and hence is deleted.

![Figure 8.2 - Anchorage of links](image2)
Clause 8.6 – Figure 8.3

Figure 8.3 - Welded transverse bar as anchoring device

Figure 8.3 - Welded transverse bar as anchoring device

The symbol “C” is amended to “c” in order to tally with the notation used in Equations 8.5 and 8.6.

Clause 8.7

8.7 LAPS AND MECHANICAL COUPLERS

8.7.1 General

Forces are transmitted from one bar to another by:

(a) lapping of bars, with or without bends or hooks;
(b) welding; or
(c) mechanical devices assuring load transfer in tension and/or compression.

In joints where imposed loading is predominantly cyclic, bars should not be jointed by welding.

The title “LAPS AND MECHANICAL COUPLERS” is amended to “LAPS” as the requirements for mechanical couplers are given in Clause 3.2.8.

Clause 8.7.2 & Figure 8.4

(d) the clear transverse distance between two lapping bars should not be greater than 4\(\phi\) or 50 mm, otherwise the lap length should be increased by a length equal to the clear space exceeding 40 or 50 mm.

(e) the longitudinal distance between two adjacent laps should not be less than 0.3 times the lap length, \(L_p\), and

(f) in case of adjacent laps, the close distance between adjacent bars should not be less than 2\(\phi\) or 20 mm.

The permissible percentage of lapped bars in tension at any section may be 100% where the bars are all in one layer, or 50% where the bars are in several layers.

All bars in compression and secondary (distribution) reinforcement may be lapped in one section.

The term “several layers” is amended to “2 or more layers” for clarity.

The requirements on clear transverse distance between two lapping bars and clear distance between adjacent bars in clause 8.7.2 and Figure 8.4 are clarified.

Legend: revision/addition

Clause 9.1

Walls for single storey structures are exempt from the ductility design requirement.

Clause 9.4.1(j)

Cantilevered slabs exposed to weathering should be designed for:

(i) exposure condition 2 or higher if appropriate;
(ii) the estimated maximum crack width not exceeding 0.1 mm under serviceability limit state.

The alternative checking method for exposed cantilevered slab by limiting the stress of deformed high yield steel reinforcing bars to 100N/mm² as given in Appendix A of PNAP APP-68 is incorporated.
### Clause 9.4.4
**Details and construction**

Cantilevered structures, especially those projecting over streets, should be detailed in such a manner that they may be demolished or replaced without affecting the safety and integrity of the main structure of the building.

Cantilevered structures should be cast monolithically with and at the same time as the directly supporting members. Construction joints should not be located along the external edge of the supporting members. In case this is unavoidable, the construction method should ensure that the finished product should have a structural strength and integrity not inferior to that provided by monolithic construction, and should not invite ingress of water through the joint.

 Adequate bar spacers should be provided to maintain the position and alignment of the steel reinforcing bars. Every endeavour should be made to avoid steel reinforcing bars from being displaced or depressed. Concrete works should strictly comply with requirements stipulated in clause 10.3.

Where a wall is designed to support a cantilevered slab, it should have adequate thickness to allow the proper anchorage of the main reinforcing bars of the cantilevered slab.

The additional construction requirements for external cantilevered slab with a span exceeding 750mm as stipulated in paragraph 9 of Appendix A to PNAP APP-68 are incorporated.

---

<table>
<thead>
<tr>
<th>Legend</th>
<th>revision/addition</th>
</tr>
</thead>
</table>

### Clause 9.5
**Columns**

This clause deals with columns for which the larger dimension $b_c$ is not greater than 4 times the smaller dimension $h_c$.

The symbols “$b_c$” & “$h_c$” are deleted.

### Clause 9.5.2.2
**Rectangular or polygonal columns**

All corner bars, and alternate bars (or bundle) in an outer layer of reinforcement should be supported by links, with or without crossties, passing around the bars and having an included angle of not more than 135° (see figure 9.5a). No bar within a compression zone should be further than 150 mm from a restrained bar.

Links should be adequately anchored by means of hooks bent through an angle of not less than 135° at one end and 90° at the other end, and should be alternated end for end along the longitudinal bars (see figure 9.5b). Where there is adequate confinement to prevent the end anchorage of the link from ‘kick off’ (see figure 9.5a), the 135° hook in the links or crossties may be replaced by other standard hoods given in figure 8.2.

Textual refinements and typos are rectified.

---

<table>
<thead>
<tr>
<th>Legend</th>
<th>revision/addition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clause</td>
<td>9.5.2.4 and Figure 9.5f added</td>
</tr>
<tr>
<td>--------</td>
<td>-----------------------------</td>
</tr>
</tbody>
</table>

**Figure 9.5 - Column transverse reinforcement**

The alternative details of column transverse reinforcement as stipulated in BD’s circular letter dated 29 April 2011 are incorporated.

<table>
<thead>
<tr>
<th>Clause</th>
<th>9.9.1.3 (a)(i)</th>
</tr>
</thead>
</table>

A typo is rectified.
Clause 9.9.1.3(b) and Figure 9.7

Anchorage

Links should be adequately anchored by means of 135° or 180° hooks in accordance with clause 8.5. Anchorage by means of welded cross bars is not permitted. Where there is adequate confinement to prevent the end anchorage of the link from "kick off", the 135° hook may be replaced by other standard hooks given in figure 8.2 (see figure 9.7).

Figure 9.7 – Typical confinement in beam

Clause 9.9.2.2(c)

Anchorage

Links and ties should be adequately anchored by means of 135° hooks in accordance with clause 9.5.2 (see figure 9.5c, d & e). Where there is adequate confinement to prevent the end anchorage of the link from "kick off", the 135° hook may be replaced by other standard hooks given in figure 8.2.

Textual refinements and typos are rectified.
The alternative beam shear link details as stipulated in BD’s circular letter dated 29 April 2011 are incorporated.
Clause 9.9.2.4 and Figure 9.7a added

The typical beam-column joint shear reinforcement details as stipulated in BD’s circular letter dated 29 April 2011 are incorporated.

Figure 9.7a - Beam-Column Joint Shear Reinforcement
Confined boundary elements are the edge regions or intersections of the cross-sections of walls, which are strengthened by confining reinforcement as specified in this clause.

(a) Type 1 confined boundary element
Type 1 confined boundary element refers to the shaded portions of the walls in Figure 9.11(a), and should be provided with vertical reinforcement satisfying the following requirements:

(i) not less than 0.6% of the sectional area of the structural boundary element;
(ii) not smaller than 12mm in diameter and not less than 6 in number; and
(iii) each vertical bar is tied with links or ties of at least 10mm diameter and vertical spacing not exceeding 250 mm.

(b) Type 2 confined boundary element
Type 2 confined boundary element refers to the shaded portions of the walls in Figure 9.11(a), and should be provided with vertical reinforcement satisfying the following requirements:

(i) not less than 0.8% of the sectional area of the structural boundary element;
(ii) not smaller than 16mm in diameter and not less than 6 in number; and
(iii) each vertical bar is tied with links or ties of at least 10mm diameter and vertical spacing not exceeding 200 mm.

(c) Type 3 confined boundary element
Type 3 confined boundary element refers to the shaded portions of the walls in Figure 9.11(b), and should be provided with vertical reinforcement satisfying the following requirements:

(i) not less than 1% of the sectional area of the structural boundary element;
(ii) not smaller than 16mm in diameter and not less than 6 in number; and
(iii) spacing not exceeding 150mm; and
(iv) each vertical bar is tied with links or ties of at least 12mm diameter and vertical spacing not exceeding 150 mm.

For links and ties, where there is adequate confinement to prevent the “kick off” of the hook, hooks with bend not less than 135° may be replaced by other standard hooks given in Figure 8.2.

Legend: revision/addition

The vertical bar in the confined boundary elements of walls should be tied with links or ties.
The symbols “b_c” and “h_c” are amended to “C_x” and “C_y”, respectively to tally with the symbols in clause 1.5.
When the following situation occurs, the concrete mix design, the material quality, the production method and equipment, and the procedures of concrete sampling and testing should be reviewed and no further concreting of permanent works should be allowed until a steady and satisfactory production of the mix could be restored:

(iv) For concrete grade not exceeding C60, the calculated standard deviation exceeds 8 MPa for 100mm test cubes or 8.5 MPa for 100mm test cubes; or

(v) For concrete grade exceeding C60, the coefficient of variation exceeds 14%.

In case further concreting of permanent works is not allowed when either of above conditions occurs, investigation shall be carried out to find out the cause of such variation in cube strength distribution. Measures shall be taken to restore a steady and satisfactory production of the concrete mix. However, in line with the investigation work, temporary resumption of concrete works can be allowed under any one of the following conditions:

(vi) The average of the latest 40 cube test results exceeds the grade strength by at least 10 MPa for 150 mm test cubes or 12MPa for 100 mm test cubes and individual test results exceeds the grade strength by at least 4 MPa for 150 mm test cubes or 5 MPa for 100 mm test cubes; or

(vii) The standard deviation or coefficient of variation of the latest 40 cube test results is found to fall below the corresponding limit again with new cube test results coming up after the incident is identified showing that the variation in cube distribution has become normal again.

Permanent resumption of concreting works is allowed when either the case is confirmed to be caused by individual cube test results deviating from the general trend of other data or the remedial actions corresponding to the identified root causes are conducted.

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The conditions for temporary resumption of concreting works are added.

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The coring test requirements and corresponding acceptance criteria for further testing required under regulation 63 of the Building (Construction) Regulations are added.

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Consequential amendments (see remarks for Clause 4.2.1 in item 7 above).

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A typo is rectified.
<table>
<thead>
<tr>
<th>Clause</th>
<th>Section</th>
<th>Content</th>
</tr>
</thead>
<tbody>
<tr>
<td>12.12.3.1</td>
<td>(b)</td>
<td><strong>Cover against corrosion</strong>&lt;br&gt;The exposure conditions for the structural element should be assessed in accordance with 4.2.4.3 and the required nominal cover, grade and associated mix limitations obtained from Table 4.2. The recommendations of clause 4.2 for concrete materials and mixes also apply to Table 4.2 except that the minimum cement content should not be reduced below 300 kg/m³.</td>
</tr>
<tr>
<td></td>
<td>(b)</td>
<td><strong>Cover against corrosion</strong>&lt;br&gt;The exposure conditions for the structural element should be assessed in accordance with clause 4.2.4.3 and the required nominal cover, grade and associated mix limitations obtained from Table 4.2. The recommendations of clause 4.2 for concrete materials and mixes also apply to Table 4.2 except that the minimum cement content should not be reduced below 300 kg/m³.</td>
</tr>
</tbody>
</table>

**Consequential amendments** (see remarks for Clause 4.2.1 in item 7 above) and textural refinements.

<table>
<thead>
<tr>
<th>Clause</th>
<th>Section</th>
<th>Content</th>
</tr>
</thead>
<tbody>
<tr>
<td>13.2</td>
<td></td>
<td><strong>TEST LOADS</strong>&lt;br&gt;The total load to be carried (W) should be not less than 1.0 times the characteristic dead load plus 1.0 times the characteristic live load, and should normally be the greater of (a) the sum of the characteristic dead load and 1.20 times the characteristic imposed load or (b) 1.125 times the sum of the characteristic dead and imposed loads. In deciding on suitable figures for this, and on how to apply the test load to the structure, due allowance should be made for finishes, paritions, etc and for any load sharing that could occur in the completed structure, i.e. the level of loading should be representative and capable of reproducing the proper internal force system reasonably closely.</td>
</tr>
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<td></td>
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<td><strong>TEST LOADS</strong>&lt;br&gt;The total load to be carried (W) should be not less than 1.0 times the characteristic dead load plus 1.0 times the characteristic imposed load, and should normally be the greater of (a) the sum of the characteristic dead load and 1.20 times the characteristic imposed load or (b) 1.125 times the sum of the characteristic dead and imposed loads. In deciding on suitable figures for this, and on how to apply the test load to the structure, due allowance should be made for finishes, paritions, etc and for any load sharing that could occur in the completed structure, i.e. the level of loading should be representative and capable of reproducing the proper internal force system reasonably closely.</td>
</tr>
</tbody>
</table>

The term “characteristic live load” is amended to “characteristic imposed load”.

| Annex A | | **The list of acceptable standards** is updated to include the testing standards for post-tensioning systems. |

**Legend:** revision/addition