Detailing of Reinforcement in Concrete Structures

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Introduction:

- **Detailing** is often considered to be the preparation of working drawings showing the size and location of the reinforcement in a concrete structure.

- **Detailing** involves the communication of the engineer’s design to the contractors who build the structure. It involves the translation of a good structural design from the computer or calculation pad into the final structure.

- Good detailing ensures that reinforcement and concrete interact efficiently to provide satisfactory behaviour throughout the complete range of loading.

- In this seminar, guidelines for successful detailing in structural elements and connections are outlined.
The **detailing requirements** of a reinforcement bar depend on the reasons for its inclusion in the structure.

Reasons include:

1. To carry internal tensile forces, thereby **impacting strength and ductility**;
2. To control **flexural cracking**;
3. To control **direct tension cracking** in restrained structures;
4. To carry **compressive forces**;
5. To provide **restraint** to bars in compression;
6. To provide **confinement** to concrete in compression;
7. To **limit long-term deformation**;
8. To provide **protection against spalling**; and
9. To provide **temporary support** for other reinforcement during construction.
**Guiding principles:**

- Determine location and direction of all internal forces (i.e. establish a load path that satisfies equilibrium);
- Use adequately anchored reinforcement wherever a tensile force is required for equilibrium;
- Use only ductile reinforcement (Class N or better) when the reinforcement is required for strength;
- Never rely on the concrete’s ability to carry tension (it may not exist);
- Include adequate quantities of reinforcement for crack control;
- Ensure steel details are practical and that steel can be fixed and concrete can be satisfactorily placed and compacted around complex details with adequate cover; and
- Ensure details are economical.
Sources of tension:

1. Tension caused by bending (and axial tension):
   - Positive bending
   - Negative bending
   - Axial tension
Sources of tension:

2. Tension caused by load reversals:

- Cantilever beam or slab
  - Construction loads
  - Bottom steel required
  - Temporary end support

- Simple beam or slab
  - Top steel required
  - Temporary internal support

- Impact and rebound loading
  - Full depth cracking may occur
Sources of tension (ctd):

3. Tension caused by shear and torsion:

- Shear
- Flexure-shear cracks
- Tension carried by stirrups
Sources of tension (ctd):

4. Tension near the supports of beams:

The longitudinal tension at the support is greater than indicated by the bending moment diagram.

The tensile force at the bottom of the inclined crack is equal to the compressive force at the top of the crack.
Sources of tension (ctd):

AS3600-2009 (8.1.10.4):

Sufficient bottom steel must be anchored for a length ($L_{st}$) past the mid-point of the bearing to develop a tensile force of $V\cdot\cot \theta_v/\phi$ (plus any additional force arising from restraint).

This requirement is deemed to be satisfied if either

- $\geq \frac{1}{2} A_{st}$ is extended past the face of the support by $\geq 12d_b$; or
- $\geq \frac{1}{3} A_{st}$ is extended past the face of the support by $\geq 12d_b + D/2$

where $A_{st}$ is the tensile steel area required at mid-span.
Sources of tension (ctd):

5. Tension within the supports of beams and slabs:

- Shortening of beams due to shrinkage can cause tension and subsequent cracking in the supports if longitudinal movement is restrained.

- The problem may be overcome by the introduction of suitable contraction joints.

Cracking due to inadequate slip joint between slab and supporting brickwall.
Sources of tension (ctd):

6. Tension within connections:

(a) Internal forces

(b) Crack pattern

$\sqrt{2} T$

Primary girder

Compression struts

Hanger reinf. to carry tension

Reaction from secondary beam applied here
Sources of tension (ctd):

7. Tension at concentrated loads:

- Longitudinal crack
- Transverse tension
- Potential splitting in plane of bend
- Transverse tension
Sources of tension (ctd):

8. Tension caused by directional changes of internal forces:

(a) Tension caused by changes in internal forces.

(b) Stirrups at spacing $L_{sy,t}$.

(c) Potential crack in web at spacing $s$.
Sources of tension (ctd):

8. Tension caused by directional changes of internal forces:

Transverse tension:

\[ q_t = \frac{T}{r_m} = \frac{A_{st}f_{sy}}{r_m} \]

Required stirrup spacing:

\[ s = \frac{A_{sv}f_{vy}}{q_t} = \frac{A_{sv}}{A_{st}} \cdot \frac{f_{vy}}{f_{sy}} \cdot r_m \]
Anchorage of deformed bars is tension:

- (a) Forces exerted by concrete on a deformed bar at a tensile anchorage
- (b) Tensile stresses in concrete
- (c) Horizontal splitting due to insufficient bar spacing.
- (d) Vertical splitting due to insufficient cover
- (e) Splitting (bond) failure at a lapped splice.

Splitting failures around developing bars.
• For a reinforcement bar to reach its yield stress at a critical cross-section, a minimum length of reinforcing bar (an anchorage) is required on either side of the section.

• AS3600-2009 specifies a minimum length, called the development length, $L_{sy,t}$, over which a straight bar in tension must be embedded in the concrete in order to develop the yield stress.

\[
L_{sy,t} \geq \frac{d_b}{4} \frac{f_{sy}}{\phi f_b}
\]

• An average design ultimate bond stress $\phi f_b$ is assumed at the interface between the concrete and the reinforcing bar ($\phi = 0.6$).

• $\phi f_b$ depends on - type and condition of reinforcing bar; strength and compaction of concrete; concrete cover; bar spacing; transverse reinforcement; transverse pressure (or tension).
**AS3600-2009:**  (§13.1.2.2)

- The basic development length, \( L_{sy.tb} \), is

\[
L_{sy.tb} = \frac{0.5k_1k_3f_{sy}d_b}{k_2\sqrt{f'_c}} \geq 29k_1d_b
\]

where  \( k_1 = 1.3 \)  for a horizontal bar with > 300mm of concrete cast below it and  \( k_1 = 1.0 \)  for all other bars;

\[
k_2 = (132 - d_b)/100;
\]

\[
k_3 = 1.0 - 0.15(c_d - d_b)/d_b \quad \text{(but } 0.7 \leq k_3 \leq 1.0)\]

\( c_d \)  is the smaller of the concrete cover to the bar or half the clear distance to the next parallel bar;

\( f'c \)  shall not be taken to exceed 65 MPa
\( c_d = \min(a/2, c, c_1) \)
The development length $L_{sy.t}$ may be taken as the basic development length or may be refined to include the beneficial effects of confinements by transverse steel or transverse pressure and is

$$L_{sy.t} = k_4 k_5 L_{sy.tb}$$

where $k_4 = 1 - K \lambda$ (but $0.7 \leq k_4 \leq 1.0$); and

$$k_5 = 1.0 - 0.04 \rho_p \quad (\text{but } 0.7 \leq k_5 \leq 1.0);$$
$k_4 = 1 - K\lambda$

where

$\lambda = (\Sigma A_{tr} - \Sigma A_{tr.min})/A_s$;

$\Sigma A_{tr} =$ cross-sectional area of the transverse reinforcement along the development length $L_{sy.t}$

$\Sigma A_{tr.min} =$ cross-sectional area of the minimum transverse reinforcement, which may be taken as $0.25A_s$ for beams and 0 for slabs

$K =$ is a factor that accounts for the position of the bars being anchored relative to the transverse reinforcement, with values given below:

$K = 0.1$

$K = 0.05$

$K = 0$

FIGURE 13.1.2.3(B) VALUES OF K FOR BEAMS AND SLABS
• The development length $L_{st}$ to develop a stress $\sigma_{st}$ lower than $f_{sy}$:

$$L_{st} = L_{sy.t} \frac{\sigma_{st}}{f_{sy}} \geq 12d_b$$

When calculating $\sigma_{st}$ don’t forget to include the strength reduction factor ($\phi = 0.8$). If $T^*$ is the design ultimate tensile force in the reinforcement caused by the factored design loads, then:

$$T^* \leq \phi \sigma_{st} A_{st}$$

and therefore

$$\sigma_{st} \geq \frac{T^*}{\phi A_{st}}$$
The development length of a deformed bar with a standard hook or cog:

(a) Standard hook (180° bend)

(b) Standard hook (135° bend)

(c) Standard cog (90° bend)
**WORKED EXAMPLE:**

Consider the minimum development length required for the two terminated 28 mm diameter bottom bars in the beam shown below.

Take $f_{sy} = 500 \text{ MPa}$; $f'_c = 32 \text{ MPa}$; cover to the 28 mm bars $c = 40 \text{ mm}$; and the clear spacing between the bottom bars $a = 60 \text{ mm}$.

The cross-sectional area of one N28 bar is $A_s = 620 \text{ mm}^2$ and with N12 stirrups at 150 mm centres, $A_{tr} = 110 \text{ mm}^2$.
Worked Example  ctd  (§13.1.2.3)

For bottom bars:  \( k_1 = 1.0 \);

For 28 mm diameter bars:  \( k_2 = (132 – 28)/100 = 1.04 \);

The concrete confinement dimension,  \( c_d = a/2 = 30 \text{ mm} \), and therefore

\[ k_3 = 1.0 – 0.15(30 – 28)/28 = 0.99 \]

The basic development length is therefore

\[ L_{sy.tb} = \frac{0.5k_1k_3f_{sy}d_b}{k_2\sqrt{f_c'}} = 1178 \text{ mm} \ (> 29k_1d_b) \]

The minimum number of stirrups that can be located within the basic development length is 7. Therefore,  \( \Sigma A_{tr} = 7 \times 110 = 770 \text{ mm}^2 \).

Taking  \( \Sigma A_{tr.min} = 0.25A_s = 155 \text{ mm}^2 \), the parameter

\[ \lambda = (770 – 155)/620 = 0.99 \]
From Figure 13.1.2B, $K = 0.05$ (as it is the two interior bars that are being developed) and therefore

$$k_4 = 1.0 - K\lambda = 1.0 - 0.05 \times 0.99 = 0.95$$

It is assumed that in this location the transverse pressure perpendicular to the anchored bar ($\rho_p$) is zero, and hence $k_5 = 1.0$.

From Eq. 13.1.2.3:

$$L_{sy.t} = k_4 k_5 L_{sy.tb} = 0.95 \times 1.0 \times 1178 = 1120 \text{ mm}.$$  

The strength of the beam must be checked at the point where the two bars are terminated (ie. at $L_{sy.t} + d$ from the constant moment region)
Lapped Splices for bars in tension (13.2.2 – AS3600-2009):

PLANAR VIEW

\[ s_L \]

\[ d_b \]

Note: For the purposes of determining \( c_d \), the dimension \( a \) shall be taken equal to \((s_L-d_b)\) irrespective of the value of \( s_b \).

\[ c_d = \min \left( \frac{a}{2}, c \right) \]

(i) 100% of bars spliced (no staggered splices)

\[ \geq 0.3L_{sy,t,lap} \]

PLANAR VIEW

\[ s_L \]

\[ a \]

Note: For the purposes of determining \( c_d \), the dimension \( a \) shall be taken equal to \( 2s_L \) irrespective of the value of \( s_b \).

\[ c_d = \min \left( \frac{a}{2}, c \right) \]

(ii) 50% staggered splices
AS3600-2009: §13.2.2

Lapped Splices for bars in tension:

\[ L_{syt,lap} = k_7 L_{syt} \geq 29k_1 d_b \]

\(k_7\) shall be taken as 1.25, unless \(A_s\) provided is greater than \(A_s\) required and no more than one-half of the tensile reinforcement at the section is spliced, in which case \(k_7 = 1\).

In narrow elements or members (such as beam webs and columns), the tensile lap length \(L_{syt,lap}\) shall be not less than the larger of \(k_7 L_{syt}\) and \(L_{syt} + 1.5s_b\), where \(s_b\) is the clear distance between bars of the lapped splice as shown in Figure 8.15.
Consider the lapped splice requirements for N12 bars at 200 mm centres in the bottom of a slab. Cover = 20 mm. Concrete strength = 25 MPa.

**AS3600-2009:**

\[
L_{\text{sy.t.lap}} = 1.25L_{\text{sy.tb}} = \frac{0.5k_1k_3f_{\text{sy}}d_b}{k_2\sqrt{f'_c}}
\]

\[
= 1.25 \times \frac{0.5 \times 1.0 \times 0.90 \times 500 \times 12}{1.2 \times \sqrt{25}}
\]

\[
= 563 \text{ mm} = 46.9d_b
\]

(c.f. 369 mm = 30.7 \(d_b\) in AS3600–2001)

**ACI 318-08: (Simplified – Clause 12.2.2)**

\[
L_{\text{lap}} = 1.3\ell_d = 1.3 \frac{f_y\psi_t\psi_e}{2.1\lambda\sqrt{f'_c}} \times d_b
\]

\[
= 1.3 \times \frac{500 \times 1.0 \times 1.0}{2.1 \times 1.0 \times \sqrt{25}} \times 12
\]

\[
= 743 \text{ mm} = 61.9d_b
\]

**ACI 318-08: (Refined – Clause 12.2.3)**

\[
L_{\text{lap}} = 1.3\ell_d = 1.3 \frac{f_y\psi_t\psi_e\psi_s}{1.1\lambda\sqrt{f'_c}(c_b + K_{vt})} \times d_b
\]

\[
= 1.3 \times \frac{500 \times 1.0 \times 1.0 \times 0.8}{1.1 \times 1.0 \times \sqrt{25} \left(\frac{26 + 0}{12}\right)} \times 12
\]

\[
= 524 \text{ mm} = 43.7d_b
\]
Detailing of beams:

Anchorage of longitudinal reinforcement:

When bottom reinforcement is terminated away from the support, the diagonal compression in the web improves the anchorage.
AS3600-2009 Clause 8.1.10.1

Current wording:
“The design for flexural strength and detailing of flexural reinforcement and pretensioned tendons at termination shall be extended from the theoretical cut-off point, or debonding point, by a length of $1.0D + L_{sy.t}$ or $1.0D + L_{pt}$, where D is the member depth at the theoretical cut-off point or theoretical debonding point”

- Problem 1: The wording does not make sense
- Problem 2: The rule is incorrect – a bar does not have to develop its yield stress at the theoretical cut-off point

Amended wording:
“Where flexural reinforcement and pretensioned tendons are to be terminated, the bars or tendons shall be extended from the theoretical cut-off point, or theoretical debonding point, by a length of at least $1.0D + L_{sy.t}$ or $1.0D + L_{pt}$, respectively, where D is the member depth at the theoretical cut-off point or theoretical debonding point”
Detailing of beams (ctd):

The transverse tension that may cause splitting in the plane of a **hooked anchorage** at a support can be overcome at a beam support simply by tilting the hook and exposing it to the normal reaction pressure.
Detailing of beams (ctd):

If the bearing length at a support is small and close to the free end of a member, a *sliding shear failure* along a steep inclined crack may occur. Additional small diameter bars may be required perpendicular to the potential failure plane.
Detailing of beams (ctd):

Where the length available for anchorage is small, mechanical anchorages in the form of welded cross-bars or end plates may be used. Common in precast elements, corbels, brackets and at other support points.
In short span members, where load is carried to the support by arch action, it is essential that all bottom reinforcement (the tie of the arch) is fully developed at each support. Closely spaced transverse stirrups can be used to improve anchorage of the tie reinforcement.
Detailing of beams (ctd):

Concentrating top steel at a support in a beam within the web can lead to crack control problems in the adjacent slab (Leonhardt et al.)

![Graph showing crack width vs load with two different cross-sections and area values: As = 1030 mm² and As = 1020 mm².](image)
Detailing of beams (ctd):

Anchorage of Stirrups:

- **Tension in stirrup** is more or less constant over height of vertical leg. Therefore, all points on vertical leg must be fully developed.

- **Stirrup anchorages** should be located in the compressive zone and be shown on the structural drawings.

- The area of shear reinforcement required at a particular cross-section should be provided for a distance $D$ from that cross-section in the direction of decreasing shear (AS3600-2009 – Clause 8.2.12.3).
Detailing of beams (ctd):

Types of Stirrups:

(a) Incorrect
Inadequate anchorage
A 90° cog is ineffective if the cover concrete is lost

(b) Undesirable (but satisfactory)
In regions where ductility is required, the open stirrups (commonly used in post-tensioned beams) do not confine the compressive concrete

(c) Satisfactory

Compressive side

Tensile side

Tensile lapped splice
Detailing of beams (ctd): Types of Stirrups:

- Multi-leg stirrups should be used in members with wide webs to avoid the undesirable distribution of diagonal compression shown.
- Multi-leg sturrups better control shear cracking and help maintain shear transfer through aggregate interlock.
Detailing of beams (ctd):

Types of Stirrups:

- Multi-leg stirrups are also far better for controlling the longitudinal splitting cracks (known as *dowel cracks*) that precipitate bond failure of the longitudinal bars in the shear span.

- Often this critical shear crack occurs where *bottom bars are terminated* in the shear span. Additional shear reinforcement may be required in this region (Clause 8.1.10.5 – AS3600-2009).
Detailing of beams (ctd):

Crack control provided by shear reinforcement (Leonhardt et al):

![Graph showing crack width vs. load for different beam reinforcement setups. The graph plots maximum crack width in millimeters against load in kilonewtons (kN). Four beam configurations are shown, each with a different level of shear reinforcement.](image)
Detailing of beams (ctd):

Support and Loading Points:

- When the support is at the soffit of a beam or slab, the diagonal compression passes directly into the support as shown.
- When the support is at the top of the beam, the diagonal compression must be carried back up to the support via an internal tie.
- It is essential that adequately anchored reinforcement be included to act as the tension tie and the reinforcement must be anchored into the support.
Detailing of beams (ctd):

Slab supported by upturned beam:

- The vertical component of the diagonal compression in the slab (i.e. the reaction from the slab) must be carried in tension up to the top of the upturned beam.
- Don’t rely on the unreinforced surface to carry this tension.
Detailing of beams (ctd):

Beam-to-beam connection:

- The area of additional suspension reinforcement is:
  \[ A_{sr} = \frac{R^*}{\phi f_{sy}} \]

- The diagram shows:
  - (a) Section
  - (b) Primary girder - Elevation
  - (c) Primary girder – Truss analogy
When a load is applied to the underside of a reinforced concrete beam, some device must be used to transfer this hanging load to the top of the beam.
Detailing of beams (ctd):

Half-Joint or dapped-end joint:

(a) Half joint

(b) Strut and tie model

(c) Alternative strut and tie model

Suspension reinforcement

Reinforcement detail
Detailing for Crack Control

- Excessive cracking due to restrained deformation or external loads is a common cause of damage in reinforced concrete structures.

- Shrinkage causes a gradual widening of existing cracks and time-dependent cracking in previously uncracked regions.

- Excessive cracking in the hardened concrete can be avoided by including sufficient reinforcement at sufficiently close spacings.

- The minimum quantities of reinforcement specified for crack control in AS3600 may not be what is actually required in all circumstances.
• The width of a crack depends on
  - the quantity, orientation and distribution of the reinforcing steel crossing the crack;
  - concrete cover and member size;
  - the bond between concrete and reinforcement in the vicinity of the crack;
  - the deformation characteristics of concrete; and
  - the shrinkage strain (and therefore the time after crack formation).
  - the cause of the crack
  - the degree of restraint

• Often significantly more reinforcement than the minimum specified amount is required.
Crack spacing, $s$, varies between $0.5d$ and $1.5d$ and depends on:
- steel area and distribution
- cover
and the average crack spacing decreases with time due to shrinkage.

Crack width, $w$, depends on:
- steel stress
- bar diameter and bar spacing
- cover
- adjacent crack spacings
and increases with time due to shrinkage.

Maximum crack widths increase with time by a factor of between 2 and 4.
Simplified Approach for Flexural Crack Control in AS3600-2009 (Clause 8.6.1 and 9.4.1):

For reinforced concrete beams and slabs, cracking is deemed to be controlled (crack widths will be less than 0.3 mm) if each of the following is satisfied:

(a) the quantity of tensile reinforcement in a beam or slab provides an ultimate strength at least 20% higher than the cracking moment calculated assuming $\sigma_{cs} = 0$;

(b) the distance from the side or soffit of the member to the centre of the nearest longitudinal bar shall not exceed 100 mm;

(c) The centre-to-centre spacing of bars near a tension face of a beam or slab shall not exceed 300 mm for a beam and the lesser of two times the slab thickness and 300 mm for a slab.

(d) The stress in the tensile steel is less than a limiting value (as follows):
Simplified Approach for Flexural Crack Control in AS3600-2009 (Clause 8.6.1 and 9.4.1): Ctd

For members subject primarily to flexure, the calculated steel stress caused by the serviceability design moment shall not exceed the larger of the maximum steel stresses given in Tables 8.6.1(A) and 8.6.1(B) for beams and Tables 9.4.1(A) and 9.4.1(B) for slabs.

**Table 8.6.1(A):** Maximum steel stress for tension or flexure in r.c. beams.

<table>
<thead>
<tr>
<th>Nominal bar diameter (mm)</th>
<th>Maximum steel stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>360</td>
</tr>
<tr>
<td>12</td>
<td>330</td>
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<tr>
<td>16</td>
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<td>160</td>
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<tr>
<td>36</td>
<td>140</td>
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<tr>
<td>40</td>
<td>120</td>
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</tbody>
</table>

**Table 8.6.1(B):** Maximum steel stress for flexure in r.c. beams.

<table>
<thead>
<tr>
<th>Centre-to-centre spacing (mm)</th>
<th>Maximum steel stress (MPa)</th>
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</thead>
<tbody>
<tr>
<td>50</td>
<td>360</td>
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<tr>
<td>100</td>
<td>320</td>
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<tr>
<td>150</td>
<td>280</td>
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<td>200</td>
<td>240</td>
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<tr>
<td>250</td>
<td>200</td>
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<tr>
<td>300</td>
<td>160</td>
</tr>
</tbody>
</table>
Simplified Approach for Flexural Crack Control in AS3600-2009 (Clause 8.6.1 and 9.4.1): Ctd

For members subject primarily to tension, the calculated steel stress caused by the serviceability design actions shall not exceed the maximum steel stresses given in Tables 8.6.1(A) for beams and Tables 9.4.1(A) for slabs.

**Table 9.4.1(A):** Maximum steel stress for flexure in r.c. slabs.

<table>
<thead>
<tr>
<th>Nominal bar diameter (mm)</th>
<th>Maximum steel stress (MPa) for overall depth $D_s$ (mm)</th>
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<tbody>
<tr>
<td></td>
<td>$\leq 300$</td>
<td>$&gt; 300$</td>
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<tr>
<td>6</td>
<td>375</td>
<td>450</td>
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<td>8</td>
<td>345</td>
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<td>24</td>
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<td>210</td>
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</tbody>
</table>

**Table 9.4.1(B):** Maximum steel stress for flexure in r.c. slabs.

<table>
<thead>
<tr>
<th>Centre-to-centre spacing (mm)</th>
<th>Maximum steel stress (MPa)</th>
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</thead>
<tbody>
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<td>50</td>
<td>360</td>
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<tr>
<td>100</td>
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<td>250</td>
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<td>300</td>
<td>160</td>
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</table>
**Restrained Shrinkage Cracking in Slabs:**

- Provided that bonded reinforcement at reasonable spacing crosses the crack and that the member does not deflect excessively, flexural cracks are usually well controlled in reinforced concrete beams and slabs.

- In contrast, direct tension cracks due to restrained shrinkage and temperature changes frequently lead to serviceability problems, particularly in regions of low moment.

- Such cracks usually extend completely through the member and are more parallel sided than flexural cracks.

- If uncontrolled, these cracks can become very wide and lead to waterproofing and corrosion problems.

- They can also disrupt the integrity and the structural action of the slab.
The slab is restrained by beams and shrinkage induces tension in the slab in the direction of the beams.

- Full depth restrained shrinkage cracks
- Flexural cracks

One-way floor slab supported on beams

- Usually more steel is required to control the restrained shrinkage cracks than is required to control the flexural cracks and provide adequate strength.
Restrained Shrinkage Cracking in Slabs Ctd:

- In the **primary direction**, shrinkage will cause small increases in the widths of the many fine flexural cracks and may cause additional flexure type cracks in the previously uncracked regions.

- However, in the **secondary direction**, which is in effect a direct tension situation, shrinkage generally causes a few widely spaced cracks which penetrate completely through the slab.

- If the amount of reinforcement crossing a direct tension crack is small, yielding of the steel will occur and a wide, unserviceable crack will result. To avoid this eventuality, the **minimum steel ratio**, \( \rho_{\text{min}} \) is

\[
\rho_{\text{min}} = \left( \frac{A_{\text{st}}}{b d} \right)_{\text{min}} = \frac{1.2 f_{\text{ct}}}{f_{\text{sy}}}
\]

where \( f_{\text{ct}} = 0.25 \sqrt{f_c^*} \). For 32 MPa concrete, \( \rho_{\text{min}} = 0.0034 \).

- For a serviceable crack width, **significantly more steel than this** is required.
Crack Control in Slabs –AS3600-2009:

- Where the ends of a slab are restrained and the slab is not free to expand or contract in the secondary direction, the minimum area of reinforcement in the restrained direction is given by either Eq. 1a, 1b or 1c, as appropriate (see below).

- For a slab fully enclosed within a building except for a brief period of weather exposure during construction:

  (i) where a strong degree of control over cracking is required:

  \[
  (A_s)_{\text{min}} = (6.0 - 2.5\sigma_{cp})b \times D \times 10^{-3} \quad (1a)
  \]

  (ii) where a moderate degree of control over cracking is required:

  \[
  (A_s)_{\text{min}} = (3.5 - 2.5\sigma_{cp})b \times D \times 10^{-3} \quad (1b)
  \]

  (iii) where a minor degree of control over cracking is required:

  \[
  (A_s)_{\text{min}} = (1.75 - 2.5\sigma_{cp})b \times D \times 10^{-3} \quad (1c)
  \]
For all other slab surface conditions in Exposure Classification A1 and for exposure classification A2, Eq. 1a applies where a strong degree of control over cracking is required for appearance or where cracks may reflect through finishes

\[(A_s)_{\text{min}} = (6.0 - 2.5\sigma_{cp})b\ D\times10^{-3}\]  

(1a)

and Eq. 1b applies where a moderate degree of control over cracking is required and where cracks are inconsequential or hidden from view.

\[(A_s)_{\text{min}} = (3.5 - 2.5\sigma_{cp})b\ D\times10^{-3}\]  

(1b)

For Exposure Classifications B1, B2, C1 and C2, Eq. 1a always applies.

The minimum steel area given by Eq. 1c is appropriate in an unrestrained direction where the slab is free to expand or contract.

In the primary direction of a one-way slab or in each direction of a two-way slab, the minimum quantity of reinforcement is the greater of the minimum quantity required for the strength limit state or 75% of the minimum area required by Eqs. 1a, 1b or 1c, as appropriate.
Consider a slab restrained at each end. With time, restrained shrinkage cracks occur at roughly regular centres depending on the amount of reinforcement:

(a) Portion of restrained member after all cracking

(b) Average concrete stress after all shrinkage

(c) Steel stress after all shrinkage cracking
Typical values:

Consider a 140 mm thick, 4m long slab fully-restrained at both ends and symmetrically reinforced with N12 bars at 250 mm centres top and bottom. Hence, $A_s = 900 \text{ mm}^2/\text{m}$ and $\rho = A_s/A_c = 0.00643$.

For 25 MPa concrete with a final shrinkage strain of 0.0007 and typical material properties, a shrinkage cracking analysis of this restrained slab indicates 4 or 5 full depth cracks within the 4 m length with the maximum final crack width about 0.3 mm.
If $p = 0$:

- **$p = A_s/A_c$**
  - $140$ mm
  - $4$ m

One large unserviceable crack

If $p = 0.0035$:

- Approx. $0.6 - 0.7$ mm

About three unserviceable (?) cracks

If $p = 0.006$:

- Approx. $0.3 - 0.4$ mm

Four or five serviceable (?) cracks
Detailing of columns:

Lapped compressive splices:

Normal fitment spacing, $s$

Additional fitment spacing, $s' < 4c$

Additional fitments at compressive splice
Detailing of columns:

Typical tie arrangements in columns:

All longitudinal bars in these columns are restrained at
(i) a bend in a fitment of 135° or less; or
(ii) at a fitment hooks with included angle of 135° or less, as shown.

AS3600-2009 requirements for restraining single longitudinal bars in columns:
(i) Every corner bar;
(ii) All bars - where bars are spaced at centres > 150 mm;
(iii) At least every alternate bar - where bar centres ≤ 150mm.

For bundled bars – each bundle must be restrained.
Detailing of columns:

Minimum bar diameters for fitments (AS3600-2009):

<table>
<thead>
<tr>
<th>Longitudinal bar diameter (mm)</th>
<th>Minimum bar diameter for fitment and helix (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Single bars up to 20</td>
<td>6</td>
</tr>
<tr>
<td>Single bars 24 to 28</td>
<td>10</td>
</tr>
<tr>
<td>Single bars 28 to 36</td>
<td>12</td>
</tr>
<tr>
<td>Single bar 40</td>
<td>16</td>
</tr>
<tr>
<td>Bundled bars</td>
<td>12</td>
</tr>
</tbody>
</table>

Maximum spacing of fitments (AS3600-2009):

The spacing of fitments (or the pitch of a helix) should not exceed the smaller of:

\[ D_c \text{ and } 15d_b \text{ for single bars} \]
\[ 0.5D_c \text{ and } 7.5d_b \text{ for bundled bars} \]
Detailing of Beam-column Connections:

Knee Connections (or two-member connections):

(a)                                      (b)                                          (c)                                          (d)

Figure 8.37  Two-member connections.
Detailing of Beam-column Connections:

Knee Connections under “Opening” Moment:

(a) Internal forces
(b) Crack pattern

(c) Potentially satisfactory

\[ A_{sv} = \frac{\sqrt{2} T}{\phi f_{sy} f} = \frac{\sqrt{2} A_{st} f_{sy}}{\phi f_{sy} f} \]
Detailing of Beam-column Connections:

Knee Connections under “Opening” Moment – Suggested detail:

- Diagonal stirrups
- Diagonal flexural bars
Detailing of Beam-column Connections:

Knee Connections under “Closing” Moment:

(a) Internal forces  (b) Crack pattern

(a) Wall or slab connection (when \( p \leq f_{ct,f}/f_{sy} \))  (b) Beam to column knee connection
Detailing of Beam-column Connections:

Three-member connections:

(a) Internal forces
(b) Crack pattern

High bond stress

Poor anchorage conditions
Detailing of Beam-column Connections:

Three-member connections – Reinforcement detail:

- Larger diameter bar to distribute bearing stresses in bend
- Ties to carry diagonal tension, to control vertical splitting and to confine the concrete core
Detailing of Beam-column Connections:

Four-member connections:

(a) Internal forces
(b) Crack pattern
(c) Reinforcement detail
Detailing of Corbels:

(a) Strut-and-tie action

(b) Reinforcement detail

(c) Welded primary steel

(d) Satisfactory weld details (17)
Design of Corbels:

Tie: \[ T = \frac{V^*}{\tan \theta} = \phi \ A_s \ f_{sy} \] \quad (\phi = 0.8)

\[ \therefore \ A_s = \frac{V^*}{\phi \ f_{sy} \ \tan \theta} \]

Strut: \[ \phi_{st} \ C_u = \phi_{si} \ \beta_s \ 0.9 \ f'_{c} \ A_c \] \quad (\phi_{st} = 0.6)

\[ \beta_s = \frac{1}{1.0 + 0.66 \cot^2 \ \theta} \] \quad (0.3 \leq \beta_s \leq 1.0)

\[ A_h \geq A_s / 2 \]

Park and Paulay suggest that a good first estimate of corbel dimensions is obtained from:

\[ \frac{V^*}{b_w \ d} \leq 0.56 \sqrt{f'_{c}} \]

and ACI318-08 suggests

\[ \frac{a}{d} \leq 1.0 \quad \text{and} \quad 0.04 \ f'_{c} / f_{sy} \leq A_s / b_w \ d \leq 0.2 \ f'_{c} / f_{sy} \]
Design of Corbels:

\[ V^* = 500\text{kN} \]

\[ d = \frac{D}{2} \]

\[ d = \frac{200}{\sin\theta} = 270\text{ mm} \]

\[ b_w = 300\text{ mm} \]

\[ f'_c = 32\text{ MPa} \quad f_{sy} = 500\text{ MPa} \quad \text{Cover} = 30\text{ mm} \]

Bearing plate = 200 x 300 mm in plan
Design of Corbels:

From \( V^* / b_w d \leq 0.56 \sqrt{f_c'} \):
\[ d \geq \frac{500 \times 10^3}{300 \times 0.56 \times \sqrt{32}} = 526 \text{ mm} \]

With \( D = d + \text{cover} + 0.5 \text{ bar dia} \) and assuming 20 mm diameter bars, take \( D = 570 \text{ mm} \) and therefore \( d = 530 \text{ mm} \).

From the geometry:
\[ \tan \theta = \frac{d - 100 \tan(90 - \theta)}{400} \]
\[ \therefore \theta = 47.7^\circ \]

and
\[ A_s = \frac{500 \times 10^3}{0.8 \times 500 \times \tan 47.7} = 1138 \text{ mm}^2 \]
Try 4 N20 bars (1240 mm²)

Now
\[ A_s / b_w d = 0.0078 = 0.122 f_c' / f_{sy} \quad \therefore \text{OK} \]

The strut efficiency factor:
\[ \beta_s = \frac{1}{1.0 + 0.66 \cot^2 \theta} = 0.65 \]

and
\[ \phi_{st} C_u = 0.6 \times 0.65 \times 0.9 \times 32 \times 81150 = 911 \text{ kN} \]
\[ > C^* = V^* / \cos \theta = 743 \text{ kN} \quad \therefore \text{OK} \]
Design of Corbels:

- 4 N20 stirrups
- 3 N12 stirrups
- N24 cross-bar (welded to N20s)
- N28 welded cross-bar

ELEVATION

PLAN
JOINTS IN STRUCTURES:

Joints are introduced into concrete structures for **two main reasons:**

1) **As stopping places** in the concreting operation. The location of these *construction joints* depends on the size and production capacity of the construction site and work force;

2) To **accommodate deformation** (expansion, contraction, rotation, settlement) without local distress or loss of integrity of the structure. Such joints include:

   - *control joints* (contraction joints);
   - *expansion joints*;
   - *structural joints* (such as hinges, pin and roller joints);
   - *shrinkage strips*; and
   - *isolation joints*.

The location of these joints depends on the anticipated movements of the structure during its lifetime and the resulting effects on structural behaviour.
Construction Joints:

(a) Butt joint

Waterstop where water tightness is required

(b) Keyed joint

1\textsuperscript{st} pour

2\textsuperscript{nd} pour

Steel dowels to improve shear strength

(c) Doweled joint
Control Joints (or Contraction Joints):

(a) Saw-cut joint in slab on ground

(b) Wall \( t < 200 \text{ mm} \)

(c) Doweled joint

(d) Wall \( t \geq 200 \text{ mm} \)

- Saw cut > 0.2 \( t \) and 20 mm
- \( t \approx 0.75 t \) Debond dowel to ensure free contraction
- Discontinue every second bar if necessary so that \( p < 0.002 \)
Typical control joint locations:

(a) Wall elevation

(b) Balcony plan
Alternative shrinkage strip details:
Expansion joint details:

(a) Double column and beams

(b) Half joint

(c) Building plans – joint locations

\[ \geq 25 \text{ mm} \]
Alternative structural hinge joints at base of a column:

- Elastic, easily compressible material
- Confinement steel
- Mesnager hinge
THANKS FOR YOUR ATTENTION

ARE THERE ANY QUESTIONS?