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

Item	Clause/ Annex	Current Version	Amendments	Remarks
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 blastfurnace 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_{pb} design tensile stress in the tendons f_{pe} design effective prestress in the tendons after all losses f_{pu} characteristic strength of a prestressing tendon f_s estimated design service stress in the tension reinforcement f_y characteristic yield strength of reinforcement f_{yv} characteristic yield strength of the shear reinforcement G_k characteristic dead load h depth of cross section measured in the plane under consideration, or thickness of wall h_{agg} maximum size of coarse aggregate h_f thickness of a beam flange l effective span of a beam or slab l_b 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 Q_k characteristic imposed load R_m tensile strength of reinforcement	f_{cu} characteristic compressive strength of concrete f_{pb} design tensile stress in the tendons f_{pe} design effective prestress in the tendons after all losses f_{pu} characteristic strength of a prestressing tendon f_s estimated design service stress in the tension reinforcement f_y specified characteristic yield strength f_{yv} characteristic yield strength of the shear reinforcement G_k characteristic dead load h depth of cross section measured in the plane under consideration, or thickness of wall h_{agg} maximum size of coarse aggregate h_f thickness of a beam flange l effective span of a beam or slab l_b 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 Q_k characteristic imposed load R_m tensile strength	Definition of symbols f_y and Rm is unified and amended to "specified characteristic yield strength" and "tensile strength" respectively.

Legend: revision/addition

<table>
<tr>
<td>3
(Cont'd)</td>
<td>Clause 1.5
(Cont'd)</td>
<td>s_b spacing of bent-up bars
s_v spacing of links along the member
V design ultimate shear force
V_b design shear resistance of bent-up bars
W_k characteristic wind load
x depth to the neutral axis of a concrete section
z lever arm
γ_f partial safety factor for load
γ_m partial safety factor for strength of materials
v design shear stress at a section
v_c design ultimate resistance shear stress of the concrete
ϕ diameter of reinforcing bar or prestressing duct
ϕ_n equivalent diameter of a bundle of reinforcing bar</td>
<td>s_b spacing of bent-up bars
s_f spacing of the flange transverse reinforcement
s_v spacing of links along the member
V design ultimate shear force
V_b design shear resistance of bent-up bars
W_k characteristic wind load
x depth to the neutral axis of a concrete section
z lever arm
γ_f partial safety factor for load
γ_m partial safety factor for strength of materials
v design shear stress at a section
v_c design ultimate resistance shear stress of the concrete
v_{sf} longitudinal shear stress at the interface between one side of a flange and the web
v_{sx} design end shear on strips of unit width and span l_x
v_{sy} design end shear on strips of unit width and span l_y
β_{vx}, β_{vy} shear force coefficients
ϕ diameter of reinforcing bar or prestressing duct
ϕ_n equivalent diameter of a bundle of reinforcing bar
Δx longitudinal length of the flange beam (see Figure 5.2b)
ΔF_d change of compressive force in the flange (see Figure 5.2b)</td>
<td>Definition of symbols s_f, v_{sf}, v_{sx}, v_{sy}, β_{vx}, β_{vy}, Δx, and ΔF_d are added.</td>
</tr>
<tr>
<td>4</td>
<td>Clause 2.2.3.2</td>
<td>Check of structural integrity
The structural integrity of the building and its members should be checked for the effects of the design fire. In the checking, the strength of concrete and reinforcement should be based on the values given in clause 3.6, and the partial safety factors for loads and materials should be based on the values given in clauses 2.3.2.7 and 2.4.3.2 respectively.</td>
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Note : Fire limit state is required to be checked if the cover of concrete members does not comply with the provisions of the Code of Practice for Fire Safety in Buildings or the design strength of concrete is greater than 60 MPa.</td>
<td>A note to specify the circumstances where fire limit state checking is added.</td>
</tr>
<tr>
<td>5</td>
<td>Clause 3.2.3 & Table 3.3</td>
<td>Strength classes
The specified characteristic strengths are given in table 3.3.
<table>
<tr><th>Grade</th><th>Specified characteristic strength, f_y (N/mm^2)</th></tr>
<tr><td>250</td><td>250</td></tr>
<tr><td>500B
500C</td><td>500</td></tr>
</table>
Table 3.3 - Strength of reinforcement</td>
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<tr><td>500B
500C</td><td>500</td></tr>
</table>
Table 3.3 - Strength of reinforcement</td>
<td>Definition of symbol f_y is unified and amended to "specified characteristic yield strength".</td>
</tr>
</table>

6	Clause 3.2.8.4	*Performance of type 2 mechanical couplers* Type 2 mechanical coupler should satisfy the following criteria: (a) The splicing assemblies shall be tested to establish that they comply with the requirements given in clause 3.2.8.3. (b) Static tension test: The splicing assemblies must develop in tension the greater of 100 percent of the specified tensile strength, R_m, of the bar, and 125 percent of the specified yield strength, f_y, of the bar (c) Static compression test: The splicing assemblies must develop in compression 125 percent of the specified yield strength, f_y, of the bar. (d) Cyclic tension-and-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. If the conditions of acceptance for the static tension test are complied with in Stage 4, the static tension test may be omitted.	Performance of type 2 mechanical couplers Type 2 mechanical coupler should satisfy the following criteria: (a) The splicing assemblies shall be tested to establish that they comply with the requirements given in clause 3.2.8.3. (b) Static tension test: The splicing assemblies must develop in tension the greater of 100 percent of the tensile strength, R_m, of the bar, and 125 percent of the specified characteristic yield strength, f_y, of the bar (c) Static compression test: The splicing assemblies must develop in compression 125 percent of the specified characteristic yield strength, f_y, of the bar. (d) Cyclic tension-and-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. If the conditions of acceptance for the static tension test are complied with in Stage 4, the static tension test may be omitted.	Definition of symbols f_y and Rm is unified and amended to “specified characteristic yield strength” and “tensile strength” respectively.
7	Clause 4.2.1	**General** One of the main characteristics influencing the durability of concrete is its permeability to the ingress of water, oxygen, carbon dioxide and other potentially deleterious substances. Permeability is governed by the constituents and procedures used in making the concrete. With normal-weight aggregates a suitably low permeability is achieved by having an adequate cement content, a sufficiently low free water/cement ratio, complete compaction of the concrete, and sufficient hydration of the cement through proper curing.	**General** One of the main characteristics influencing the durability of concrete is its permeability to the ingress of water, oxygen, carbon dioxide and other potentially deleterious substances. Permeability is governed by the constituents and procedures used in making the concrete. With normal-weight aggregates a suitably low permeability is achieved by having an adequate cementitious content, a sufficiently low free water/cement ratio, complete compaction of the concrete, and sufficient hydration of the cement through proper curing.	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.
8	Clause 4.2.1	(c) the environment (clause 4.2.3); (d) the type of cement (clauses 4.2.5 and 4.2.7); (e) the type of aggregate (clauses 4.2.5 and 4.2.7); (f) the cement content and water/cement ratio of the concrete (clause 4.2.6); and (g) workmanship, to obtain full compaction and efficient curing (clauses 10.3.5 and 10.3.6).	(c) the environment (clause 4.2.3); (d) the type of cementitious material(s) (clauses 4.2.5 and 4.2.7); (e) the type of aggregate (clauses 4.2.5 and 4.2.7); (f) the cementitious content and water/ cement ratio of the concrete (clause 4.2.6); and (g) workmanship, to obtain full compaction and efficient curing (clauses 10.3.5 and 10.3.6).	Consequential amendments (see remarks for Clause 4.2.1 in item 7 above).
9	Clause 4.2.2.1	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/m^3 or more, measures to reduce the temperature such as using material with a slower release of heat of hydration should be considered.	Where the minimum dimension of the concrete to be placed in one continuous operation is greater than 600 mm, and especially where the cementitious content is 400 kg/m^3 or more, measures to reduce the temperature such as using material with a slower release of heat of hydration should be considered.	Consequential amendments (see remarks for Clause 4.2.1 in item 7 above).

10 — Clause 4.2.4.4 - Table 4.2

Conditions of exposure (see clause 4.2.3)	Nominal cover (mm)						
Lowest grade of concrete	C20/25	C30	C35	C40	C45	C50	≥C55
Condition 1							
- slabs only	30	30	25	25	25	25	25
- other members	35	30	30	30	25	25	25
Condition 2	--	40	35	35	30	30	30
Condition 3	--	--	--	50	45	45	45
Condition 4	--	--	--	--	--	55	50
Condition 5 (see note 3)	--	--	--	--	--	--	--
Maximum free water/cement ratio	0.65	0.65	0.60	0.55	0.45	0.40	0.35
Minimum cement content (kg/m^3)	290	290	290	300	340	380	380

Notes:

1. This table relates to normal-weight aggregate 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) and the safe transmission of bond forces (see clause 8.7).
4. For prestressed concrete, grade C30 or lower should not be used and the minimum cement content should be 300 kg/m^3.

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

Conditions of exposure (see clause 4.2.3)	Nominal cover (mm)						
Lowest grade of concrete	C20/25	C30	C35	C40	C45	C50	≥C55
Condition 1							
- slabs only	30	30	25	25	25	25	25
- other members	35	30	30	30	25	25	25
Condition 2	--	40	35	35	30	30	30
Condition 3	--	--	--	50	45	45	45
Condition 4	--	--	--	--	--	55	50
Condition 5 (see note 3)	--	--	--	--	--	--	--
Maximum free water/cement ratio	0.65	0.65	0.60	0.55	0.45	0.40	0.35
Minimum cementitious content (kg/m^3)	290	310	330	350	375	400	400

Notes:

1. This table relates to normal-weight aggregate 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.
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) and the safe transmission of bond forces (see clause 8.7).
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Table 4.2 - Nominal cover to all reinforcement (including links) and minimum concrete grade to meet durability requirements for reinforced and prestressed concrete

- Values are adjusted as a result of including silica fume and pfa or ggbs in the “cementitious content”.
- Consequential amendments (see remarks for Clause 4.2.1 in item 7 above).

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11 — Clause 4.2.5.1 & Clause 4.2.5.2

4.2.5.1 Mix proportions

Table 4.2 gives maximum free water/cement ratios and minimum cement contents for concrete appropriate for use in given environments with specified covers for both reinforced and prestressed concrete. The minimum grades will generally ensure that the limits on free water/cement ratio and cement content will be met without further checking. These limits relate to concrete made using 20 mm nominal maximum sized normal-weight aggregates.

4.2.5.2 Permitted reduction in concrete grade

Where due to the nature of the constituent materials there is difficulty in complying with the concrete grades in table 4.2, the further checking not required in clause 4.2.5.1 becomes necessary to ensure compliance with the limits on the free water/cement ratio and cement content. Provided a systematic checking regime is established to ensure compliance with these limits in the concrete as placed, the concrete grades specified may be relaxed by not more than 5. This relaxation should not be applied to the mixes permitted in clause 4.2.5.5.

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

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12 — Clause 4.2.5.3

Permitted reduction in cement content

Where concrete with free water/cement ratios significantly lower than the maximum values in table 4.2, which are appropriate for nominal workability, is both manufactured and used under specially tightly controlled conditions, the cement content may be reduced provided the following requirements are met:

(a) the reduction in cement content does not exceed 10% of the appropriate value in table 4.2;
(b) the corresponding free water/cement ratio is reduced by not less than the percentage reduction in the cement content;
(c) the resulting mix can be placed and compacted properly; and
(d) systematic controls are established to ensure that the reduced limits are met in the concrete as placed.

Permitted reduction in cementitious content

Where concrete with free water/cement ratios significantly lower than the maximum values in Table 4.2, which are appropriate for nominal workability, is both manufactured and used under specially tightly controlled conditions, the cementitious content may be reduced provided the following requirements are met:

(a) the reduction in cementitious content does not exceed 10% of the appropriate value in Table 4.2;
(b) the corresponding free water/cement ratio is reduced by not less than the percentage reduction in the cementitious content;
(c) the resulting mix can be placed and compacted properly; and
(d) systematic controls are established to ensure that the reduced limits are met in the concrete as placed.

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

<table>
<tr>
<td>13</td>
<td>Clause 4.2.5.4 & Table 4.3</td>
<td>

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 they should be modified as given in table 4.3 subject to the condition that the cement content should be not less than 240 kg/m³ for the exposure conditions covered by table 4.2.

<table>
<tr><th>Nominal maximum aggregate size (mm)</th><th>Adjustment to minimum cement contents (kg/m³)</th></tr>
<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>
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Table 4.3 - Adjustments to minimum cement contents for aggregates other than 20 mm nominal maximum size

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<td>Consequential amendments (see remarks for Clause 4.2.1 in item 7 above).</td>
</tr>
<tr>
<td>14</td>
<td>Clause 4.2.6.1</td>
<td>

General

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 pfa or ggbs, and/or the use of plasticisers or water-reducing admixtures should be considered.

For normal strength concrete, i.e. $f_{cu} \leq 60$ N/mm², a total cementitious content including cement and pfa or ggbs in excess of 550 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 ($f_{cu} > 60$ 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 450 kg/m³.

For concrete made with normal-weight aggregate and used in foundations to low rises structures in non-aggressive soil conditions, a minimum grade of C20 may be used provided the minimum cement content is not less than 290 kg/m³.

For high strength concrete, reference should also be made to requirements in clause 4.3.

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<table>
<tr>
<td>15</td>
<td>Clause 4.2.6.2 & Table 4.4</td>
<td>

Unreinforced concrete

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>
<tr><th rowspan="2">Condition of exposure
(see clause 4.2.3.2)</th><th colspan="3">Concrete not containing embedded metal</th></tr>
<tr><th>Maximum free water/cement ratio</th><th>Minimum cement content
(kg/m^3)</th><th>Lowest grade of concrete</th></tr>
<tr><td>1</td><td>0.65</td><td>290</td><td>C20</td></tr>
<tr><td>2</td><td>0.65</td><td>290</td><td>C30</td></tr>
<tr><td>3</td><td>0.55</td><td>325</td><td>C35</td></tr>
<tr><td>4</td><td>0.50</td><td>350</td><td>C45</td></tr>
<tr><td>5</td><td>0.50</td><td>350</td><td>C50</td></tr>
<tr><td colspan="4">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.</td></tr>
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Table 4.4 - Durability of unreinforced concrete made with normal-weight aggregates of 20 mm nominal maximum size

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Unreinforced concrete

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<td>Consequential amendments (see remarks for Clause 4.2.1 in item 7 above).</td>
</tr>
<tr>
<td>16</td>
<td>Clause 4.2.6.3</td>
<td>

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).

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</tr>
<tr>
<td>17</td>
<td>Clause 4.3.1.2(b)</td>
<td>

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/m^3 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.

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(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/m^3 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.

Note : 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.

</td>
<td>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.</td>
</tr>
</table>

<table>
<tr>
<td>18</td>
<td>Clause 5.2.1.2(a)</td>
<td>Note 1: Unless $b_{eff,i}$ is taken as $\leq 0.1 l_{pi}$, the shear stress between the web and flange should be checked and provided with transverse reinforcement.

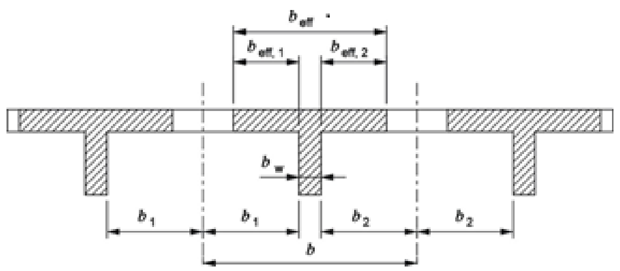


Figure 5.2 - Effective flange width parameters

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.</td>
<td>Note 1:
(a) Unless $b_{eff,i}$ is taken as $\leq 0.1 l_{pi}$, the shear stress between the web and flange should be checked and provided with transverse reinforcement.

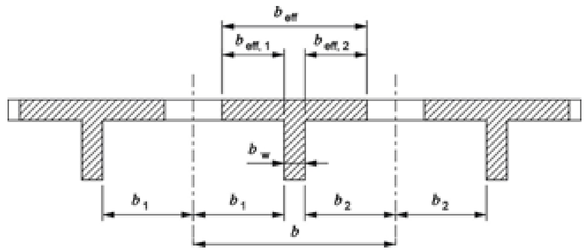


Figure 5.2a - Effective flange width parameters

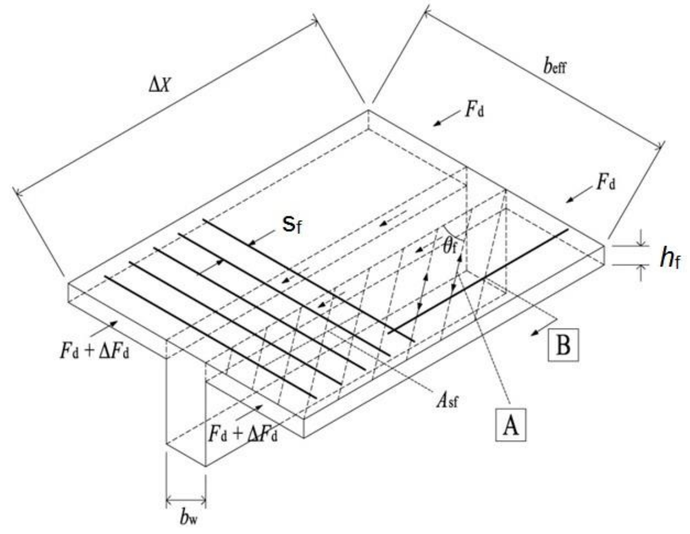
(b) The longitudinal shear stress, v_{sf}, at the interface between one side of a flange and the web, should be taken as:

$v_{sf} = \Delta F_d/(h_f \Delta x)$ (5.3a)

where:

h_f is the thickness of the beam flange
Δx 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
ΔF_d is the change of compressive force in the flange over the length Δx</td>
<td>Method for designing flange reinforcements in flange beams is added.</td>
</tr>
</table>

Legend: revision/addition

18 (Cont'd)	Clause 5.2.1.2(a) (Cont'd)		 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** (c) Transverse reinforcements per unit length A_{sf}/s_f should be determined by assuming the flange to behave as a braced framework consisting of concrete struts and ties formed by tensile reinforcements and using the following equation: $0.87f_yA_{sf}/s_f \geq v_{sf}h_f/\cot\theta_f$ (5.3b) where: A_{sf} is the area of flange transverse reinforcement s_f 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: $v_{sf} \leq (0.68f_{cu}/\gamma_m)\sin\theta_f\cos\theta_f$ (5.3c) In the absence of more rigorous calculation, the following recommended values for cot θ_f can be used: $1.0 \leq \cot\theta_f \leq 2.0$ for compression flanges ($45^o \geq \theta_f \geq 26.5^o$) $2.0 \leq \cot\theta_f \leq 1.25$ for tension flanges ($45^o \geq \theta_f \geq 38.6^o$) (d) 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. (e) 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 strut required to transmit the force back to the web at the section where this reinforcement is required (see Figure 5.2b). (f) 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

19	Clause 6.1.3.3(g)	Loads on supporting beams 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: $v_{sy} = \beta_{vy} n l_x$ 6.33 $v_{sx} = \beta_{vx} n l_x$ 6.34 where: 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, 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, β_{vx} and β_{vy} are the shear force coefficients shown in table 6.7.	Loads on supporting beams 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: $v_{sy} = \beta_{vy} n l_x$ 6.33 $v_{sx} = \beta_{vx} n l_x$ 6.34 where: 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 supporting beam, 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 supporting beam, β_{vx} and β_{vy} are the shear force coefficients shown in Table 6.7.	Clarification of the definition of v_{sx} and v_{sy} by amending the word “edge” to “supporting beam” for clarity.
20	Clause 6.1.3.5 – Table 6.8 – Note 1	Notes: 1. v_r = 0.4 for f_{cu} ≤ 40 N/mm² or 0.4(f_{cu} /40)2/3 for f_{cu} > 40 N/mm² with the value of f_{cu} not to be taken as greater than 80 N/mm²	Notes: 1. v_r = 0.4 for f_{cu} ≤ 40 N/mm² or $0.4(f_{cu}/40)^{2/3}$ for f_{cu} > 40 N/mm² with the value of f_{cu} not to be taken as greater than 80 N/mm²	A typo is rectified.

21

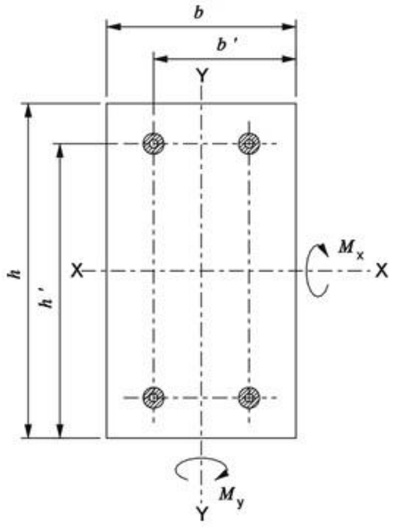
Clause 6.2.1.4 (e)

(e) Shear in columns

The design shear strength of columns may be checked in accordance with clause 6.1.2.5(k). For rectangular sections in compression no check is required provided that M/N does not exceed $0.6h$ and v does not exceed the maximum value given in clause 6.1.2.5(k).

$N/(bhf_{cu})$	0	0.1	0.2	0.3	0.4	0.5	≥0.6
β	1.00	0.88	0.77	0.65	0.53	0.42	0.30

Table 6.14 - Values of the coefficient β



(e) Shear in columns

(i) Design concrete shear stress

The design shear strength of columns should be checked in accordance with clause 6.1.2.5(k).

(ii) Design shear resistance of rectangular column

For rectangular sections in compression, no checking is required provided that M/N does not exceed 0.6h and v does not exceed the maximum value given in clause 6.1.2.5(k). Otherwise, shear resistance of rectangular column should be checked in accordance with clause 6.1.2.5.

$N/(bhf_{cu})$	0	0.1	0.2	0.3	0.4	0.5	≥0.6
β	1.00	0.88	0.77	0.65	0.53	0.42	0.30

Table 6.14 - Values of the coefficient β

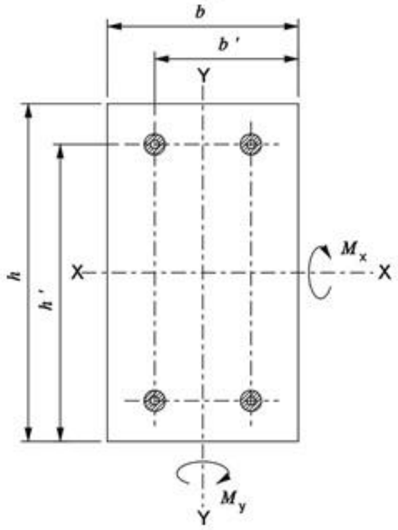


Figure 6.18 - Biaxially bent columns

(iii) Design shear resistance of circular column

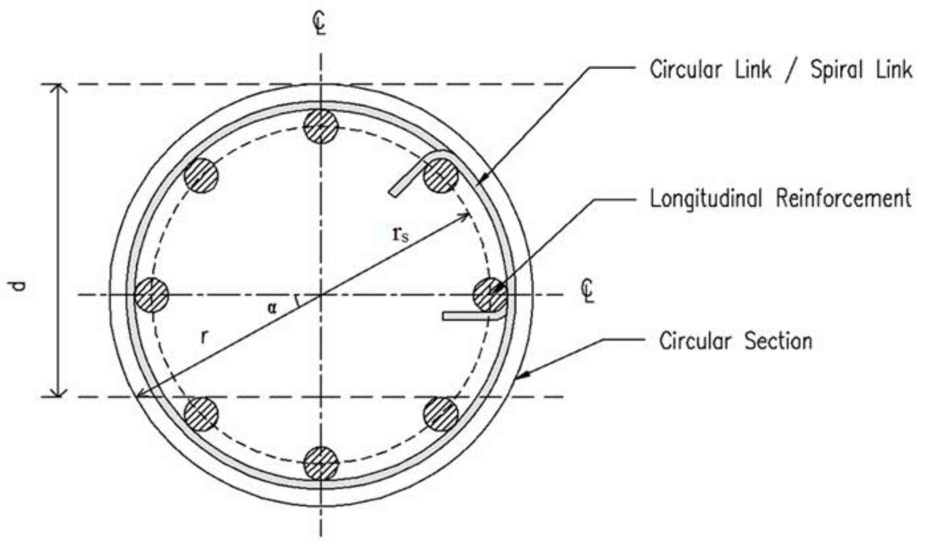
Shear resistance of circular column should be checked in accordance with clause 6.1.2.5 with the following definitions.

(1) For equation 6.19, $b_v d = r^2 (\pi/2 + \alpha + \sin\alpha \cos\alpha)$ 6.58a

where:

- r is the radius of the circular section
- r_s is the radius of the circumference along the centre line of the longitudinal reinforcement (see Figure 6.18a)
- $d = r(1+\sin\alpha)$ is the effective depth taken as the depth to the centroid of the reinforcement below the centre line of the cross section of the circular column (see Figure 6.18a)
- $\sin\alpha = 2r_s / (\pi r)$ $(0 < \alpha < \pi/2)$

A new clause to provide guidelines for the design of shear reinforcement in circular column is added.

21 (Cont'd)	Clause 6.2.1.4 (e) (Cont'd)		 Figure 6.18a – Geometry of the Circular Section (2) For Table 6.2, the term (v_c + v_r) should be replaced by v_c such that nominal links should be provided when $0.5v_c < v < v_c$ and shear reinforcement should be provided when $v_c < v < 0.8\sqrt{f_{cu}}$ or 7.0 N/mm²; (3) For Table 6.3, calculation of v_c , A_s should be taken as half the total area of longitudinal steel and $b_v d$ 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: $A_{sv} \geq \dfrac{2r\,s_v(v - v_c)}{0.87 f_{yv}}$ 6.58b or $A_{sv} \geq \dfrac{r\,s_v(v - v_c)}{0.87 f_{yv}(1 - 0.225 s_v / r)}$ 6.58c where: s_v is the spacing of circular links along the member for equation 6.58b or the pitch spacing of spiral links along the member for equation 6.58c Note: Since each link is cut twice by the shear plane, A_{sv} is twice the cross sectional area of the link.	A new clause to provide guidelines for the design of shear reinforcement in circular column is added.

<table>
<tr><td>22</td><td>Clause 6.8.1.2</td><td>

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.

</td><td>

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.

</td><td>The design forces for beam-column joint of lateral load resisting frame should be calculated under the most adverse load combinations at ultimate limit state as specified in table 2.1. The beam-column joint design should be calculated by taking the "required" amount of longitudinal beam reinforcement instead of the "provided" amount of longitudinal beam reinforcement.</td></tr>
<tr><td>23</td><td>Clause 6.8.1.3 – equation 6.71</td><td>

Joint shear stress

The horizontal joint shear stress computed with equation 6.71 shall not exceed $0.2 f_{cu}$.

$$v_{jh} = \frac{V_{jh}}{b_j h_c} \qquad 6.71$$

where:

V_{jh} is the total horizontal design joint shear force in the direction being considered, i.e. either V_{jx} or V_{jy} as appropriate

</td><td>

Joint shear stress

The horizontal joint shear stress computed with equation 6.71 shall not exceed $0.2 f_{cu}$.

$$v_{jh} = \frac{V_{jh}}{b_j h_c} \qquad 6.71$$

where:

V_{jh} is the total horizontal design joint shear force in the direction being considered, i.e. either V_{jx} or V_{jy} 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.

</td><td>The effect of all forces on the beam-column joints including beneficial column shear forces should be considered in deriving the total horizontal design joint shear force V_{jh}.
.</td></tr>
<tr><td>24</td><td>Clause 7.2.1 – Table 7.1</td><td>

Exposure condition	Reinforced members and prestressed members with unbonded tendons	Prestressed members with bonded tendons
	Quasi-permanent load combination	Frequent load combination
1, 2, and 3	0.3 mm[1]	0.2 mm
4	0.3 mm	0.2 mm
Water retaining structures[2]	0.2 mm	-

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

Table 7.1 - Limitations of maximum estimated surface crack widths

</td><td>

Exposure condition	Reinforced members and prestressed members with unbonded tendons	Prestressed members with bonded tendons ....
1, 2, and 3	0.3 mm[1]	0.2 mm
4	0.3 mm	0.2 mm
Water retaining structures[2]	0.2 mm	-

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

Table 7.1 - Limitations of maximum estimated surface crack widths

</td><td>The terms "Quasi-permanent load combination" and "Frequent load combination" are deleted.</td></tr>
<tr><td>25</td><td>Clause – 7.2.3 - equation 7.2</td><td>

$$\varepsilon_m = \varepsilon_1 - \frac{b_t(h-x)(a'-x)}{3E_s A_s(d-x)} \qquad 7.2$$

</td><td>

$$\varepsilon_m = \varepsilon_1 - \frac{b_t(h-x)(a'-x)}{3E_s A_s(d-x)}$$ for a limiting design surface crack width of 0.2mm ; 7.2

$$\varepsilon_m = \varepsilon_1 - \frac{1.5b_t(h-x)(a'-x)}{3E_s A_s(d-x)}$$ for a limiting design surface crack width of 0.1mm 7.2(a)

</td><td>The equation for the determination of average strain ε_m for a limiting crack width of 0.1mm to facilitate the assessment of crack widths for structures with design crack widths limited to 0.1mm is added.</td></tr>
<tr><td>26</td><td>Clause 7.3.2</td><td>

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 to $H/500$ when considering a static characteristic wind load should result in an acceptable environment for occupants in normal buildings.

</td><td>

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 to $H/500$, where H should be measured from the highest floor level excluding plant rooms/roof features and alike, when considering a static characteristic wind load should result in an acceptable environment for occupants in normal buildings.

</td><td>The height H for determination of building deflection should be measured from the highest floor level excluding plant rooms / roof features and alike.</td></tr>
</table>

Legend: revision/addition

27

Clause 7.3.4.4 – Table 7.4 – Note 2

Service stress		M/bd^2								
		0.50	0.75	1.00	1.50	2.00	3.00	4.00	5.00	6.00
	100	2.00	2.00	2.00	1.86	1.63	1.36	1.19	1.08	1.01
	150	2.00	2.00	1.98	1.69	1.49	1.25	1.11	1.01	0.94
(f_y = 250)	167	2.00	2.00	1.91	1.63	1.44	1.21	1.08	0.99	0.92
	200	2.00	1.95	1.76	1.51	1.35	1.14	1.02	0.94	0.88
	250	1.90	1.70	1.55	1.34	1.20	1.04	0.94	0.87	0.82
	300	1.60	1.44	1.33	1.16	1.06	0.93	0.85	0.80	0.76
(f_y =500)	333	1.41	1.28	1.18	1.05	0.96	0.86	0.79	0.75	0.72

Notes:

1. The values in the table are derived from the following equation:

   Modification factor = $0.55 + \frac{(477 - f_s)}{120\left(0.9 + \frac{M}{bd^2}\right)} \le 2.0$

   where:

   M 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:

   $f_s = \frac{2 f_y A_{st,req}}{3 A_{st,prov}} \times \frac{1}{\beta_b}$ - see clause 6.1.2.4 (b) for definition of β_b

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 7.4 - Modification factor for tension reinforcement

Service stress		M/bd^2								
		0.50	0.75	1.00	1.50	2.00	3.00	4.00	5.00	6.00
	100	2.00	2.00	2.00	1.86	1.63	1.36	1.19	1.08	1.01
	150	2.00	2.00	1.98	1.69	1.49	1.25	1.11	1.01	0.94
(f_y = 250)	167	2.00	2.00	1.91	1.63	1.44	1.21	1.08	0.99	0.92
	200	2.00	1.95	1.76	1.51	1.35	1.14	1.02	0.94	0.88
	250	1.90	1.70	1.55	1.34	1.20	1.04	0.94	0.87	0.82
	300	1.60	1.44	1.33	1.16	1.06	0.93	0.85	0.80	0.76
(f_y =500)	333	1.41	1.28	1.18	1.05	0.96	0.86	0.79	0.75	0.72

Notes:

1. The values in the table are derived from the following equation:

   Modification factor = $0.55 + \frac{(477 - f_s)}{120\left(0.9 + \frac{M}{bd^2}\right)} \le 2.0$

   where:

   M 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:

   $f_s = \frac{2 f_y A_{s,req}}{3 A_{s,prov}} \times \frac{1}{\beta_b}$ - see clause 6.1.2.4 (b) for definition of β_b

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 7.4 - Modification factor for tension reinforcement

Typos are rectified.

28

Clause 7.3.4.5 – Table 7.5

$100\frac{A'_{s,prov}}{bd}$	Factor
0.00	1.00
0.15	1.05
0.25	1.08
0.35	1.10
0.50	1.14
0.75	1.20
1.00	1.25
1.50	1.33
2.00	1.40
2.50	1.45
≥ 3.00	1.50

Notes:

1. The values in this table are derived from the following equation

   Modification factor for compression reinforcement = $1 + \frac{100A'_{s,prov}}{bd} \Big/ \left(3 + \frac{100A'_{s,prov}}{bd}\right) \le 1.5$

2. The area of compression reinforcement $A'_{s,prov}$ used in this table may include all bars in the compression zone, even those not effectively tied with links.

Table 7.5 - Modification factor for compression reinforcement

$100\frac{A_s'{}_{,prov}}{bd}$	Factor
0.00	1.00
0.15	1.05
0.25	1.08
0.35	1.10
0.50	1.14
0.75	1.20
1.00	1.25
1.50	1.33
2.00	1.40
2.50	1.45
≥ 3.00	1.50

Notes:

1. The values in this table are derived from the following equation

   Modification factor for compression reinforcement = $1 + \frac{100A_s'{}_{,prov}}{bd} \Big/ \left(3 + \frac{100A_s'{}_{,prov}}{bd}\right) \le 1.5$

2. The area of compression reinforcement $A_s'{}_{,prov}$ used in this table may include all bars in the compression zone, even those not effectively tied with links.

Table 7.5 - Modification factor for compression reinforcement

Typos are rectified.

29 Clause 7.3.6 – Table 7.7

$100\frac{A_{st}}{bd}$	$100\frac{A'_s}{bd}$								
	0.00	0.25	0.50	0.75	1.00	1.25	1.50	1.75	2.00
0.25	0.44	0.31	0.26	0.22	0.20	0.18	0.17	0.16	0.15
0.50	0.56	0.31	0.26	0.22	0.20	0.18	0.17	0.16	0.15
0.75	0.64	0.45	0.26	0.22	0.20	0.18	0.17	0.16	0.15
1.00	0.70	0.55	0.39	0.22	0.20	0.18	0.17	0.16	0.15
1.50	0.80	0.69	0.57	0.45	0.32	0.18	0.17	0.16	0.15
2.00	0.88	0.79	0.69	0.60	0.49	0.39	0.28	0.16	0.15
2.50	0.95	0.87	0.79	0.70	0.62	0.53	0.44	0.35	0.25
3.00	1.00	0.94	0.86	0.79	0.72	0.64	0.57	0.49	0.40
3.50	1.00	1.00	0.93	0.87	0.8	0.74	0.67	0.60	0.52
4.00	1.00	1.00	1.00	0.93	0.87	0.81	0.75	0.69	0.62

Table 7.7 - Values of ρ_o for calculation of shrinkage curvatures

$100\frac{A_s}{bd}$	$100\frac{A_s'}{bd}$								
	0.00	0.25	0.50	0.75	1.00	1.25	1.50	1.75	2.00
0.25	0.44	0.31	0.26	0.22	0.20	0.18	0.17	0.16	0.15
0.50	0.56	0.31	0.26	0.22	0.20	0.18	0.17	0.16	0.15
0.75	0.64	0.45	0.26	0.22	0.20	0.18	0.17	0.16	0.15
1.00	0.70	0.55	0.39	0.22	0.20	0.18	0.17	0.16	0.15
1.50	0.80	0.69	0.57	0.45	0.32	0.18	0.17	0.16	0.15
2.00	0.88	0.79	0.69	0.60	0.49	0.39	0.28	0.16	0.15
2.50	0.95	0.87	0.79	0.70	0.62	0.53	0.44	0.35	0.25
3.00	1.00	0.94	0.86	0.79	0.72	0.64	0.57	0.49	0.40
3.50	1.00	1.00	0.93	0.87	0.8	0.74	0.67	0.60	0.52
4.00	1.00	1.00	1.00	0.93	0.87	0.81	0.75	0.69	0.62

Table 7.7 - Values of ρ_o for calculation of shrinkage curvatures

Typos are rectified.

30 Clause 7.3.6 – Figure 7.2

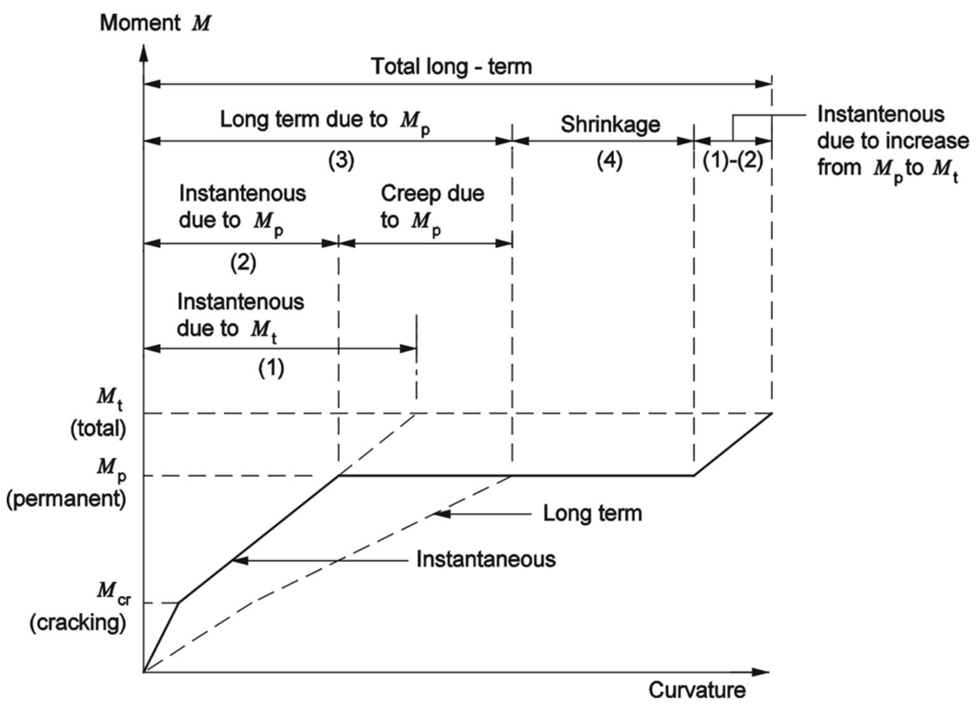


Figure 7.2 - Loading history for serviceability limit state - deflection

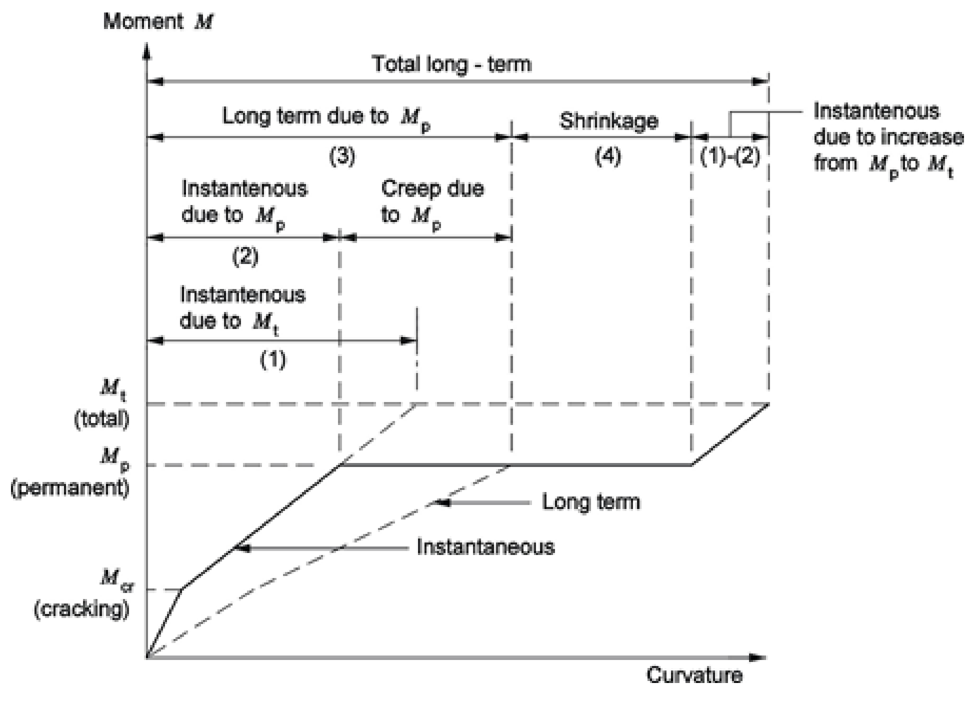


Figure 7.2 - Loading history for serviceability limit state - curvature

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

31 Clause 8.4.3 – equation 8.2

$$f_b = F_s/\pi\phi l_b \qquad 8.2$$

where:

F_s is the force in the bar or group of bars

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

$$f_b = F_s/(\pi\phi l_b) \qquad 8.2$$

where:

F_s is the force in the bar or group of bars

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

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

<table>
<tr>
<td>32</td>
<td>Clause 8.4.4</td>
<td>
Values for design ultimate anchorage bond stress
Values for design ultimate anchorage bond stress f_{bu} may be obtained from the equation:
$f_{bu} = \beta\sqrt{f_{cu}}$ 8.3
where:
f_{cu} is the characteristic compressive cube strength of concrete, limited to 60 N/mm^2 for the purpose of calculating ultimate anchorage bond stress,
f_{bu} is the design ultimate anchorage bond stress,
β 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 6.3, the values of β may be taken from table 8.3. These values include a partial safety factor γ_m of 1.4
<table>
<tr><th rowspan="2">Bar type</th><th colspan="2">β</th></tr>
<tr><th>Bars in tension</th><th>Bars in compression</th></tr>
<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>0.81</td></tr>
</table>
Table 8.3 - Values of bond coefficient β
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.
</td>
<td>
Values for design ultimate anchorage bond stress
Values for design ultimate anchorage bond stress f_{bu} may be obtained from the equation:
$f_{bu} = \beta\sqrt{f_{cu}}$ 8.3
where:
f_{cu} is the characteristic compressive cube strength of concrete, limited to 60 N/mm^2 for the purpose of calculating ultimate anchorage bond stress,
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For bars in tension in slabs or in beams where minimum links have been provided in accordance with Table 6.2, the values of β may be taken from Table 8.3. These values include a partial safety factor γ_m of 1.4
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Table 8.3 - Values of bond coefficient β
In beams where minimum links in accordance with Table 6.2 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.
</td>
<td>Typos are rectified.</td>
</tr>
<tr>
<td>33</td>
<td>Clause 8.4.5 - equation 8.4</td>
<td>
Minimum ultimate anchorage bond lengths
The ultimate anchorage bond length l_b should be greater than or equal to the value calculated from:
$l_b \ge f_s\phi/4f_{bu}$ 8.4
where:
f_s is $0.87f_y$
Values for anchorage bond lengths are given in table 8.4 as multiples of bar diameter.
</td>
<td>
Minimum ultimate anchorage bond lengths
The ultimate anchorage bond length l_b should be greater than or equal to the value calculated from:
$l_b \ge f_s\phi/(4f_{bu})$ 8.4
where:
f_s is $0.87f_y$
Values for anchorage bond lengths are given in table 8.4 as multiples of bar diameter.
</td>
<td>A bracket is added to the denominator of equation 8.4 for clarity.</td>
</tr>
</table>

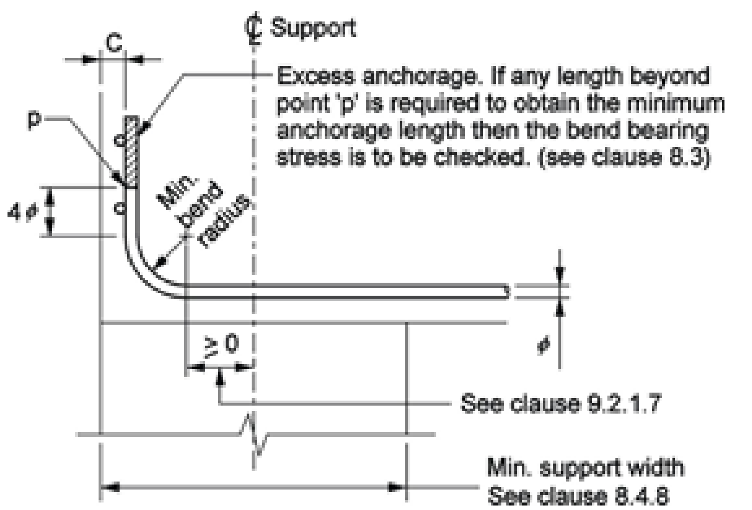
<table>
<tr><td>34</td><td>Clause 8.4.6 – Figure 8.1</td><td>

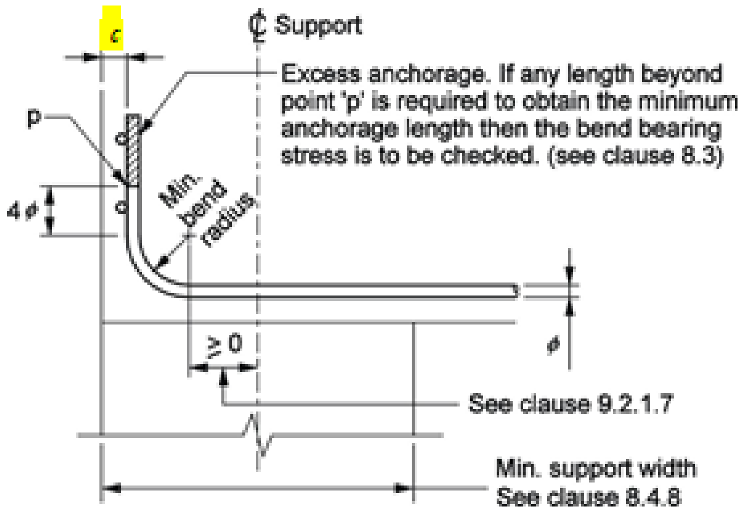
Figure 8.1 - Requirements of a bend anchorage</td><td>

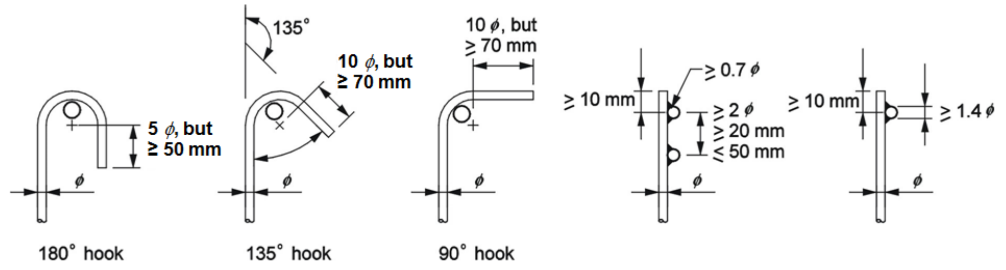
Figure 8.1 - Requirements of a bend anchorage</td><td>The symbol “C” is amended to “c” in order to tally with the notation used in Equations 8.5 and 8.6.</td></tr>
<tr><td>35</td><td>Clause 8.4.8</td><td>Minimum support widths
Supports in the form of beams, columns and walls etc. should have a minimum width of:
$2(4\phi + c)$ for anchored bars less than 20 mm in diameter, or 8.5
$2(5\phi + c)$ for anchored bars 20 mm or larger in diameter. 8.6
See figure 8.1 for notation.</td><td>Minimum support widths
Supports in the form of beams, columns and walls etc. should have a minimum width of:
$2(3\phi + c)$ for anchored bars less than or equal to 12 mm in diameter, or 8.5a
$2(4\phi + c)$ for anchored bars larger than 12 mm but less than 20 mm in diameter, or 8.5
$2(5\phi + c)$ for anchored bars 20 mm or larger in diameter. 8.6
See figure 8.1 for notation.</td><td>An equation for calculating the minimum support width for anchor bars less than or equal to 12mm in diameter is added to tally with the minimum bend radius for hooks and loops given in Table 8.2.</td></tr>
<tr><td>36</td><td>Clause 8.5 – Figure 8.2</td><td>

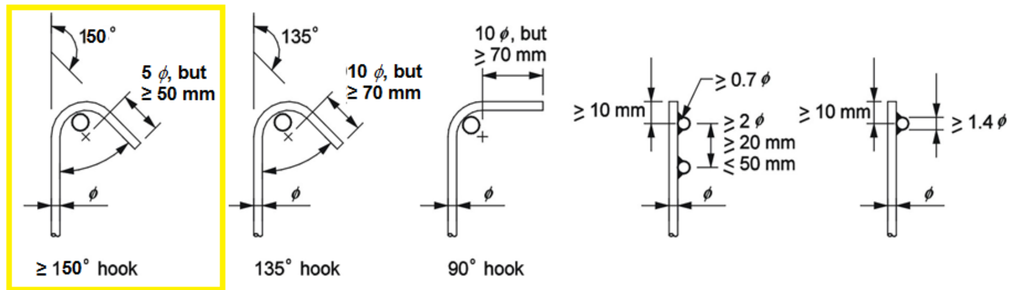
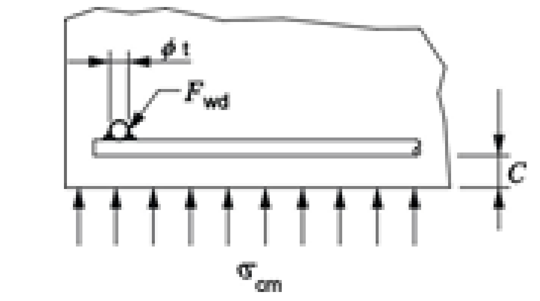
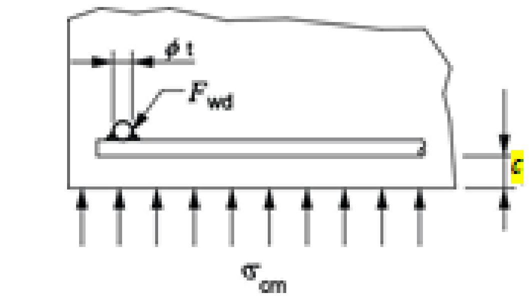
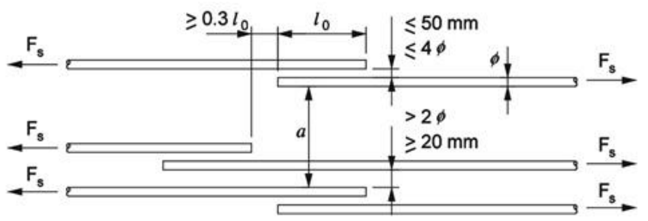
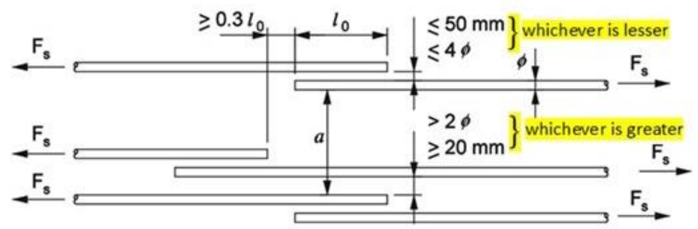
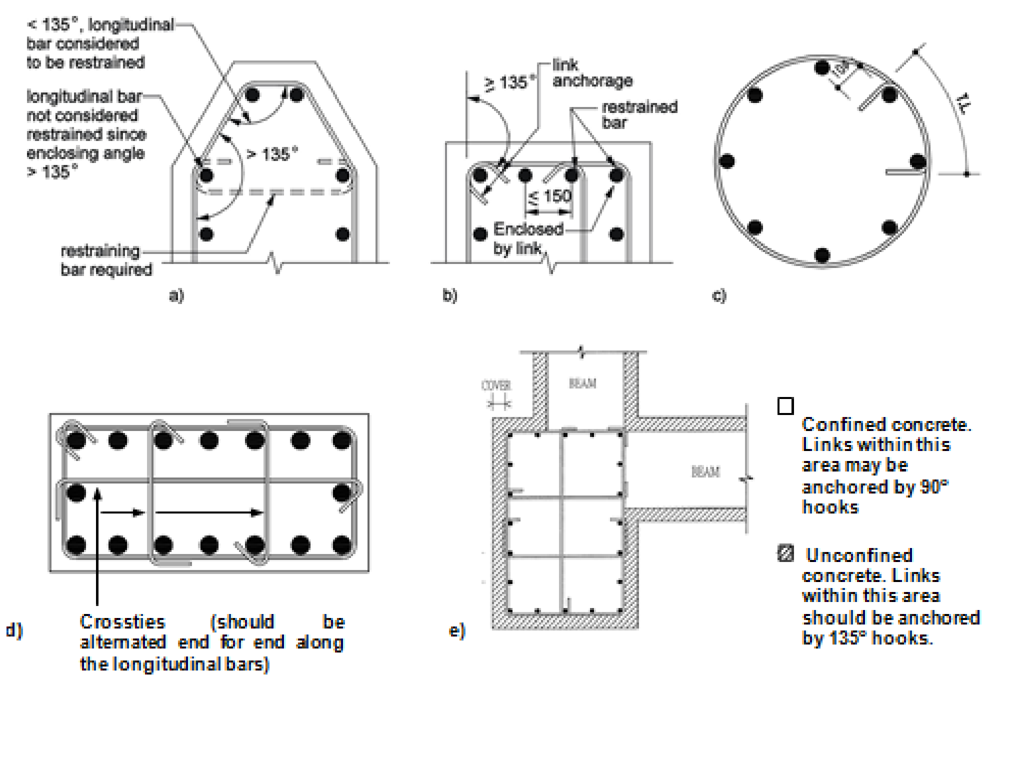
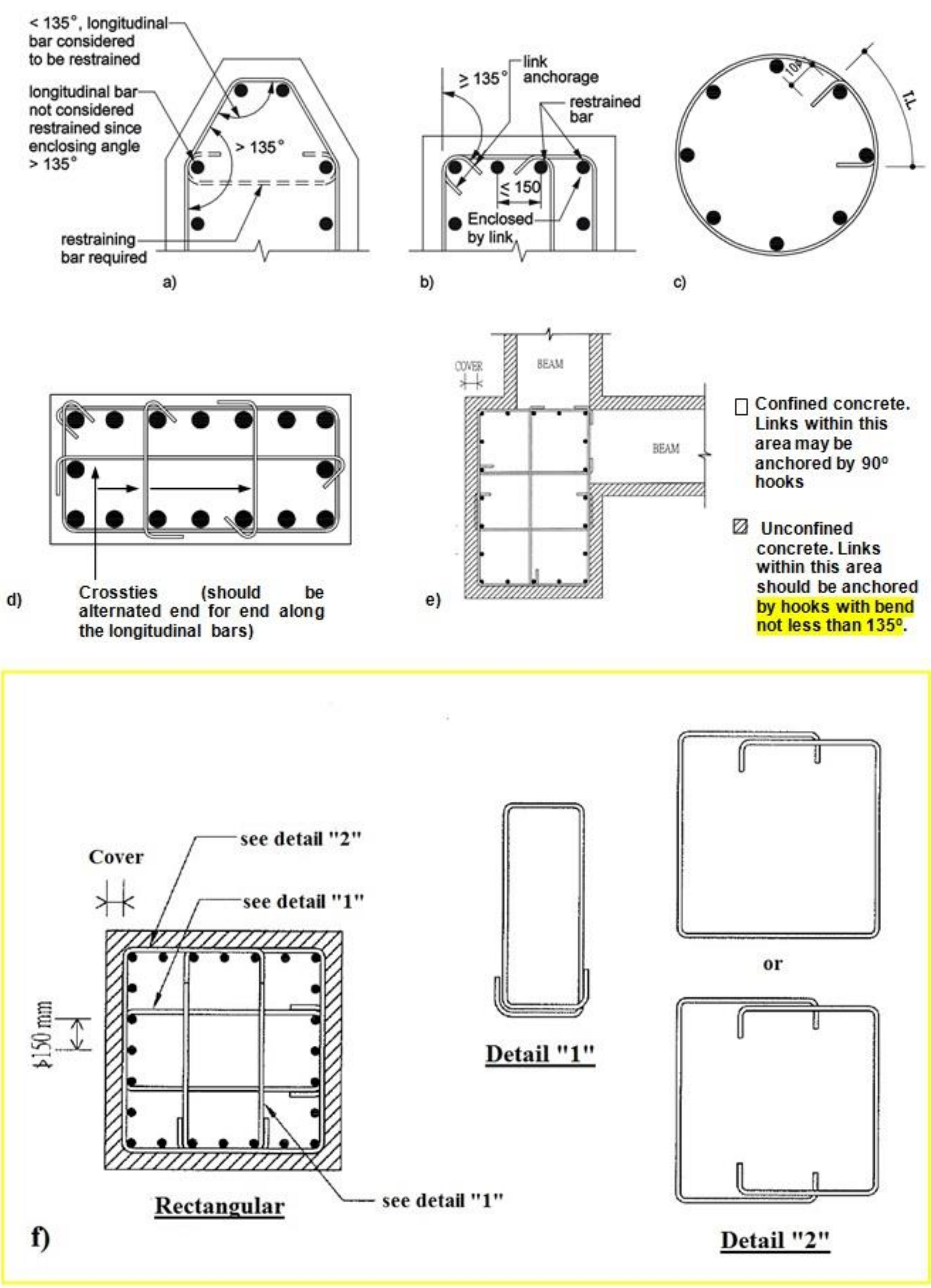
Figure 8.2 - Anchorage of links</td><td>

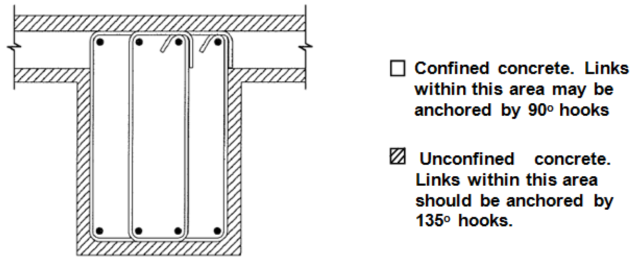
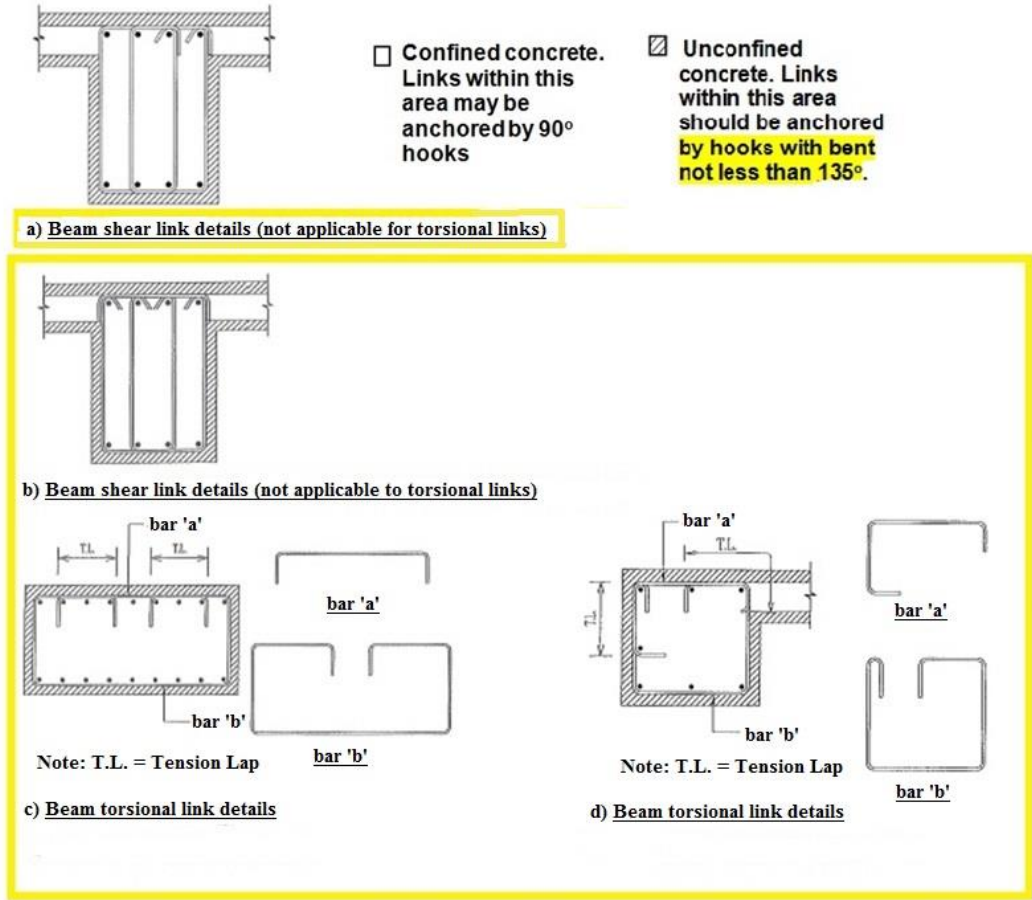
Figure 8.2 - Anchorage of links</td><td>Additional details of anchorage of links for hooks with 150° & above is added.
Original detail of 180° hook is covered by the new details and hence is deleted.</td></tr>
</table>

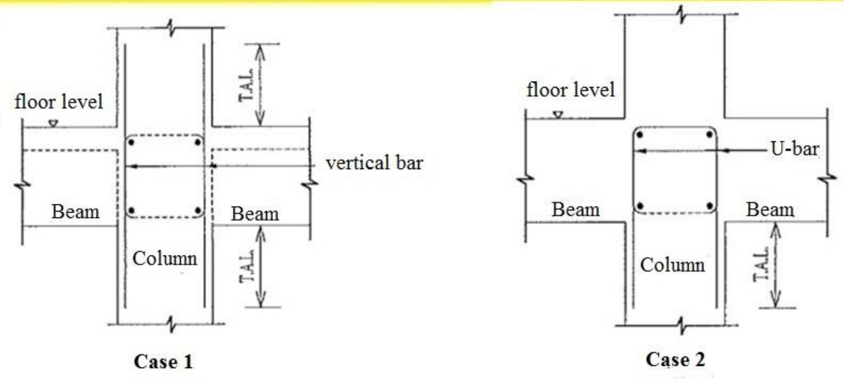
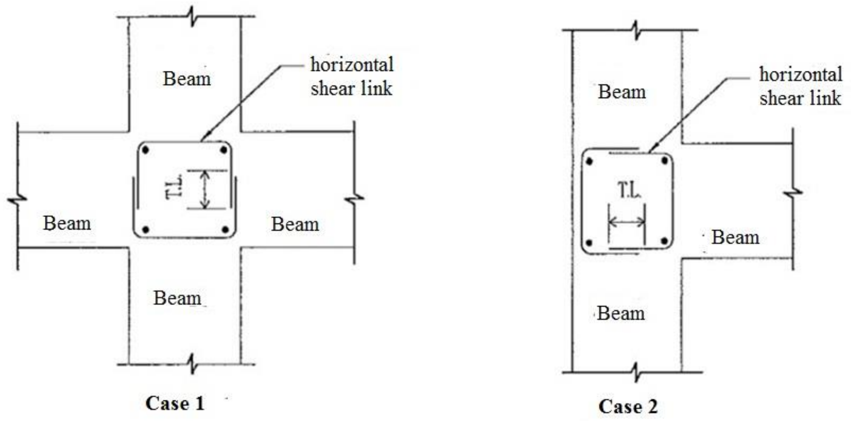
37	Clause 8.6 – Figure 8.3	 **Figure 8.3 - Welded transverse bar as anchoring device**	 **Figure 8.3 - Welded transverse bar as anchoring device**	The symbol "C" is amended to "c" in order to tally with the notation used in Equations 8.5 and 8.6.
38	Clause 8.7	**8.7 LAPS AND MECHANICAL COUPLERS** **8.7.1 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 cyclical bars should not be joined by welding.	**8.7 LAPS** **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 cyclical bars should not be joined by welding.	The title "LAPS AND MECHANICAL COUPLES" is amended to "LAPS" as the requirements for mechanical couplers are given in Clause 3.2.8.
39	Clause 8.7.2 & Figure 8.4	(d) the clear transverse distance between two lapping bars should not be greater than 4ϕ or 50 mm, otherwise the lap length should be increased by a length equal to the clear space exceeding 4ϕ or 50mm; (e) the longitudinal distance between two adjacent laps should not be less than 0.3 times the lap length, l_0; and (f) in case of adjacent laps, the clear distance between adjacent bars should not be less than 2ϕ 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.  **Figure 8.4 - Adjacent laps**	(d) the clear transverse distance between two lapping bars should not be greater than 4ϕ or 50 mm, otherwise the lap length should be increased by a length equal to the clear space exceeding 4ϕ or 50mm, whichever is lesser; (e) the longitudinal distance between two adjacent laps should not be less than 0.3 times the lap length, l_0; and (f) in case of adjacent laps, the clear distance between adjacent bars should not be less than 2ϕ or 20 mm, whichever is greater. 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 2 or more layers. All bars in compression and secondary (distribution) reinforcement may be lapped in one section.  **Figure 8.4 - Adjacent laps**	● 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.
40	Clause 9.1	Detailing of members should normally comply with both the general detailing rules given in clauses 9.2 to 9.8 and the particular rules for ductility given in clause 9.9. However, members not contributing in the lateral load resisting system do not need to conform to the requirements of clause 9.9.	Detailing of members should normally comply with both the general detailing rules given in clauses 9.2 to 9.8 and the particular rules for ductility given in clause 9.9. However, members not contributing in the lateral load resisting system or walls for single storey structures do not need to conform to the requirements of clause 9.9.	Walls for single storey structures are exempt from the ductility design requirement.
41	Clause 9.4.1(j)	Cantilevered slabs exposed to weathering should be designed for : (i) exposure condition 2 or higher if appropriate; (j) estimated maximum crack width not exceeding 0.1 mm under serviceability limit state.	Cantilevered slabs exposed to weathering should be designed for : (i) exposure condition 2 or higher if appropriate; (j) estimated maximum crack width not exceeding 0.1 mm under serviceability limit state or the stress of deformed high yield steel reinforcing bars used should not exceed 100 N/mm^2 when checking the flexural tension under the working load condition.	The alternative checking method for exposed cantilevered slab by limiting the stress of deformed high yield steel reinforcing bars to 100N/mm^2 as given in Appendix A of PNAP APP-68 is incorporated.

Legend: revision/addition

42	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.	**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. External cantilevered slabs with a span exceeding 750 mm exposed to weathering should satisfy the following requirements:- (a) concrete should be water-proof concrete of characteristic compressive strength not less than 35MPa at 28 days; (b) all main steel reinforcing bars should be hot-dip galvanized to BS EN ISO 1461; and (c) water-proof membrane/tanking should be provided and protected by 1:3 cement sand mortar of 0.65 maximum free water/cement ratio or other equivalent means.	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.
43	Clause 9.5	**COLUMNS** This clause deals with columns for which the larger dimension h_c is not greater than 4 times the smaller dimension b_c.	**COLUMNS** This clause deals with columns for which the larger dimension is not greater than 4 times the smaller dimension .	The symbols “b_c” & “h_c” are deleted.
44	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 though an angle of not less than 135° (see figure 9.5b). Crossties 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 9.5d). Where there is adequate confinement to prevent the end anchorage of the link from “kick off”(see figure 9.5e), the 135° hook in the links or crossties may be replaced by other standard hoods given in figure 8.2.	*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 though an angle of not less than 135° (see Figure 9.5b). Crossties 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 9.5d). Where there is adequate confinement to prevent the end anchorage of the link from “kick off”(see Figure 9.5e), hooks with bend not less than 135° in the links or crossties may be replaced by other standard hooks given in Figure 8.2.	Textual refinements and typos are rectified.

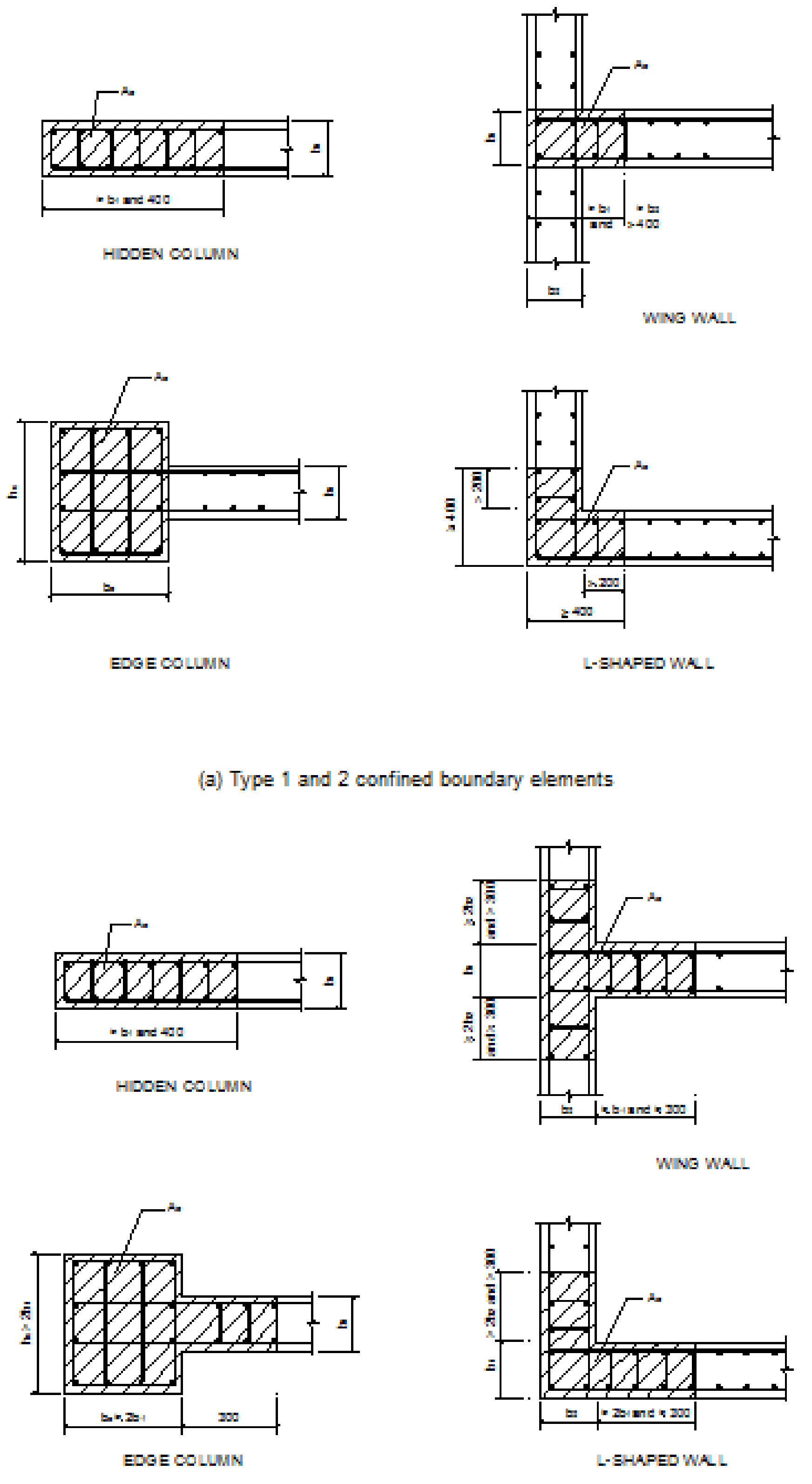
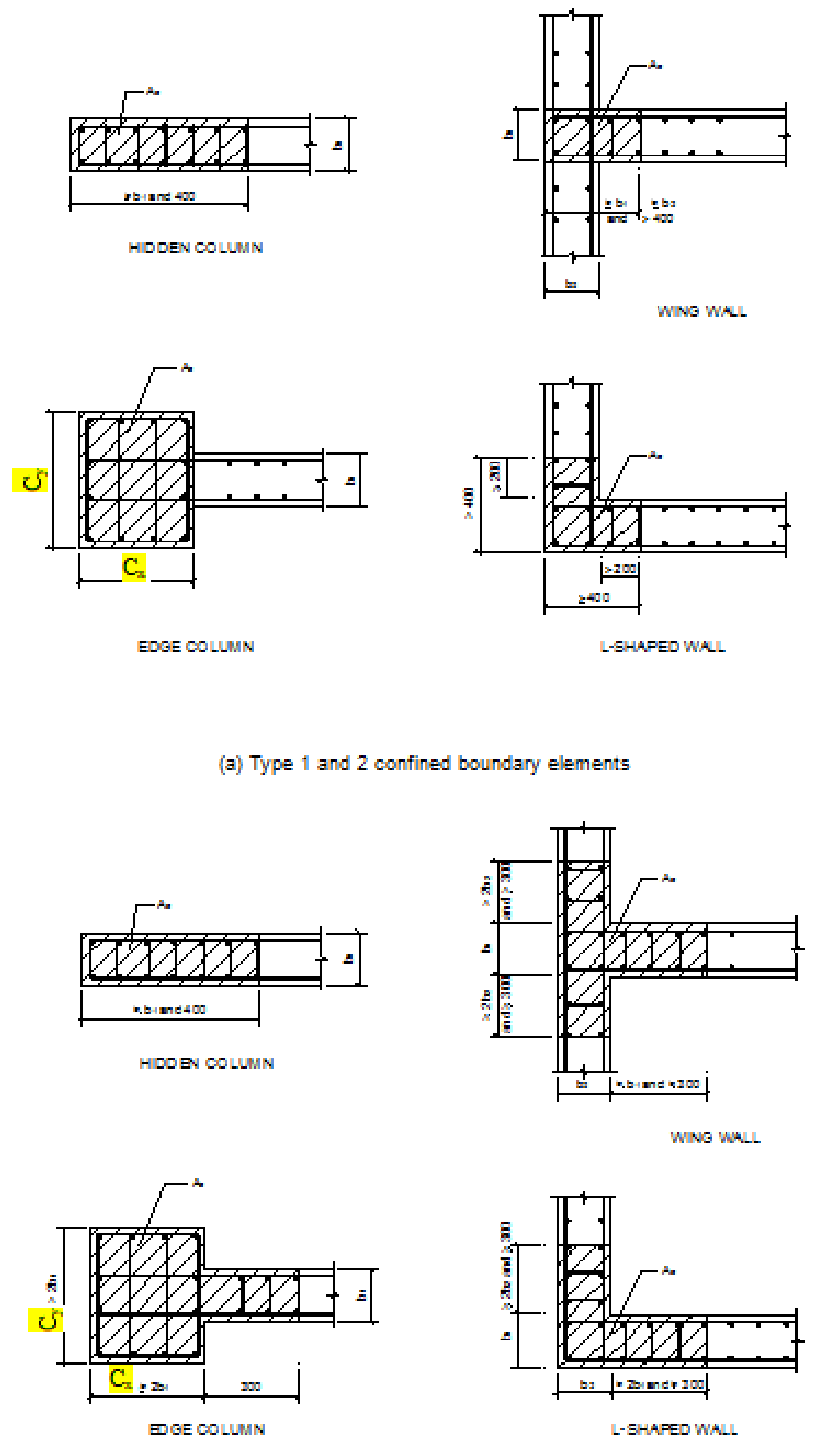
45	Clause 9.5.2.4 and Figure 9.5f added	 **Figure 9.5 - Column transverse reinforcement**	*9.5.2.4 Alternative details* Alternative arrangement of column transverse reinforcement is given in Figure 9.5f.  **Figure 9.5 - Column transverse reinforcement**	The alternative details of column transverse reinforcement as stipulated in BD's circular letter dated 29 April 2011 are incorporated.
46	Clause 9.9.1.3 (a)(i)	The centre-to-centre spacing of links or ties along a beam shall not exceed: (i) Inside the critical zone: the larger of 150mm or 8 times the longitudinal bar diameter	The centre-to-centre spacing of links or ties along a beam shall not exceed: (i) Inside the critical zone: the smaller of 150mm or 8 times the longitudinal bar diameter	A typo is rectified.

47	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 hoods given in figure 8.2 (see figure 9.7).  Figure 9.7 – Typical confinement in beam	Anchorage Links should be adequately anchored by means of hooks with bend not less than 135° 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”, hooks with bent not less than 135° may be replaced by other standard hooks given in Figure 8.2 (see Figure 9.7).  Figure 9.7 - Typical confinement in beam	Textual refinements and typos are rectified. The alterative beam shear link details as stipulated in BD’s circular letter dated 29 April 2011 are incorporated.
48	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 9.5b, c, 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 hoods given in figure 8.2.	Anchorage Links and ties should be adequately anchored by means of hooks with bend not less than 135° in accordance with clause 9.5.2 (see Figure 9.5b, c, d & e). Where there is adequate confinement to prevent the end anchorage of the link from “kick off”, hooks with bend not less than 135° may be replaced by other standard hooks given in Figure 8.2.	Textual refinements and a typo are rectified.

49	Clause 9.2.4 and Figure 9.7a added		9.9.2.4 *Shear reinforcement in beam-column joints* Typical details of shear reinforcement in beam-column joints are given in figure 9.7a.  Notes: 1. Vertical shear reinforcement may be provided by vertical bars or inverted U-bars with adequate anchorages into column above or below the joint. 2. T.A.L. = Tension Anchorage Length a) **Beam-column joint - elevation showing vertical shear reinforcement**  Notes: 1. Horizontal shear reinforcement may be provided by U-bars. 2. Shear reinforcement should not extend outside column section. b) **Beam-column joint - plan showing horizontal shear reinforcement** **Figure 9.7a - Beam-Column Joint Shear Reinforcement**	The typical beam-column joint shear reinforcement details as stipulated in BD's circular letter dated 29 April 2011 are incorporated.

Legend: revision/addition

50	Clause 9.9.3.2	*Confined boundary elements* 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 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 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 of at least 12mm diameter and vertical spacing not exceeding 150 mm.	*Confined boundary elements* 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. Links and ties should be adequately anchored by means of hooks with bend not less than 135° in accordance with clause 9.5.2 (see Figure 9.5b, c, d, e & f). Where there is adequate confinement to prevent the end anchorage of the link from "kick off", hooks with bend not less than 135° may be replaced by other standard hooks given in Figure 8.2.	• The vertical bar in the confined boundary elements of walls should be tied with links or ties. • 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.

51	Clause 9.9.3 – Figure 9.11	 (a) Type 1 and 2 confined boundary elements (b) Type 3 confined boundary elements **Figure 9.11 – Details of confined boundary elements**	 (a) Type 1 and 2 confined boundary elements (b) Type 3 confined boundary elements **Figure 9.11 – Details of confined boundary elements**	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.

52	Clause 10.3.4.2(b)	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 150mm test cubes or 8.5 MPa for 100mm test cubes; or (v) For concrete grade exceeding C60, the coefficient of variation exceeds 14%.	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 150mm 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 should 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 all 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.	The conditions for temporary resumption of concreting works are added.
53	Clause 10.3.4.3 added		10.3.4.3 Further testing (a) When concrete is considered to be suspect from visual inspection, or when the specified grade strength has been deemed not to be attained under clause 10.3.4.2(b), the compressive strength of the concrete in the structure may be determined by drilling a sufficient number of cores from the concrete at suitable locations. (b) The core should be prepared in accordance with the requirements given in CS1. (c) Cores drilled from concrete should be prepared and tested by a recognized method to determine the compressive strength. (d) No adjustment should be made to the measured strength in respect of the age of the core when tested. (e) Criteria for acceptance (i) Concrete cores should not show evidence of segregation of individual materials. (ii) There should be no honeycombing in the cores which means interconnected voids arising from, for example, inadequate compaction or lack of mortar. (iii) For any set of cores representing a test location, the estimated in-situ cube strength of each core specimen should be at least 75% of the specified grade strength and the average estimated in-situ cube strength of the set should be at least 85% of the specified grade strength. In this respect, the estimated in-situ cube strength of each core specimen should be calculated in accordance with CS1.	The coring test requirements and corresponding acceptance criteria for further testing required under regulation 63 of the Building (Construction) Regulations are added.
54	Clause 10.3.6.1	The method of curing should be specified in detail where members are of considerable bulk or length, the cement content of the concrete is high, the surface finish is critical or special or accelerated curing methods are to be applied.	The method of curing should be specified in detail where members are of considerable bulk or length, the cementitious content of the concrete is high, the surface finish is critical or special or accelerated curing methods are to be applied.	Consequential amendments (see remarks for Clause 4.2.1 in item 7 above).
55	Clause 12.3.6.1	General The serviceability requirements for deflection are given in clause 2.2.3.2, but no numerical limits are set. For reinforced concrete, in all normal cases, deflections are controlled by limiting the ratio of span to effective depth. In general, this approach is not possible for prestressed concrete, because of the major influence of the level of prestress. When it is considered necessary to calculate deflections, the methods outlined in clause 12.3.6.2 may be used.	General The serviceability requirements for deflection are given in clause 2.2.4.2, but no numerical limits are set. For reinforced concrete, in all normal cases, deflections are controlled by limiting the ratio of span to effective depth. In general, this approach is not possible for prestressed concrete, because of the major influence of the level of prestress. When it is considered necessary to calculate deflections, the methods outlined in clause 12.3.6.2 may be used.	A typo is rectified.

Legend: revision/addition

56	Clause 12.12.3.1	(b) Cover against corrosion 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/m^3.	(b) Cover against corrosion 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 cementitious content should not be reduced below 300 kg/m^3.	Consequential amendments (see remarks for Clause 4.2.1 in item 7 above) and textural refinements.
57	Clause 13.2	***TEST LOADS*** 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 1.25 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, partitions, 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.	***TEST LOADS*** 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 1.25 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, partitions, 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.	The term "characteristic live load" is amended to "characteristic imposed load".
58	Annex A	BS 4483:2005 Steel fabric for the reinforcement of concrete. Specification BS 4486:1980 Specification for hot rolled and hot rolled and processed high tensile alloy steel bars for the prestressing of concrete BS EN 480-4:2005 Admixtures for concrete, mortar and grout. Test methods. Determination of bleeding of concrete	BS 4483:2005 Steel fabric for the reinforcement of concrete. Specification BS 4486:1980 Specification for hot rolled and hot rolled and processed high tensile alloy steel bars for the prestressing of concrete BS EN 13391:2004 Mechanical tests for post-tensioning systems ETAG 013 Post Tensioning Kits for prestressing of Structures BS EN 480-4:2005 Admixtures for concrete, mortar and grout. Test methods. Determination of bleeding of concrete	The list of acceptable standards is updated to include the testing standards for post-tensioning systems.