

THE CIVIL & STRUCTURAL ENGINEERING PANEL
ENGINEERS AUSTRALIA SYDNEY DIVISION

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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:

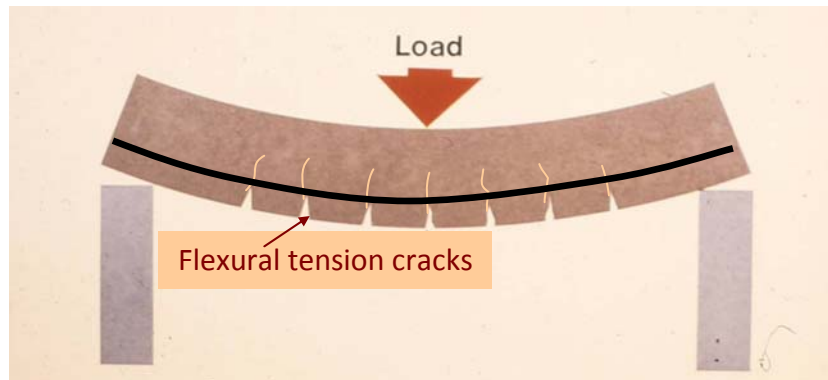
1. To carry internal tensile forces, thereby **imparting strength and ductility**;
2. To control **flexural cracking**;
3. To control **direct tension cracking** in restrained structures;
4. To carry **compressive forces**;
5. To provide **restraint** to bars in compression;
6. To provide **confinement** 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:

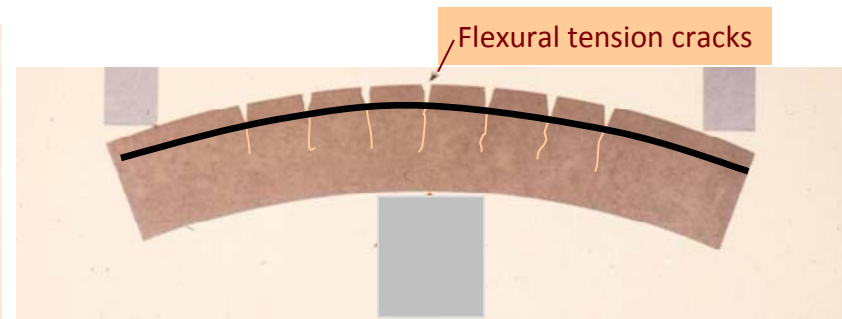
- Determine location and direction of **all internal forces** (i.e. establish a **load path** that satisfies equilibrium);
- Use **adequately anchored** reinforcement wherever a tensile force is required for equilibrium;
- Use only **ductile reinforcement** (Class N or better) when the reinforcement 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:

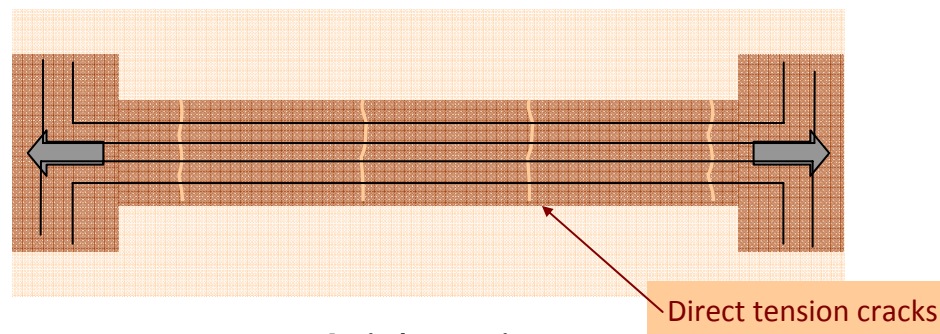
1. Tension caused by bending (and axial tension):



Positive bending



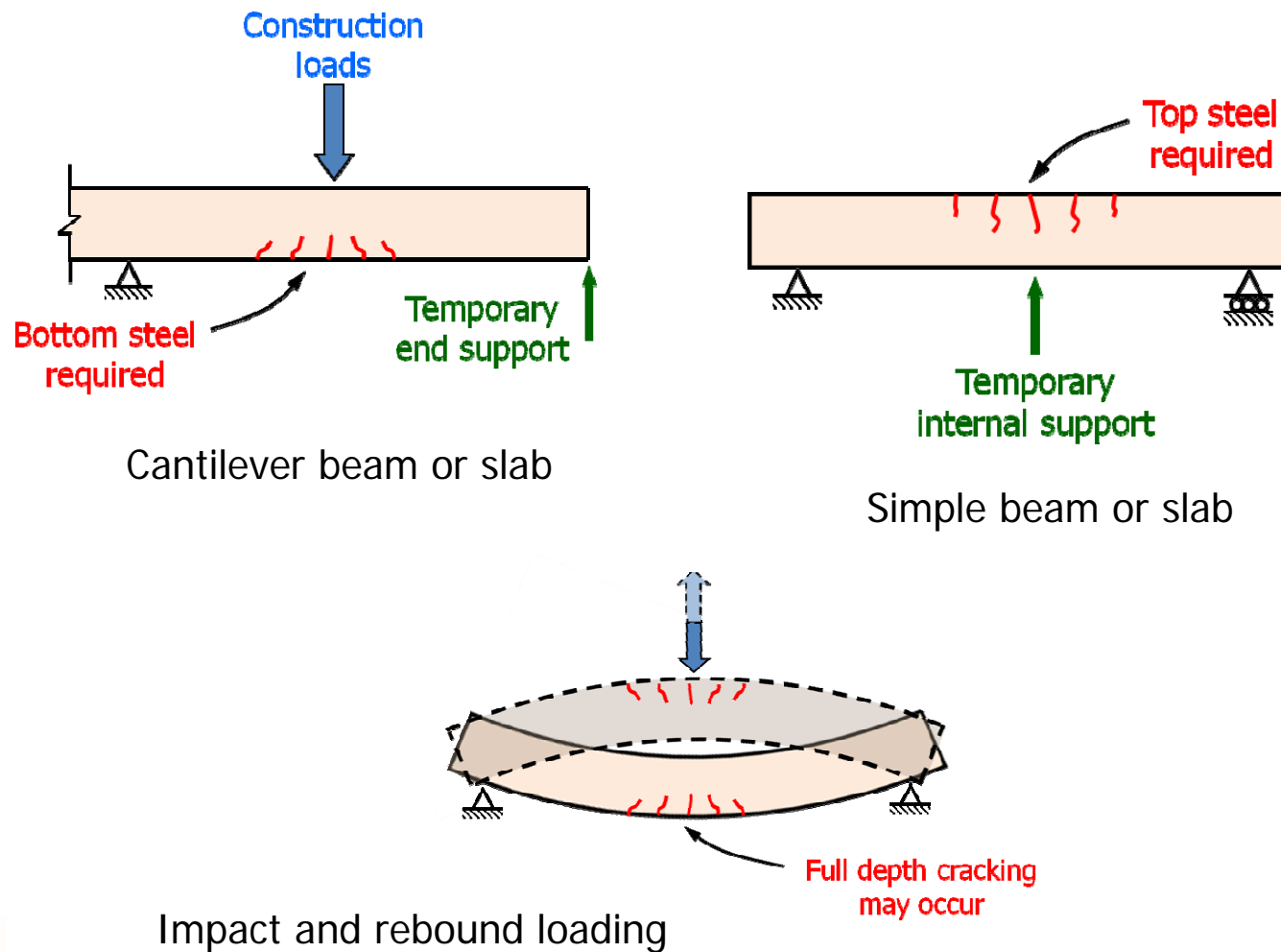
Negative bending



Axial tension

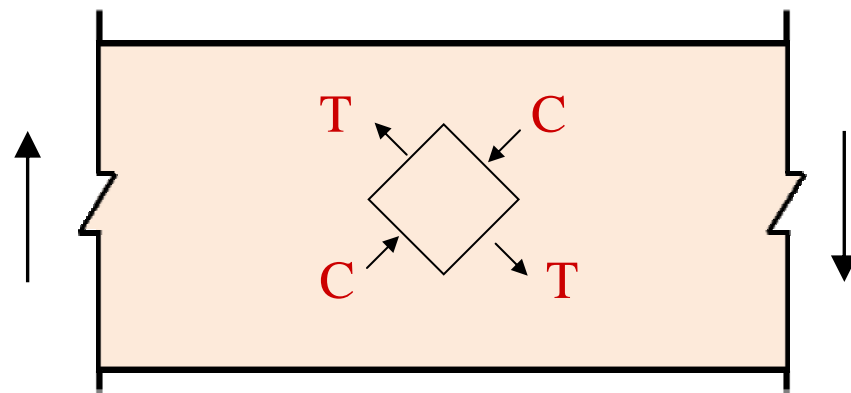
Sources of tension:

2. Tension caused by load reversals:

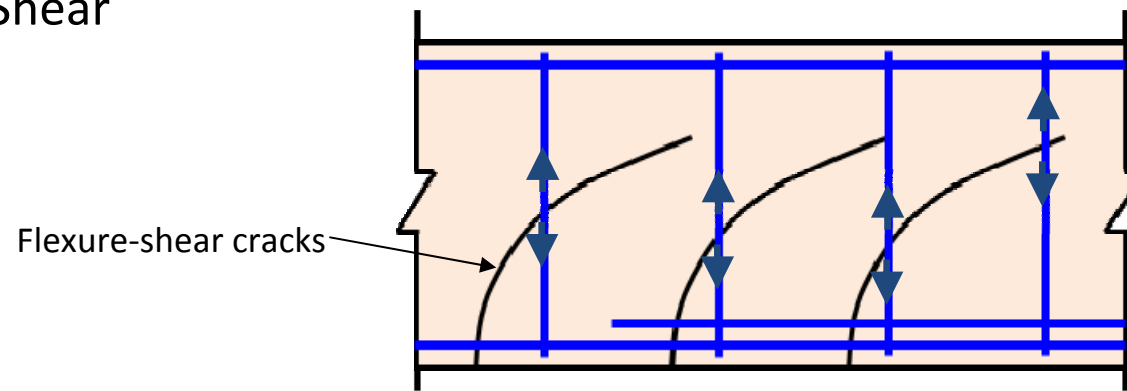


Sources of tension (ctd):

3. Tension caused by shear and torsion:



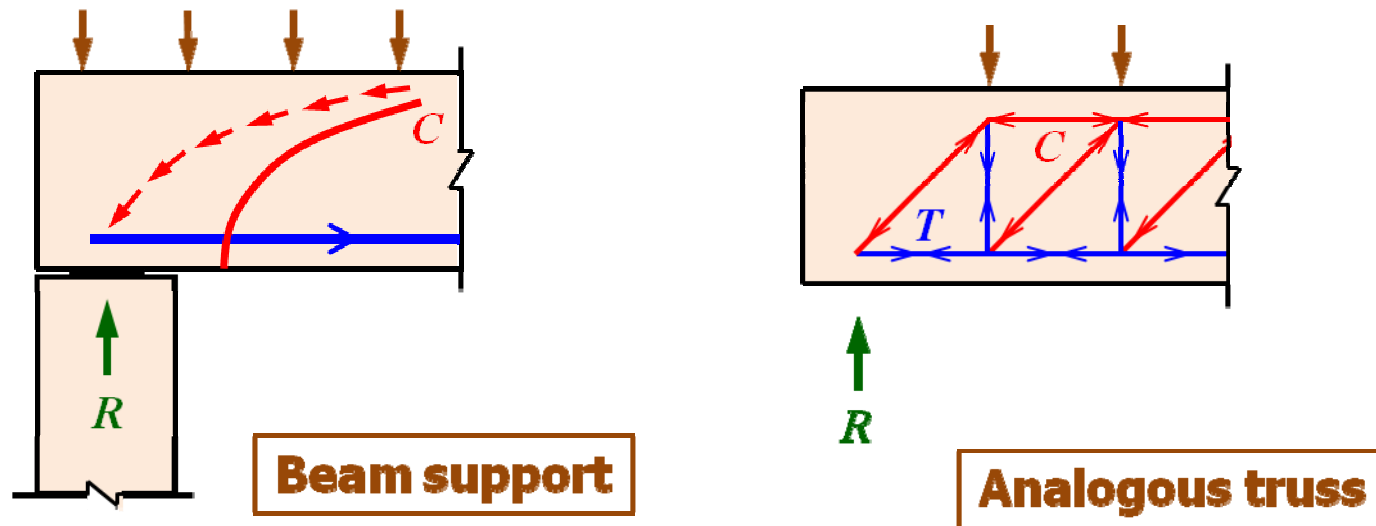
Shear



Tension carried by stirrups

Sources of tension (ctd):

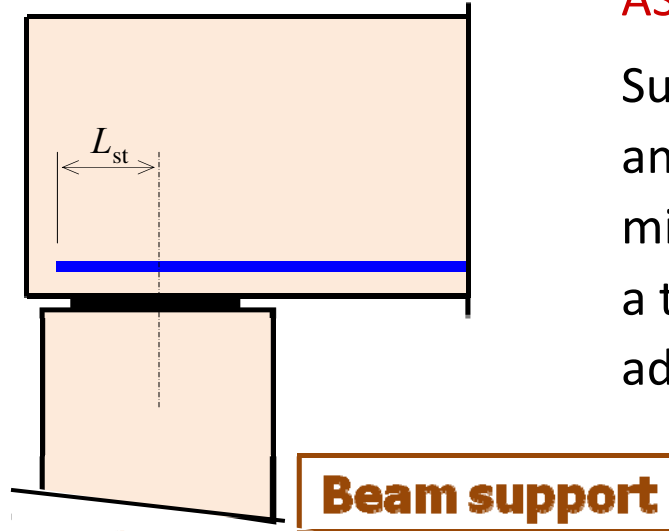
4. Tension near the supports of beams:



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 $V \cot \theta_v / \phi$ (plus any additional force arising from restraint)

This requirement is deemed to be satisfied if either

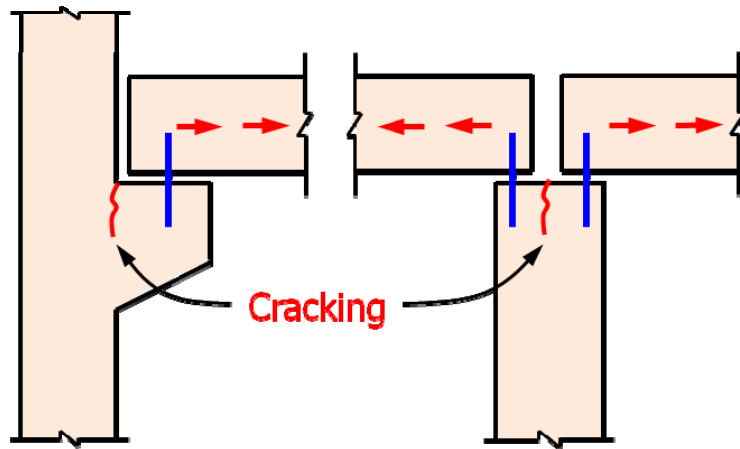
$\geq \frac{1}{2} A_{st}$ is extended past the face of the support by $\geq 12d_b$; or

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

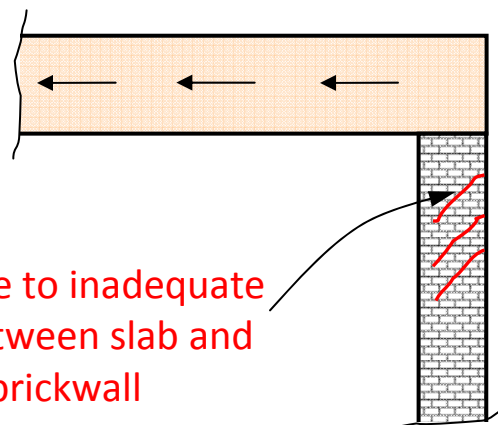
where A_{st} is the tensile steel area required at mid-span

Sources of tension (ctd):

5. Tension within the supports of beams and slabs:



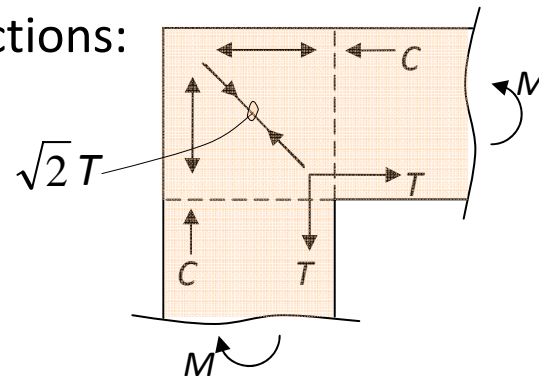
- Shortening of beams due to shrinkage can cause tension and subsequent cracking in the supports if longitudinal movement is restrained



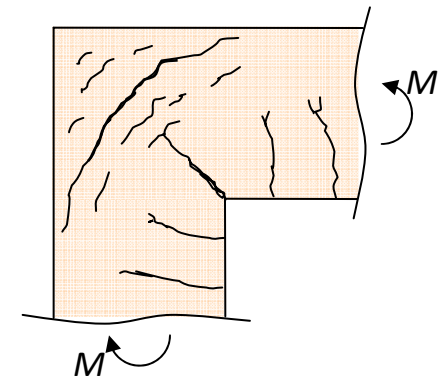
- The problem may be overcome by the introduction of suitable contraction joints

Sources of tension (ctd):

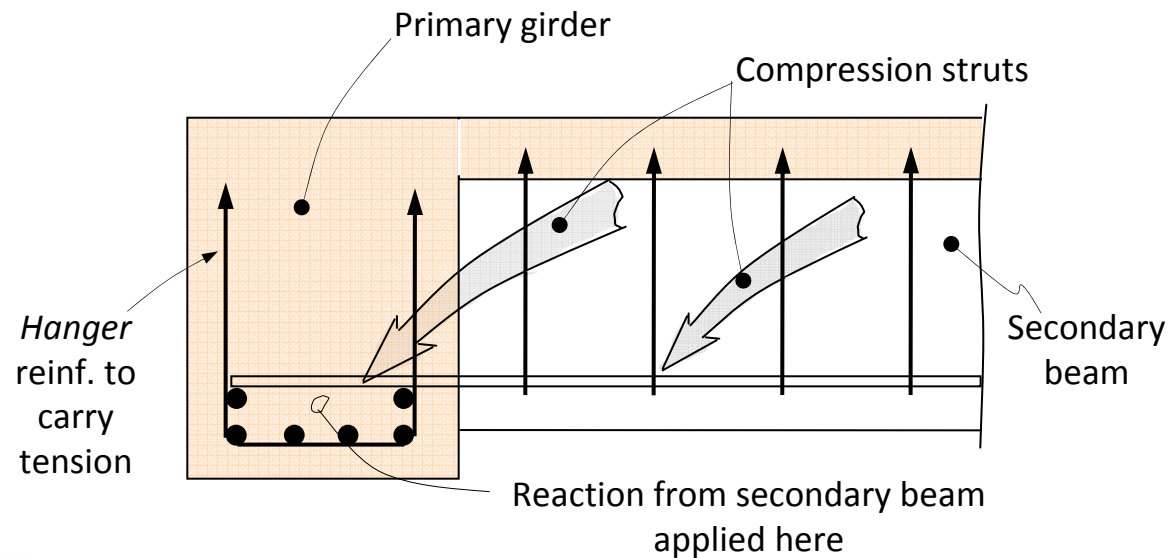
6. Tension within connections:



(a) Internal forces

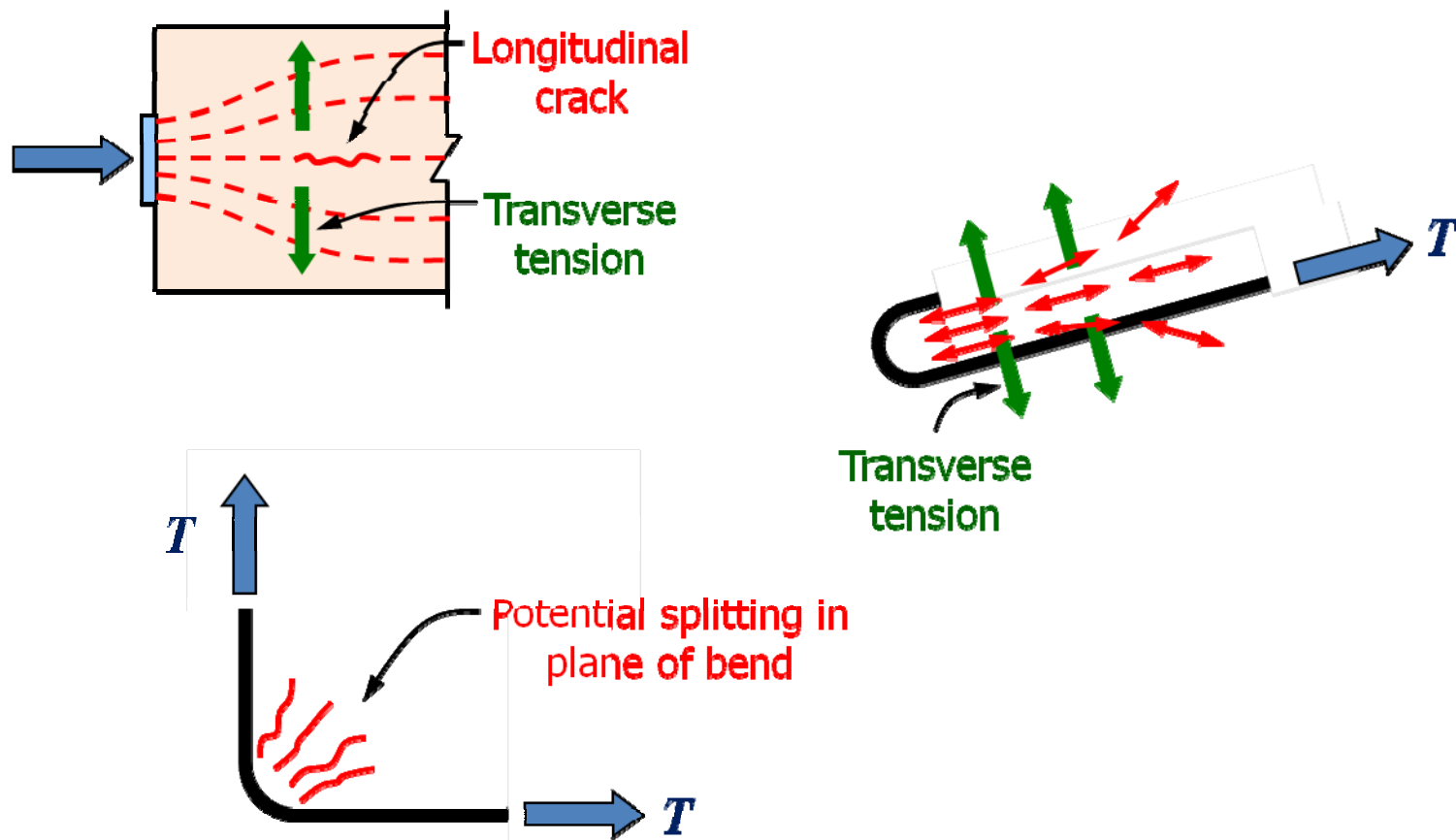


(b) Crack pattern



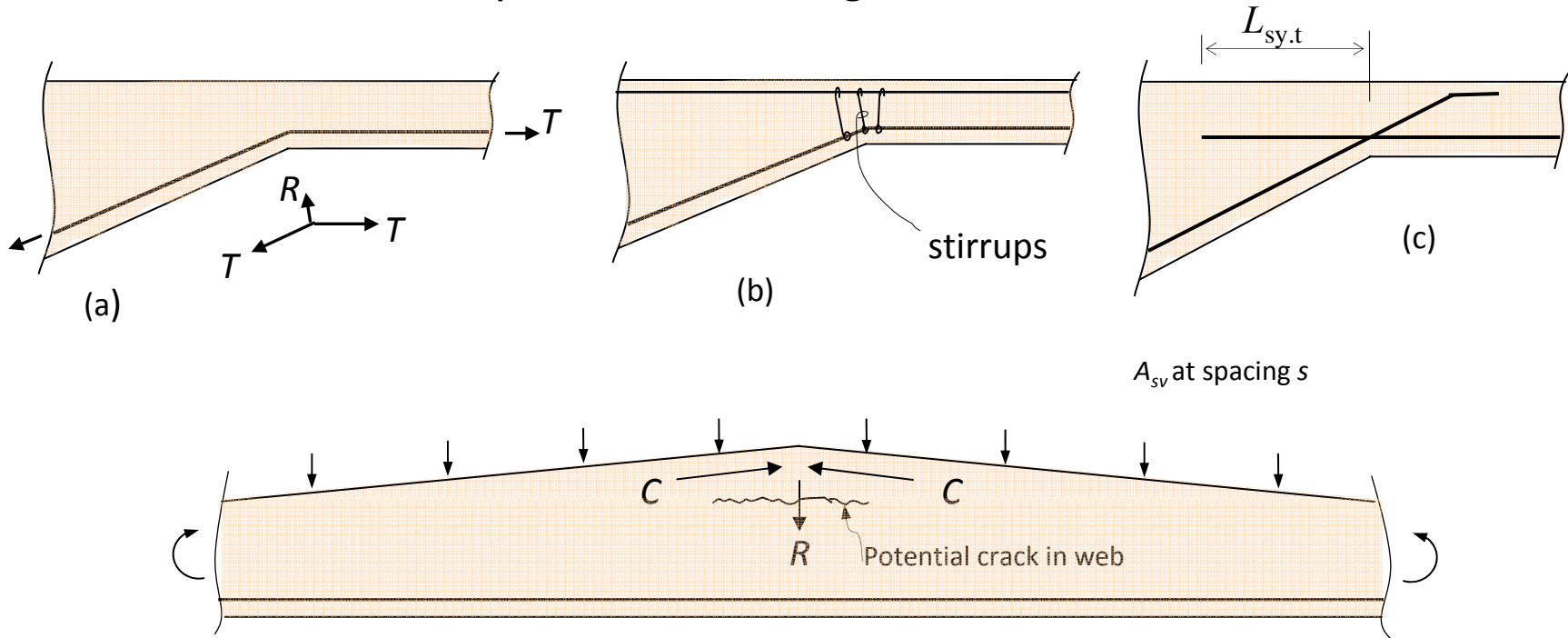
Sources of tension (ctd):

7. Tension at concentrated loads:



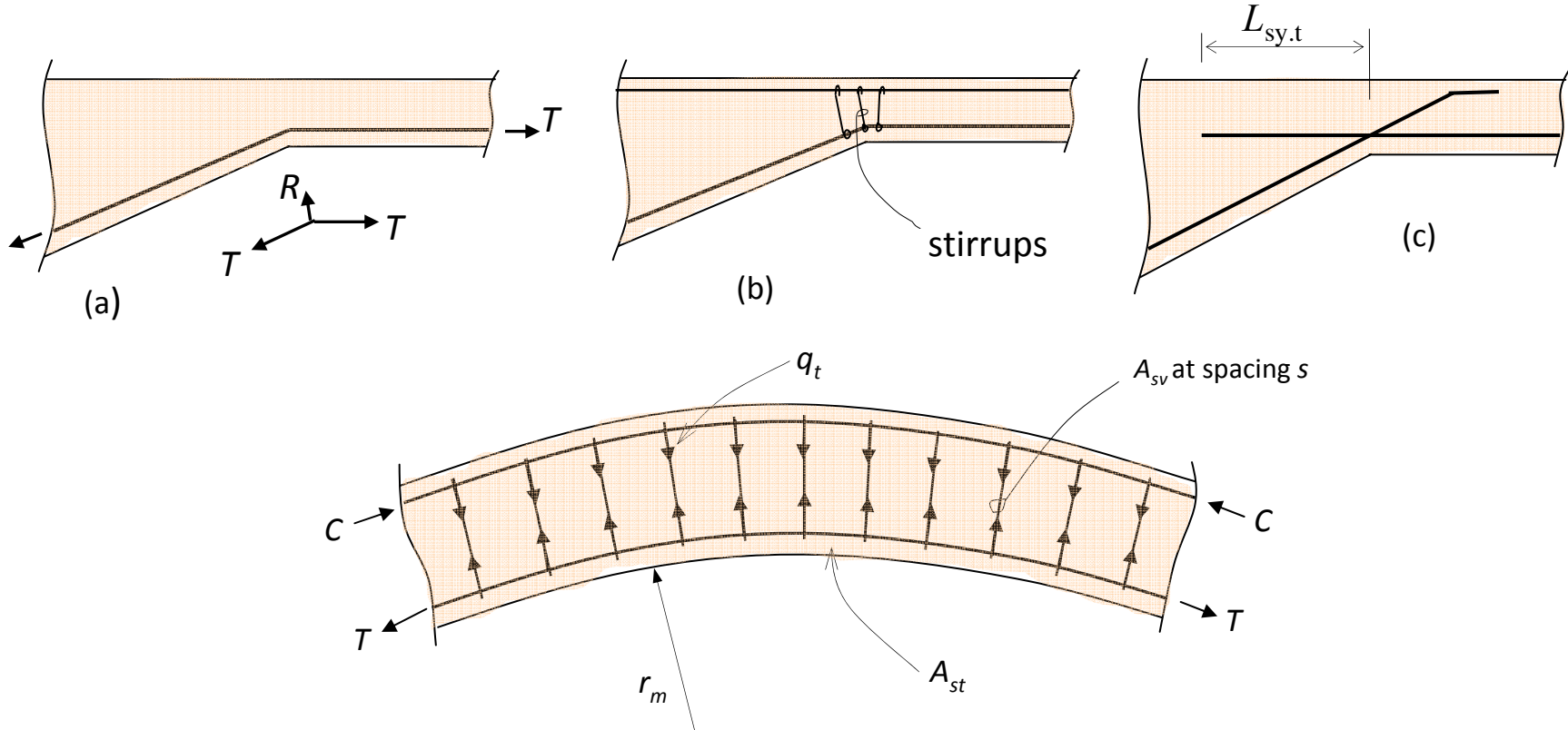
Sources of tension (ctd):

8. Tension caused by directional changes of internal forces:



Sources of tension (ctd):

8. Tension caused by directional changes of internal forces:



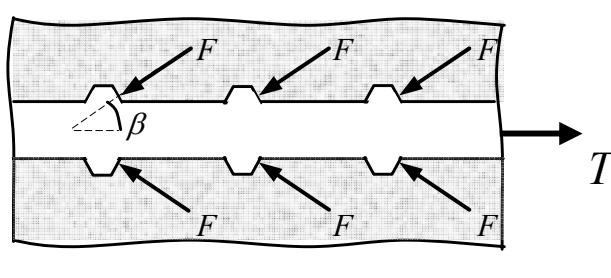
Transverse tension:

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

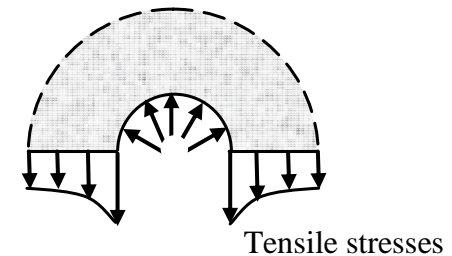
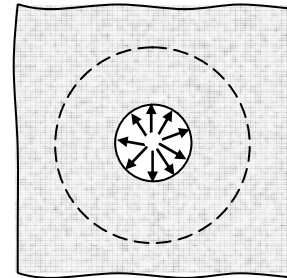
Required stirrup spacing:

$$s = \frac{A_{sv} f_{vy}}{q_t} = \frac{A_{sv}}{A_{st}} \cdot \frac{f_{vy}}{f_{sy}} \cdot r_m$$

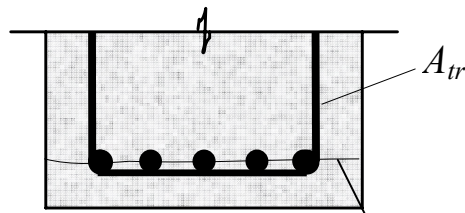
Anchorage of deformed bars is tension:



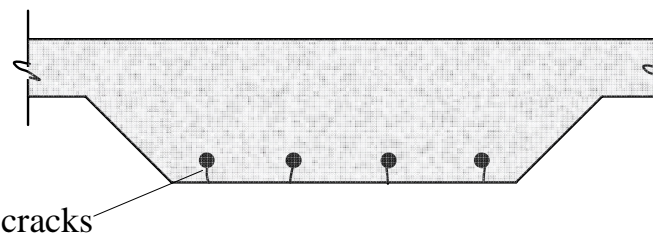
a) Forces exerted by concrete on a deformed bar at a tensile anchorage



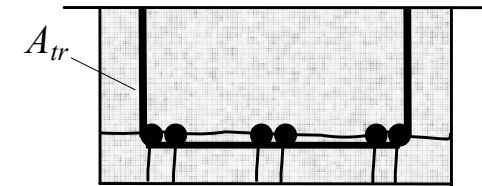
(b) Tensile stresses in concrete



(c) Horizontal splitting due to insufficient bar spacing.



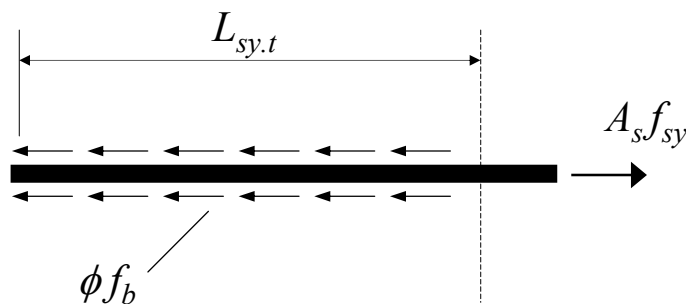
(d) Vertical splitting due to insufficient cover



(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 ϕf_b is assumed at the interface between the concrete and the reinforcing bar ($\phi = 0.6$).
- ϕf_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;

$$k_2 = (132 - d_b)/100 ;$$

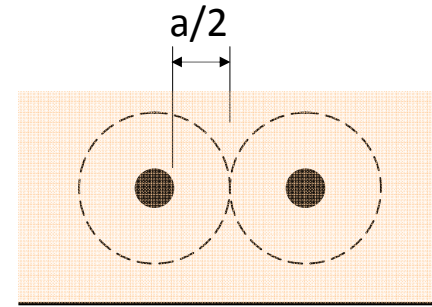
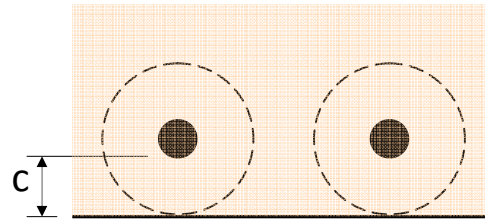
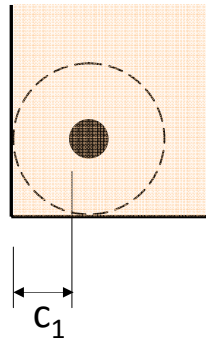
$$k_3 = 1.0 - 0.15(c_d - d_b)/d_b \quad (\text{but } 0.7 \leq k_3 \leq 1.0)$$

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

AS3600-2009

(§13.1.2.2)



$$c_d = \min(a/2, c, c_1)$$

AS3600-2009 ctd

(§13.1.2.3)

- 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

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

AS3600-2009 ctd

(§13.1.2.3)

$$k_4 = 1 - K\lambda$$

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

where

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

ΣA_{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:

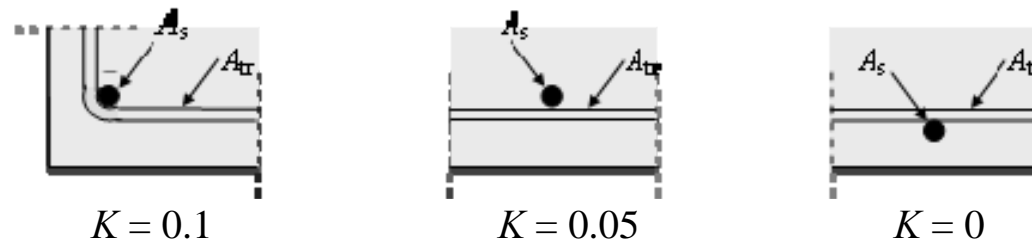


FIGURE 13.1.2.3(B) VALUES OF K FOR BEAMS AND SLABS

AS3600-2009 ctd

(§13.1.2.3)

- The **development length** L_{st} to develop a stress σ_{st} lower than f_{sy} :

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

When calculating σ_{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:

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

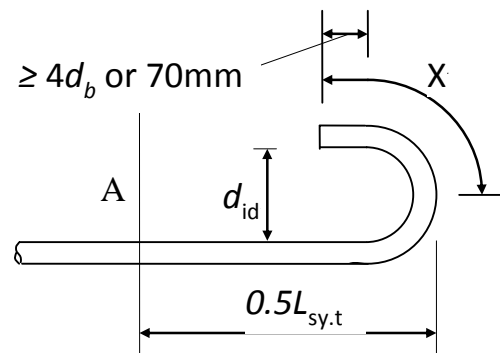
and therefore

$$\sigma_{st} \geq \frac{T^*}{\phi A_{st}}$$

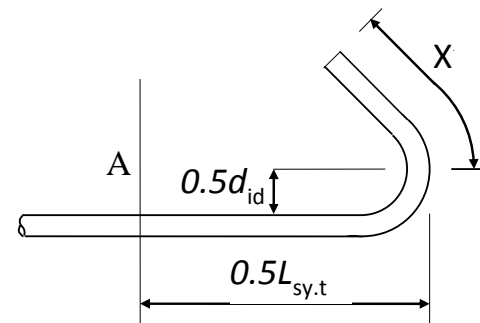
AS3600-2009 ctd

(§13.1.2.3)

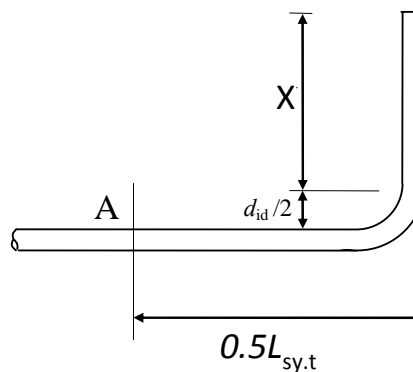
- The **development length** of a deformed bar with a standard hook or cog:



(a) Standard hook (180° bend)



(b) Standard hook (135° bend)



(c) Standard cog (90° bend)

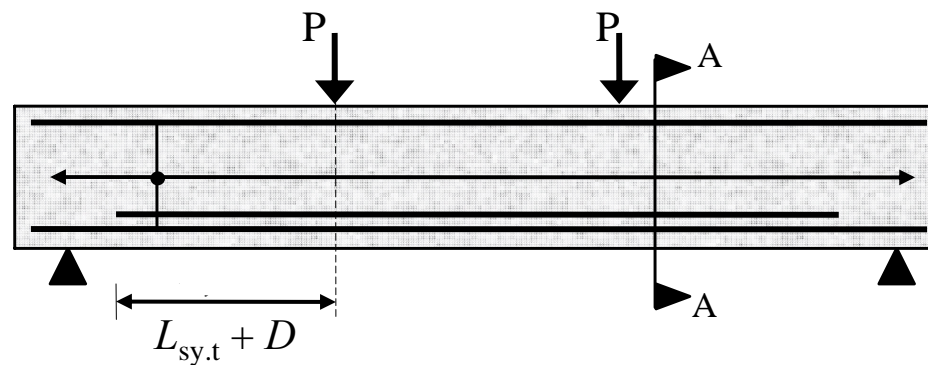
AS3600-2009

WORKED EXAMPLE:

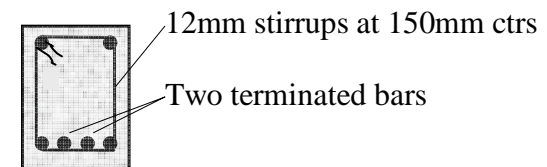
Consider the minimum **development length** required for the two terminated **28 mm diameter bottom bars** in the beam shown below.

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, $A_{tr} = 110 \text{ mm}^2$.



Elevation



Section A-A

Worked Example ctd

(§13.1.2.3)

$$L_{\text{sy.tb}} = \frac{0.5k_1k_3f_{\text{sy}}d_b}{k_2\sqrt{f'_c}}$$

For bottom bars: $k_1 = 1.0$;

For 28 mm diameter bars: $k_2 = (132 - 28)/100 = 1.04$;

The concrete confinement dimension, $c_d = a/2 = 30$ mm, and therefore

$$k_3 = 1.0 - 0.15(30 - 28)/28 = 0.99$$

The basic development length is therefore

$$L_{\text{sy.tb}} = \frac{0.5 \times 1.0 \times 0.99 \times 500 \times 28}{1.04\sqrt{32}} = 1178 \text{ mm } (> 29k_1d_b)$$

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

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

$$\lambda = (770 - 155)/620 = 0.99$$

Worked Example ctd

(§13.1.2.3)

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

From Figure 13.1.2B, $K = 0.05$ (as it is the two interior bars that are being developed) and therefore

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

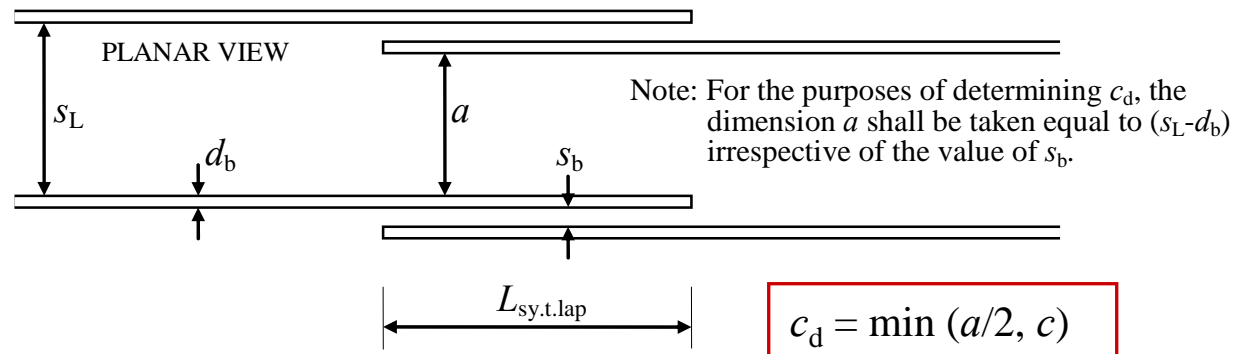
It is assumed that in this location the transverse pressure perpendicular to the anchored bar (ρ_p) is zero, and hence $k_5 = 1.0$.

From Eq. 13.1.2.3:

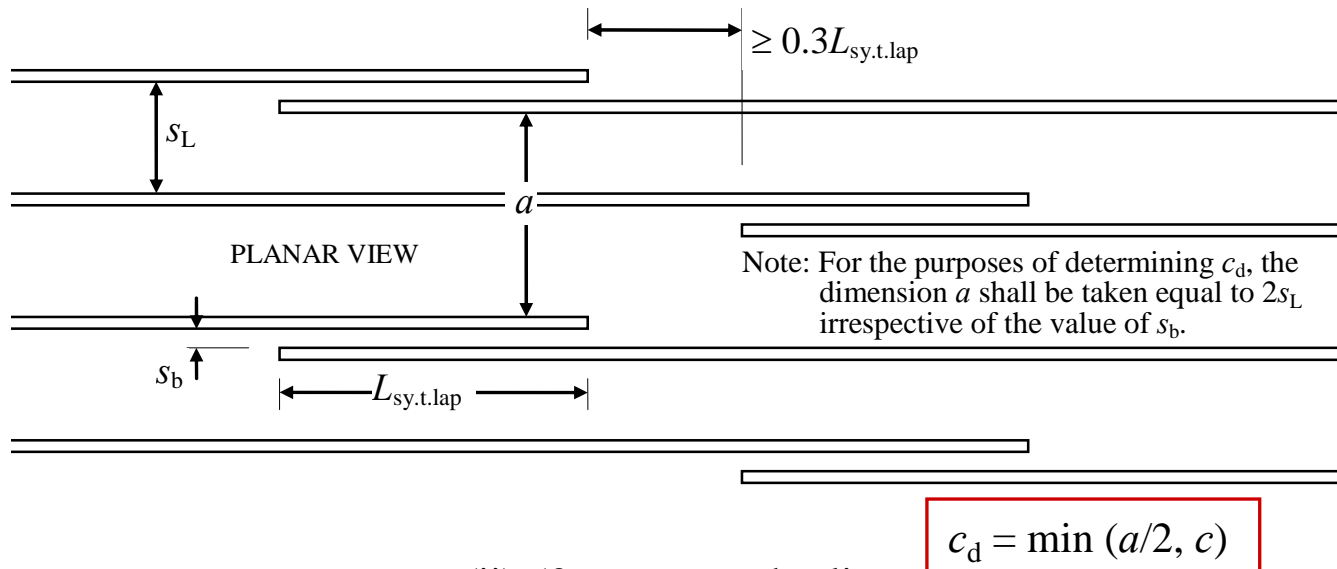
$$L_{sy.t} = k_4 k_5 L_{sy.bt} = 0.95 \times 1.0 \times 1178 = 1120 \text{ mm.}$$

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):



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



(ii) 50% staggered splices

AS3600-2009:

§13.2.2

Lapped Splices for bars in tension:

$$L_{\text{sy.t.lap}} = k_7 L_{\text{sy.t}} \geq 29k_1 d_b$$

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 ($L_{\text{sy.t.lap}}$) shall be not less than the larger of $k_7 L_{\text{sy.t}}$ and $L_{\text{sy.t}} + 1.5s_b$, 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:

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

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

$$= 563 \text{ mm} = 46.9 d_b$$

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

ACI 318-08: (Simplified – Clause 12.2.2)

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

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

$$= 743 \text{ mm} = 61.9 d_b$$

ACI 318-08: (Refined – Clause 12.2.3)

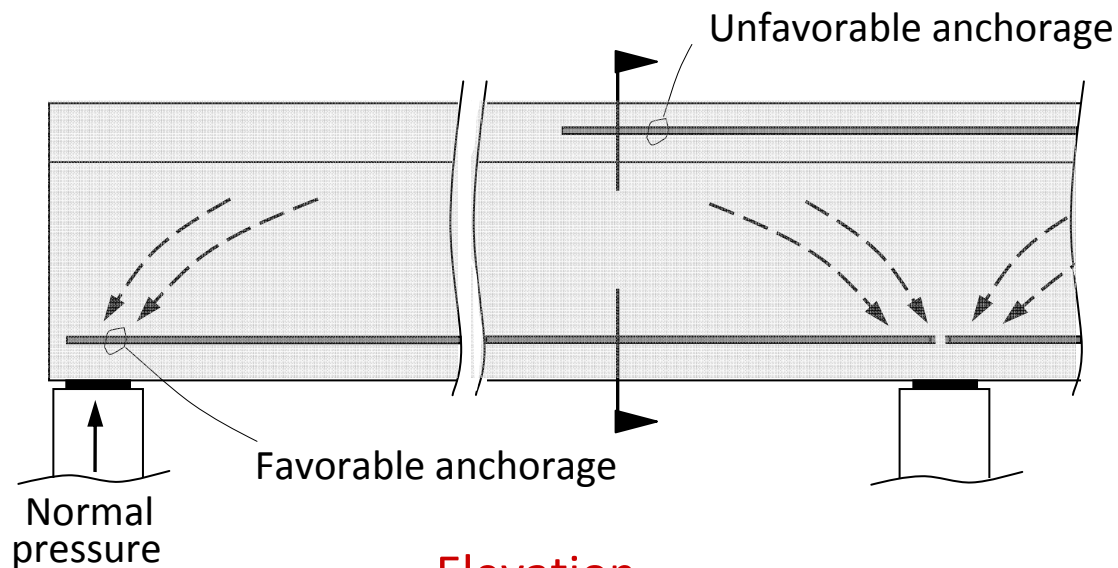
$$L_{\text{lap}} = 1.3\ell_d = 1.3 \frac{f_y\psi_t\psi_e\psi_s}{1.1\lambda\sqrt{f'_c} \left(\frac{c_b + K_{tr}}{d_b} \right)} d_b$$

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

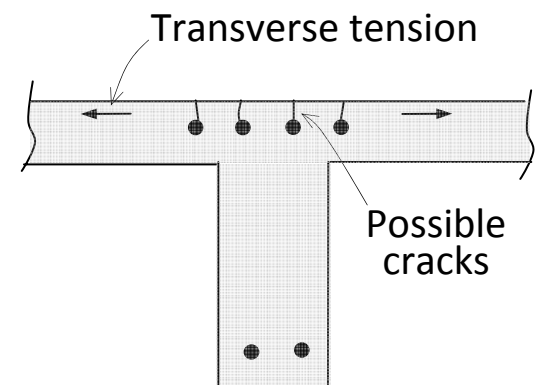
$$= 524 \text{ mm} = 43.7 d_b$$

Detailing of beams:

Anchorage of longitudinal reinforcement:

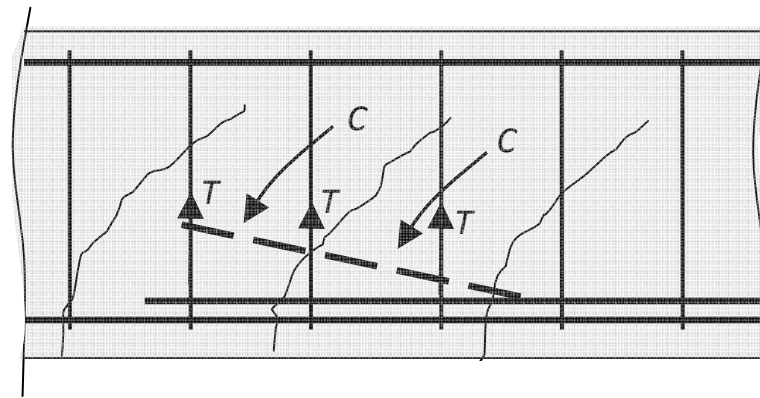


Elevation



Section

When bottom reinforcement is terminated away from the support, the diagonal compression in the web improves the anchorage.



Current wording:

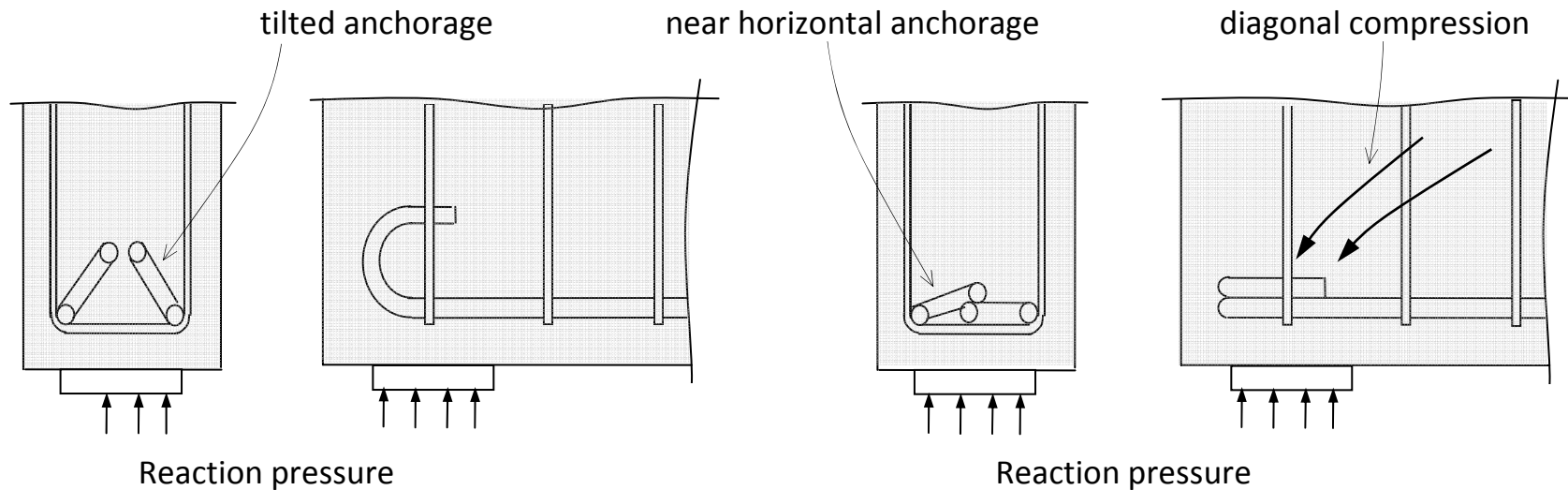
“The design for flexural strength and detailing of flexural reinforcement and pretensioned tendons at termination shall be extended from the theoretical cut-off point, or debonding point, by a length of $1.0D + L_{sy,t}$, or $1.0D + L_{pt}$, where D is the member depth at the theoretical cut-off point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

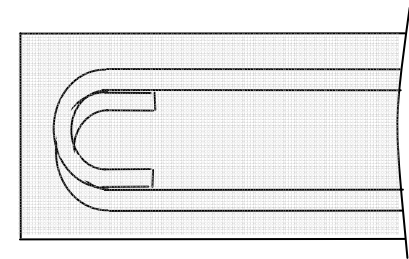
Amended wording:

“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 $1.0D + L_{st}$, 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):



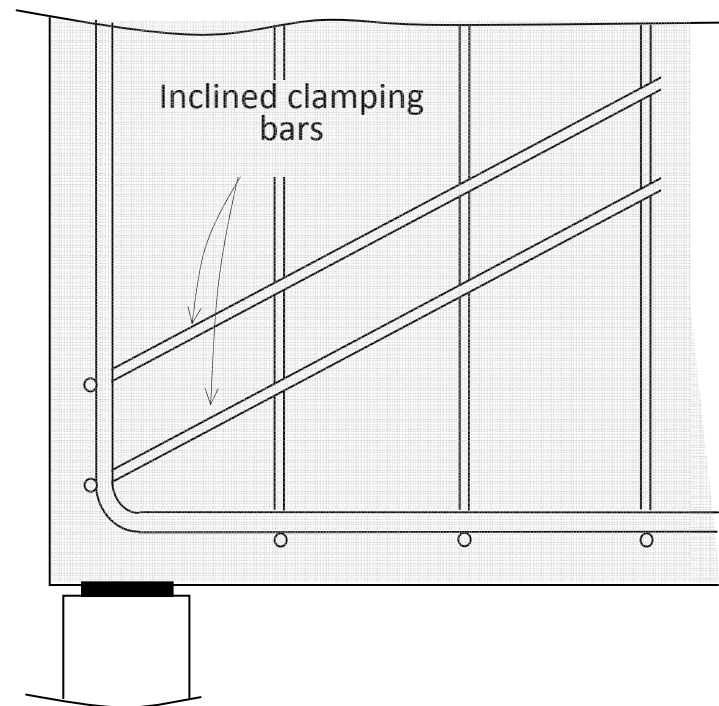
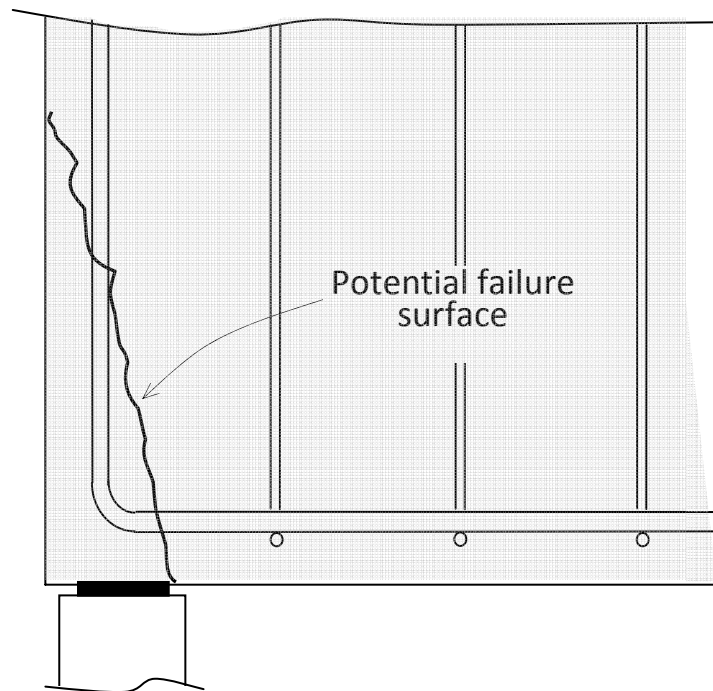
Sections and Elevations



Plan

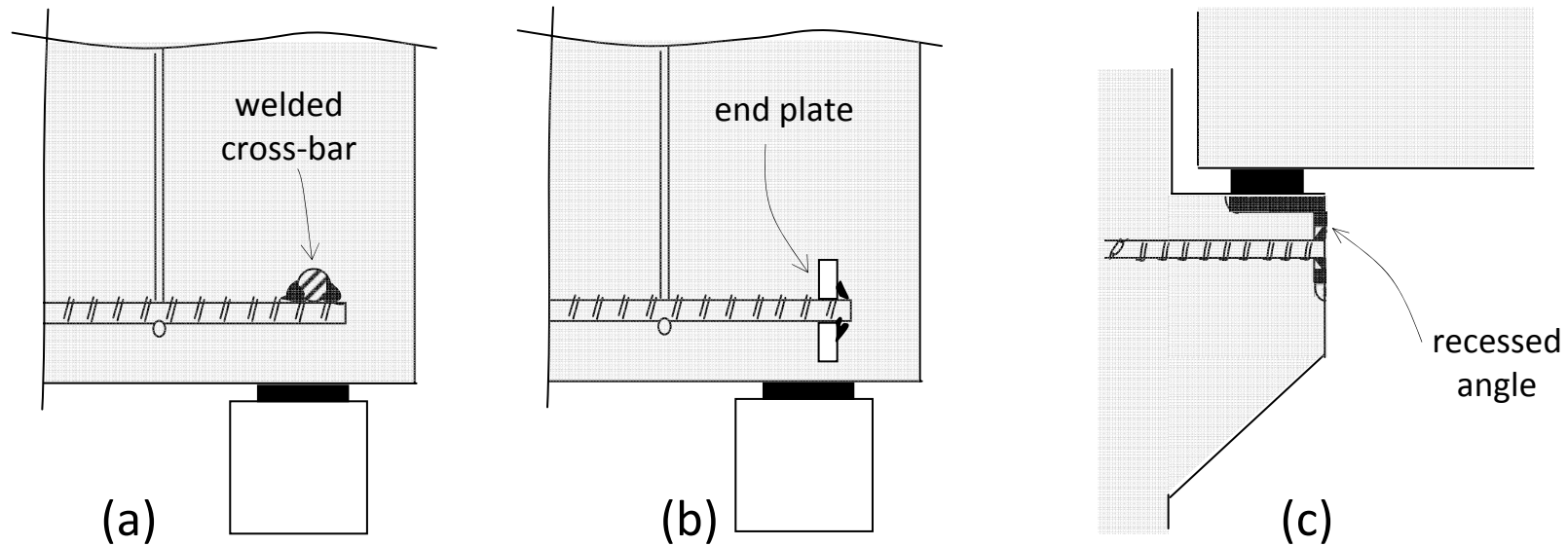
The transverse tension that may cause splitting in the plane of a **hooked anchorage** at a support can be overcome at a beam support simply by tilting the hook and exposing it to the normal reaction pressure.

Detailing of beams (ctd):



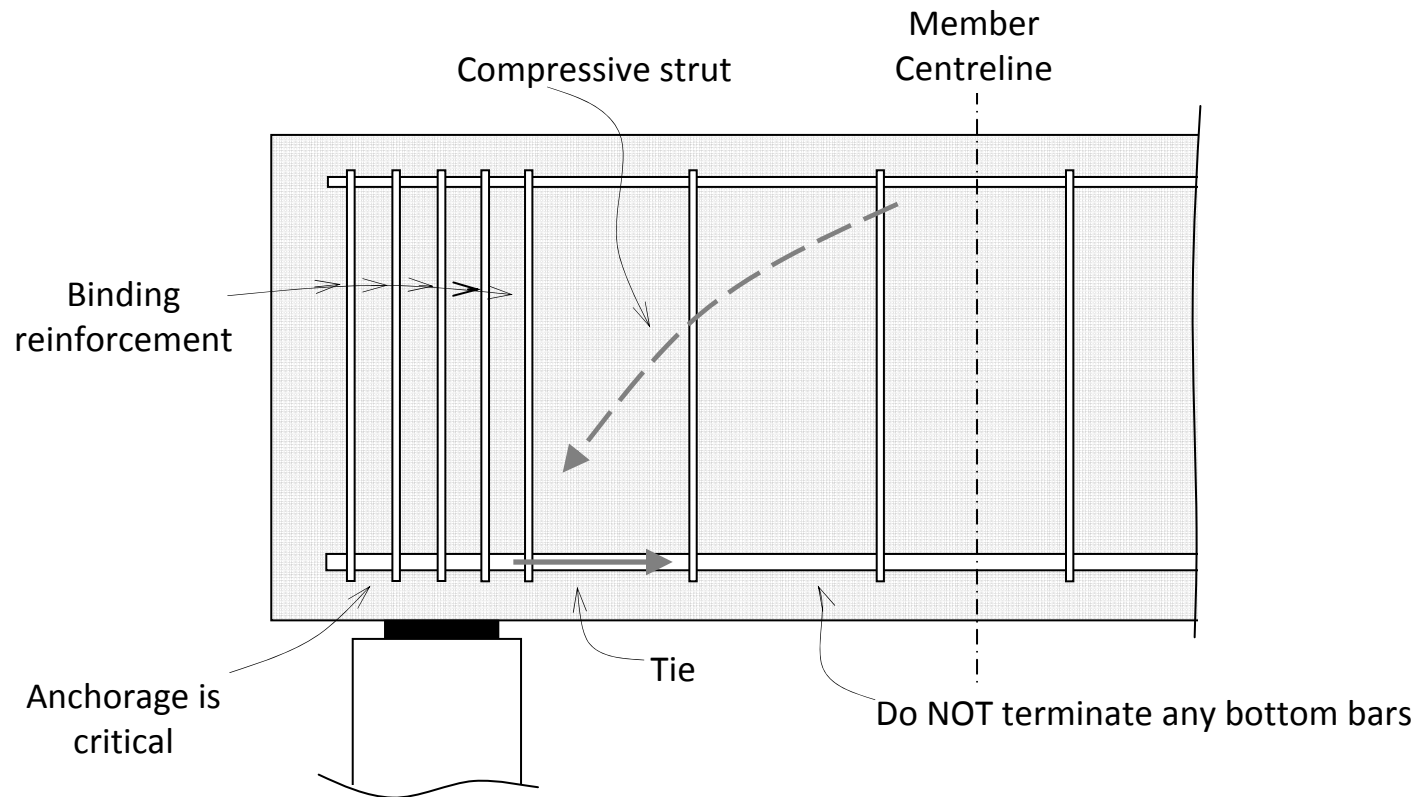
If the bearing length at a support is small and close to the free end of a member, a *sliding shear failure* along a steep inclined crack may occur. Additional small diameter bars may be required perpendicular to the potential failure plane

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.

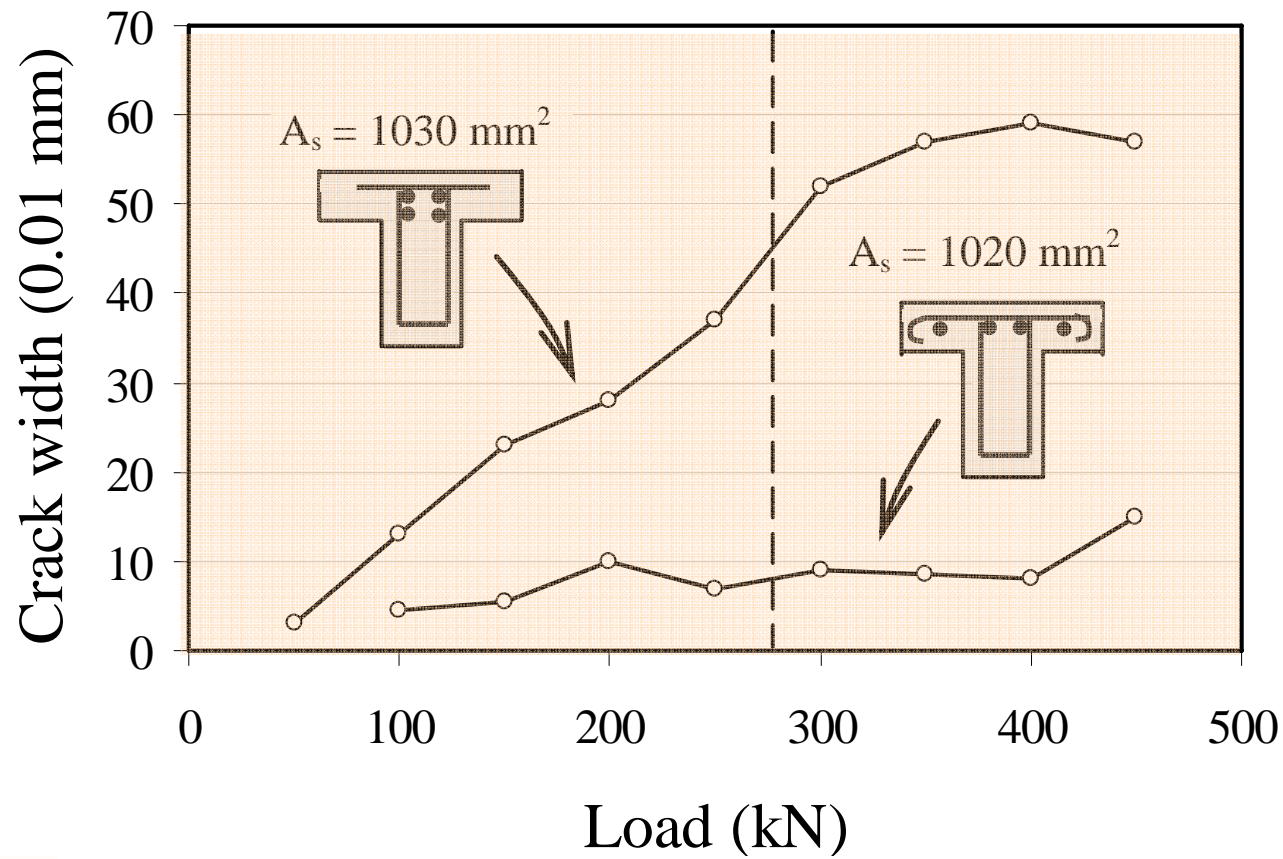
Detailing of beams (ctd):



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.

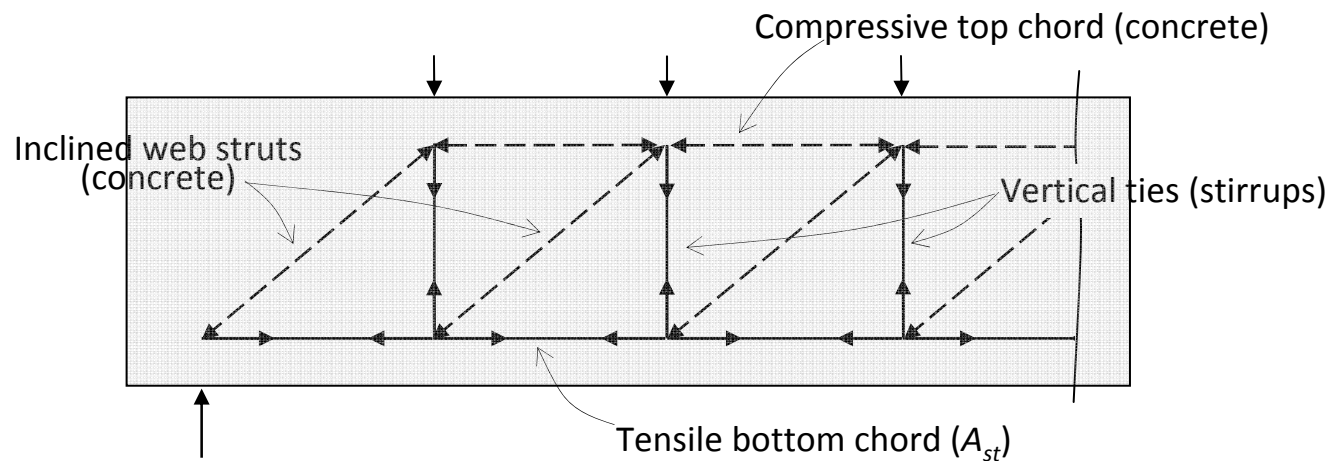
Detailing of beams (ctd):

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.)



Detailing of beams (ctd):

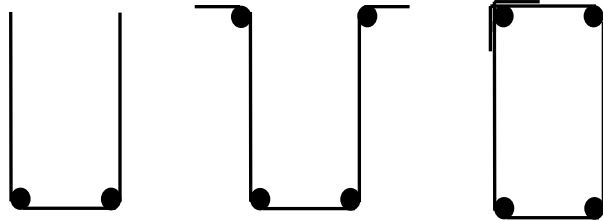
Anchorage of Stirrups:



- **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).

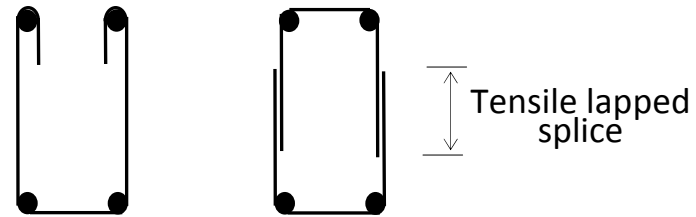
Detailing of beams (ctd):

Types of Stirrups:



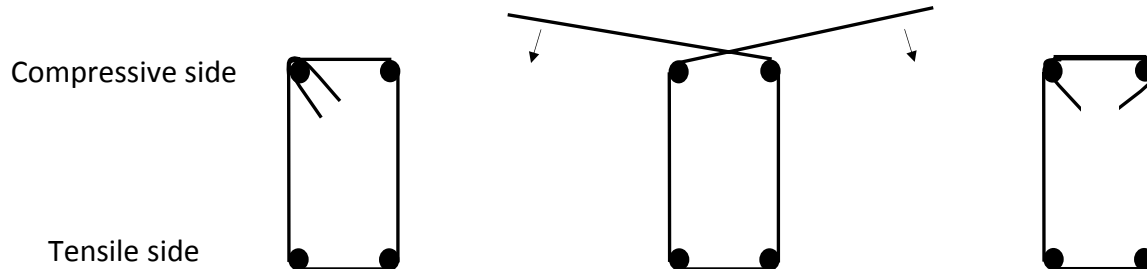
(a) Incorrect

Inadequate anchorage
A 90° cog is ineffective if the
cover concrete is lost



(b) Undesirable (but satisfactory)

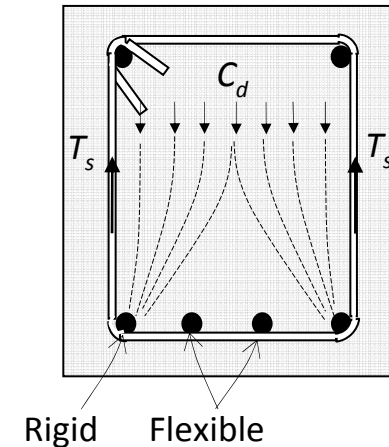
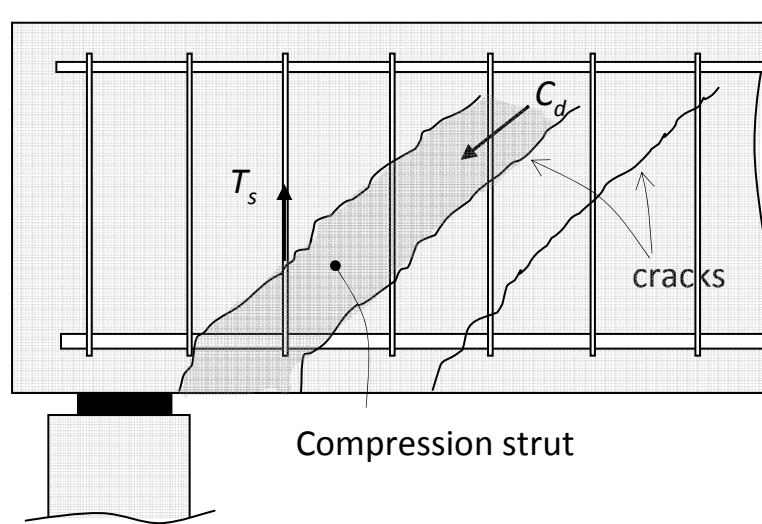
In regions where ductility is required,
the open stirrups (commonly used in
post-tensioned beams) do not confine the
compressive concrete



(c) Satisfactory

Detailing of beams (ctd):

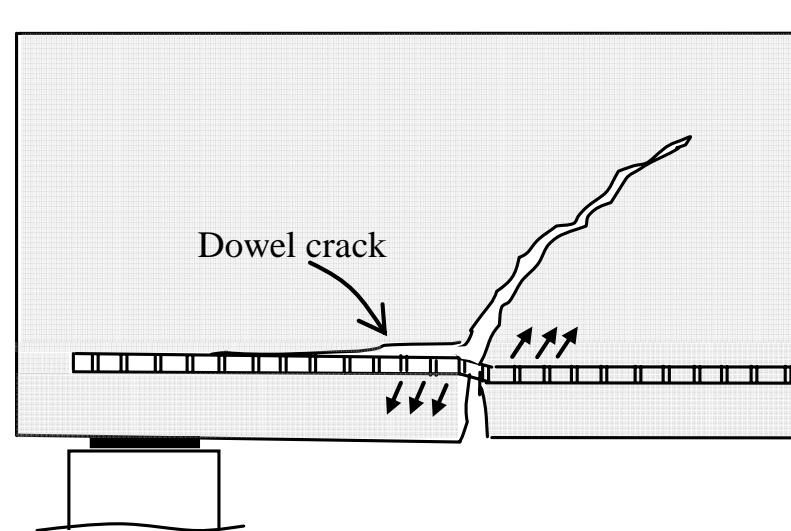
Types of Stirrups:



- **Multi-leg stirrups** should be used in members with wide webs to avoid the undesirable distribution of diagonal compression shown
- Multi-leg stirrups better control shear cracking and help maintain shear transfer though **aggregate interlock**

Detailing of beams (ctd):

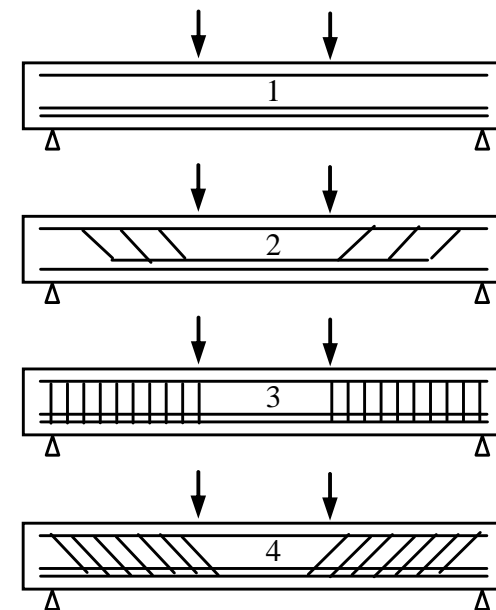
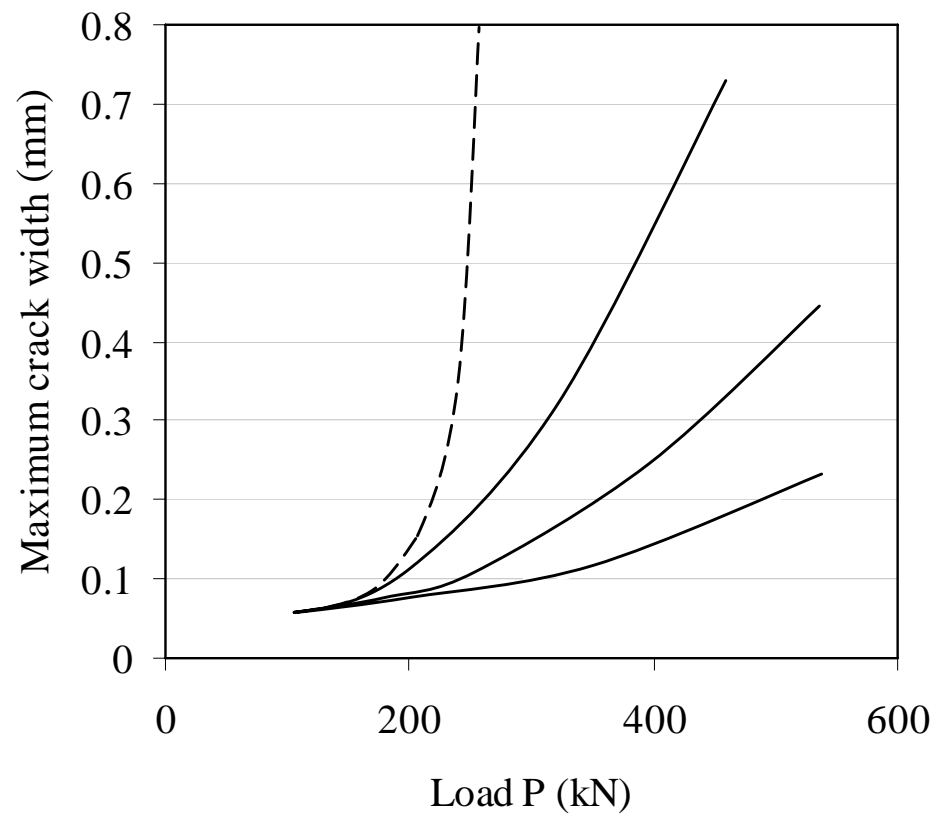
Types of Stirrups:



- **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).

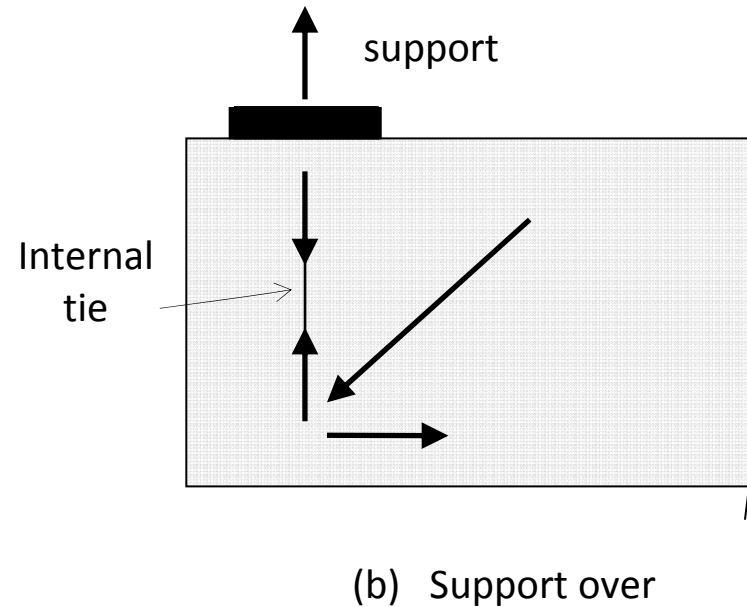
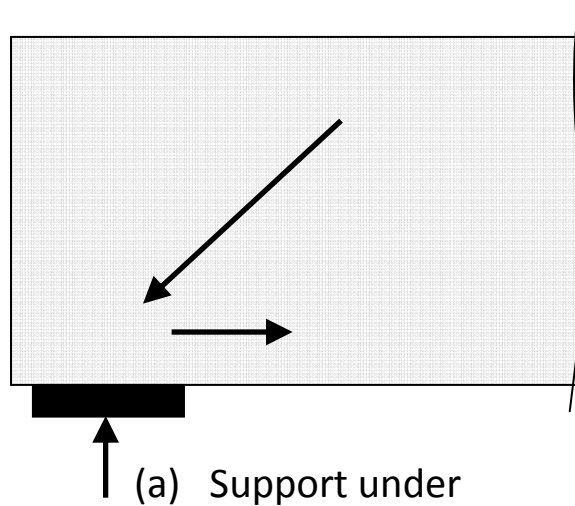
Detailing of beams (ctd):

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



Detailing of beams (ctd):

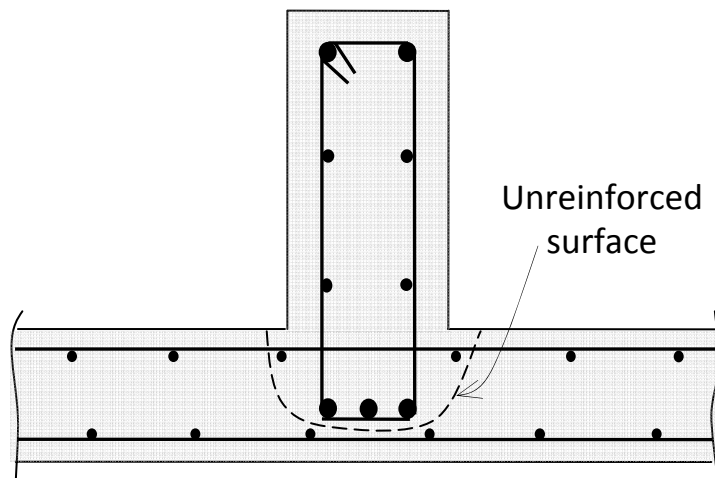
Support and Loading Points:



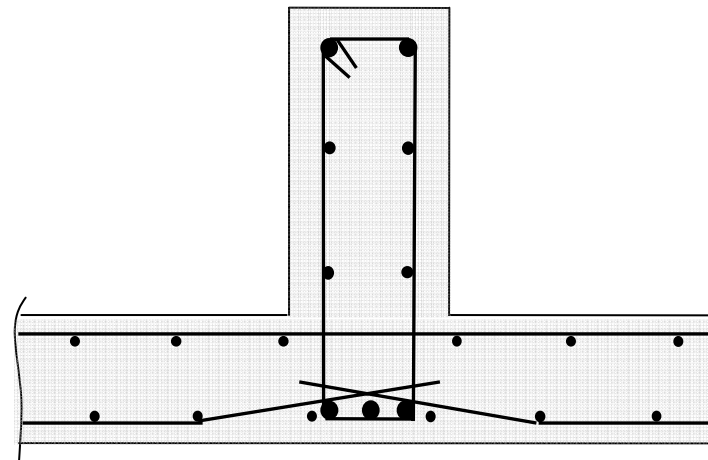
- When the support is **at the soffit of a beam or slab**, the diagonal compression passes directly into the support as shown
- When the support is **at the top of the beam**, the diagonal compression must be carried back up to the support via an internal tie.
- It is essential that **adequately anchored 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:



(a) Incorrect detail

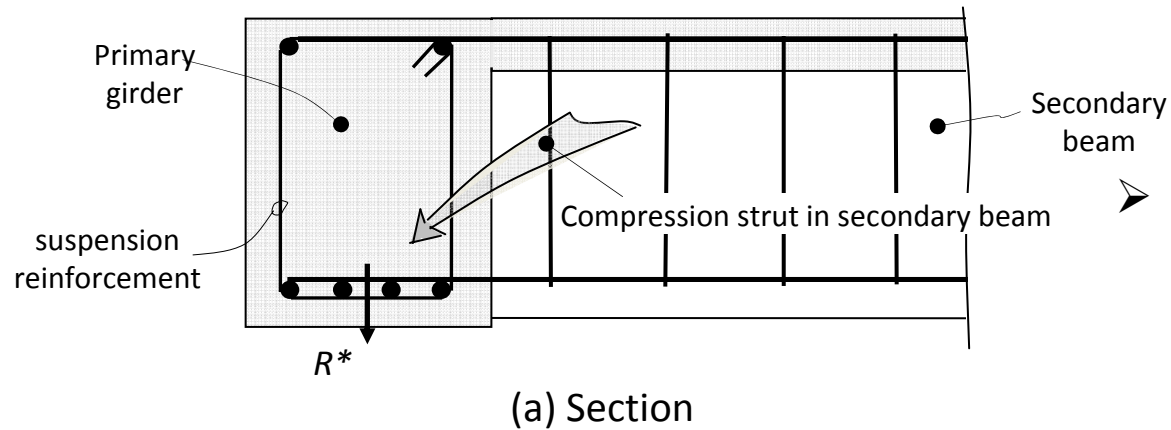


(b) Correct detail

- 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

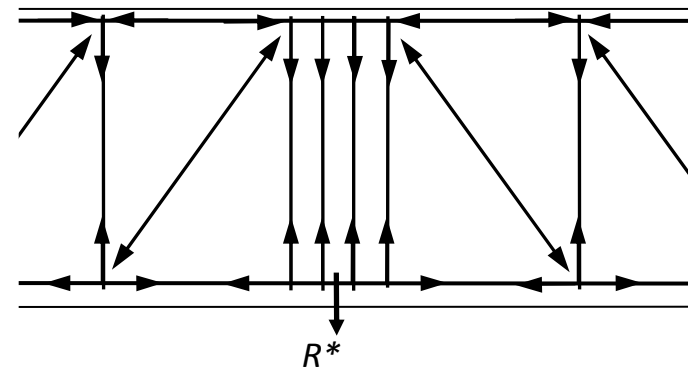
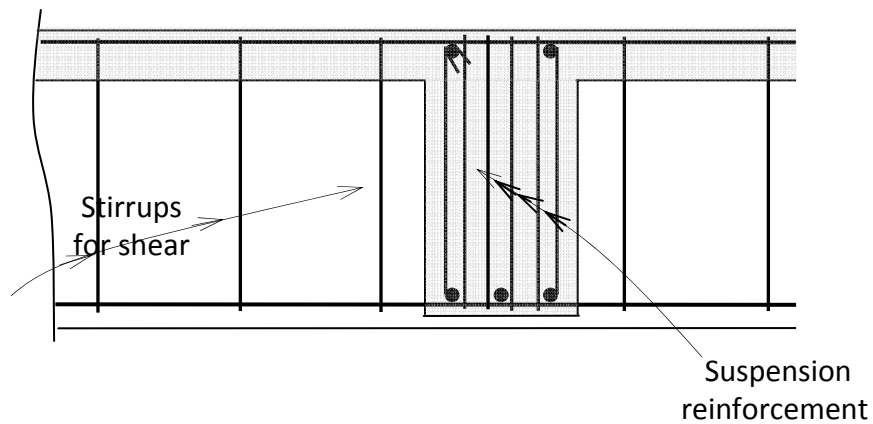
Detailing of beams (ctd):

Beam-to-beam connection:



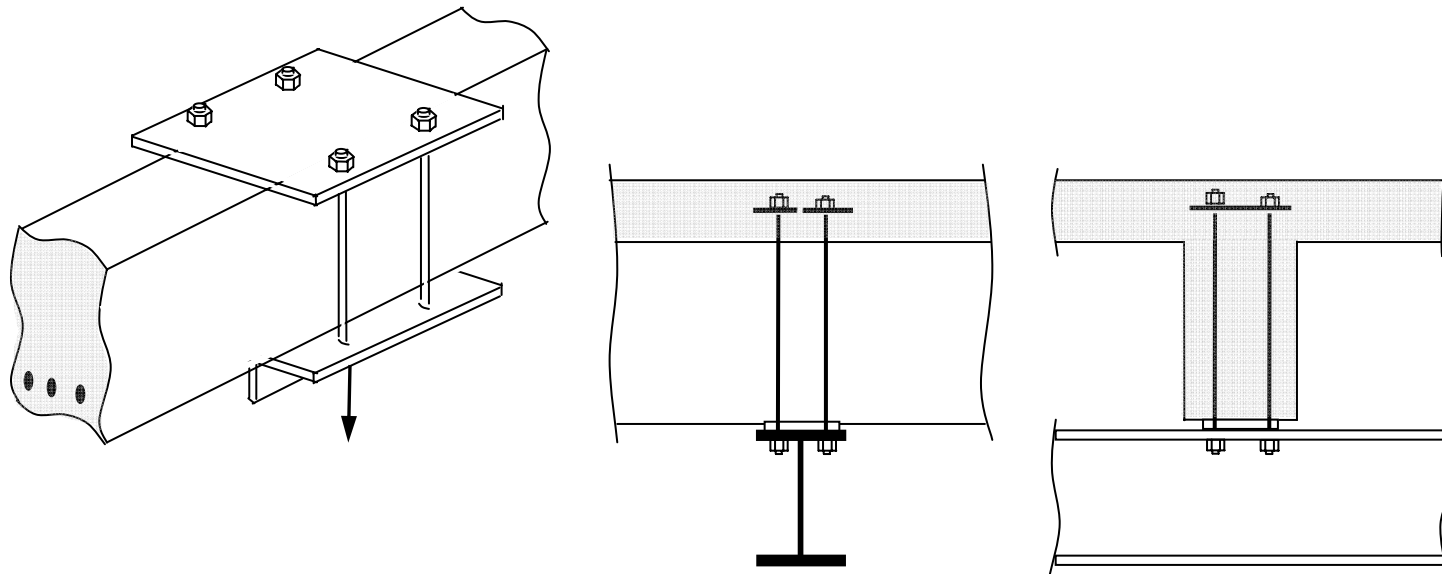
➤ The area of additional **suspension reinforcement** is

$$A_{sr} = \frac{R^*}{\phi f_{sy}}$$



Detailing of beams (ctd):

Beam-to-beam connection:

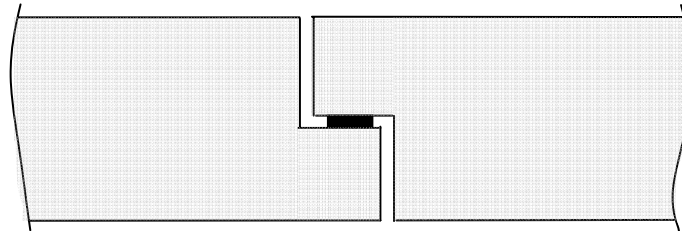


(b) Internal rods

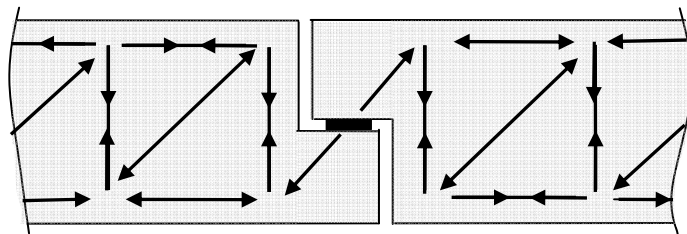
- When a **load is applied to the underside of a reinforced concrete beam**, some device must be used to transfer this hanging load to the top of the beam

Detailing of beams (ctd):

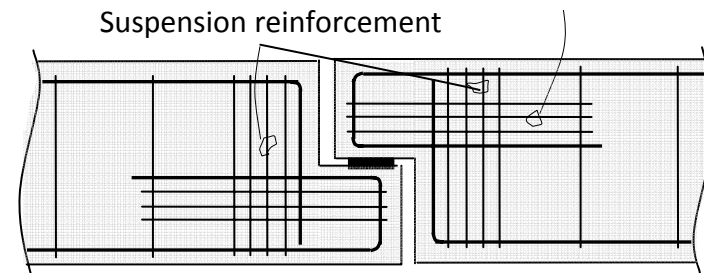
Half-Joint or dapped-end joint:



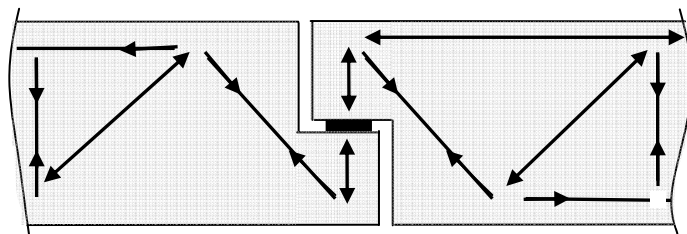
(a) Half joint



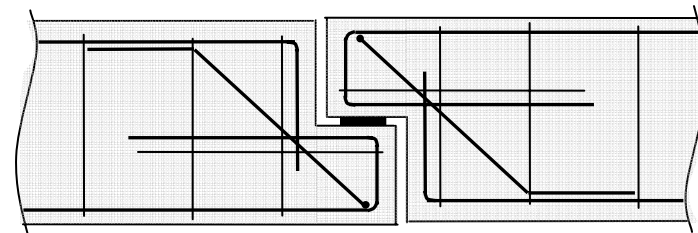
(b) Strut and tie model



Reinforcement detail



(c) Alternative strut and tie model



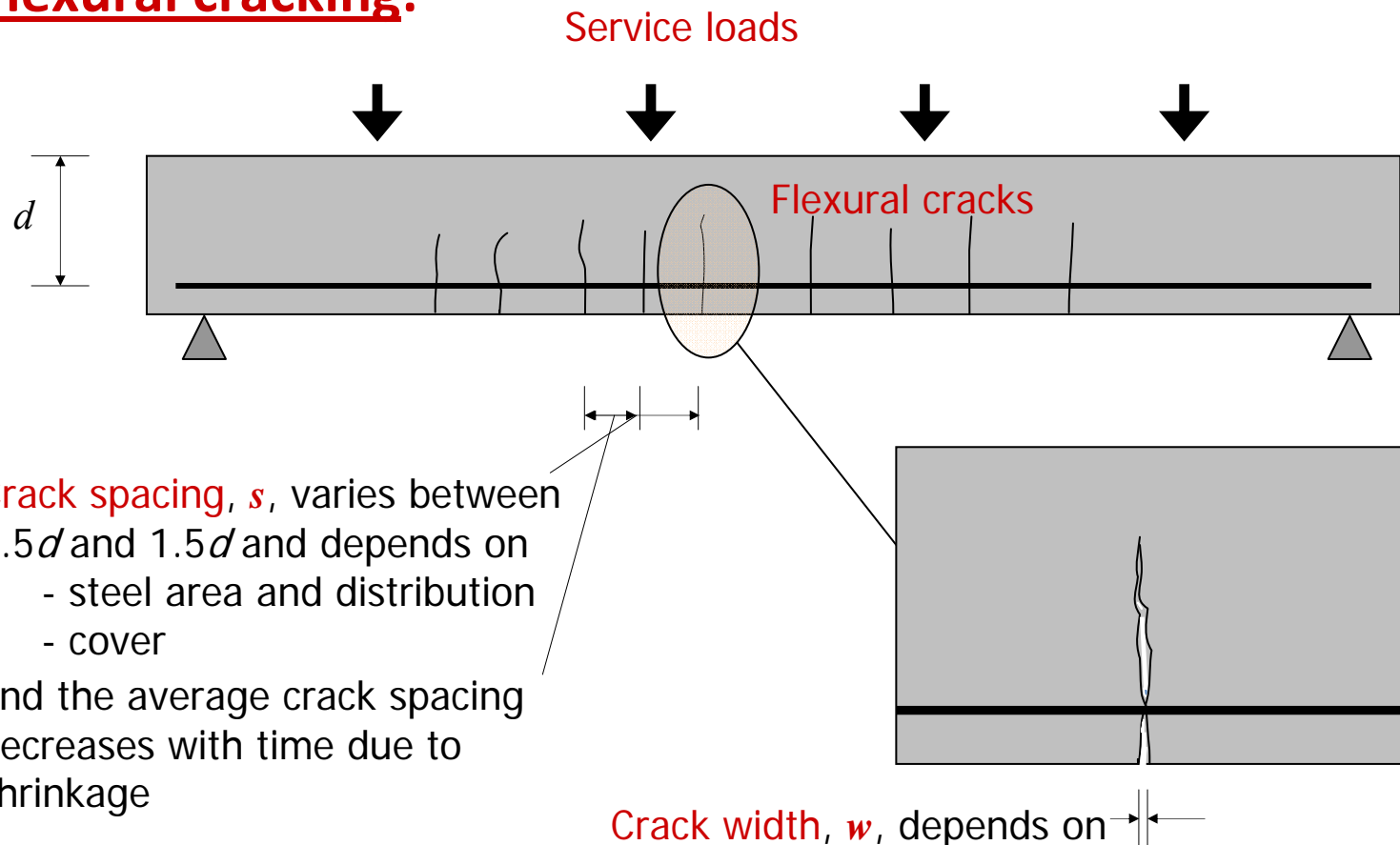
Reinforcement detail

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.

Flexural cracking:



Maximum crack widths increase with time by a factor of between 2 and 4

- steel stress
 - bar diameter and bar spacing
 - cover
 - adjacent crack spacings
- and increases with time due to shrinkage

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.

Nominal bar diameter (mm)	Maximum steel stress (MPa)
10	360
12	330
16	280
20	240
24	210
28	185
32	160
36	140
40	120

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

Centre-to-centre spacing (mm)	Maximum steel stress (MPa)
50	360
100	320
150	280
200	240
250	200
300	160

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.

Nominal bar diameter (mm)	Maximum steel stress (MPa) for overall depth D_s (mm)	
	≤ 300	> 300
6	375	450
8	345	400
10	320	360
12	300	330
16	265	280
20	240	
24	210	

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

Centre-to-centre spacing (mm)	Maximum steel stress (MPa)
50	360
100	320
150	280
200	240
250	200
300	160

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, ρ_{\min}** is

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

where $f_{ct} = 0.25\sqrt{f'_c}$. For 32 MPa concrete, $\rho_{\min} = 0.0034$.

- ❖ 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:

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

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

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

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

$$(A_s)_{\min} = (1.75 - 2.5\sigma_{cp})b D \times 10^{-3} \quad (1c)$$

- ❖ 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

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

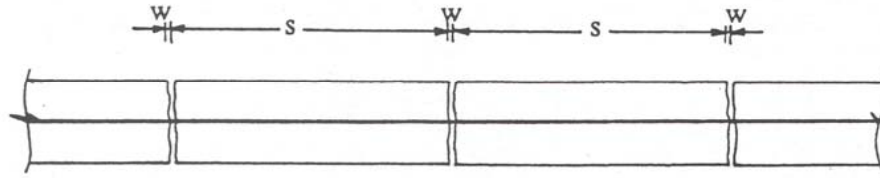
and Eq. 1b applies where a moderate degree of control over cracking is required and where cracks are inconsequential or hidden from view.

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

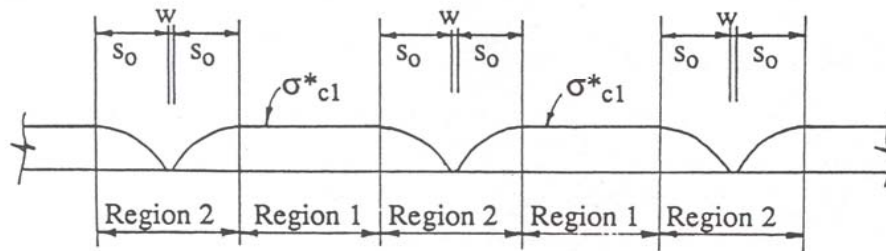
- ❖ For Exposure Classifications B1, B2, C1 and C2, Eq. 1a always applies.
- ❖ The minimum steel area given by Eq. 1c is appropriate in an unrestrained direction where the slab is free to expand or contract.
- ❖ In the primary direction of a one-way slab or in each direction of a two-way slab, the minimum 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.

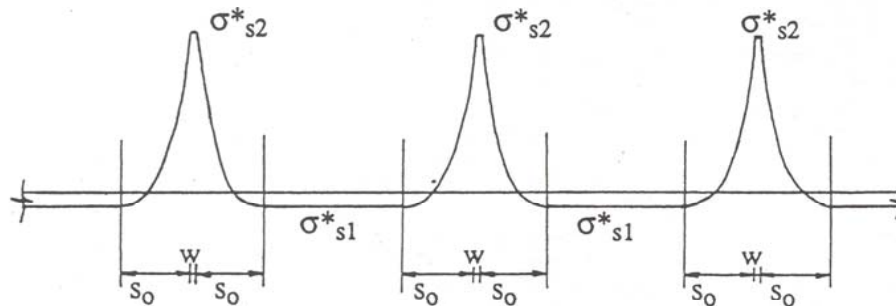
With time, restrained shrinkage cracks occur at roughly regular centres depending on the amount of reinforcement:



(a) Portion of restrained member after all cracking



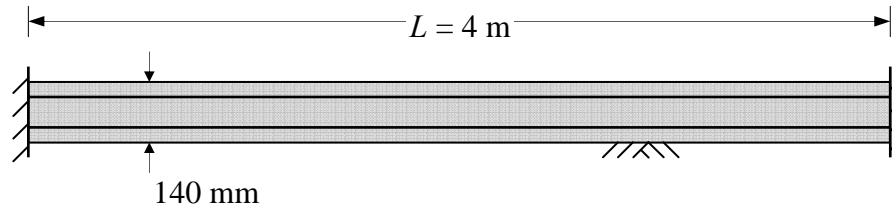
(b) Average concrete stress after all shrinkage



(c) Steel stress after all shrinkage cracking

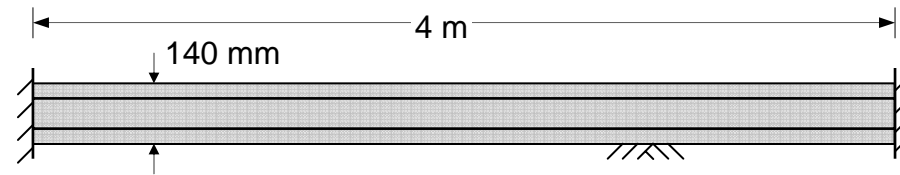
Typical values:

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 $\rho = A_s/A_c = 0.00643$.

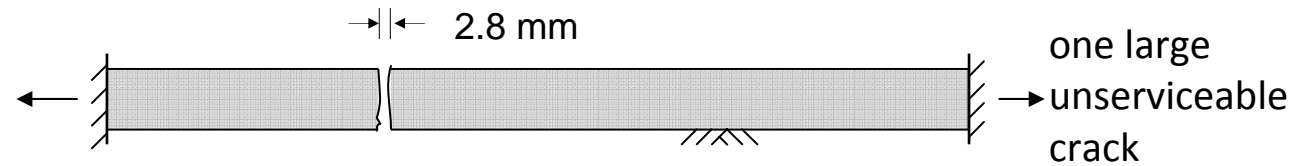


For 25 MPa concrete with a final shrinkage strain of 0.0007 and typical material properties, a shrinkage cracking analysis of this restrained slab indicates **4 or 5 full depth cracks** within the 4 m length with the **maximum final crack width** about **0.3 mm**.

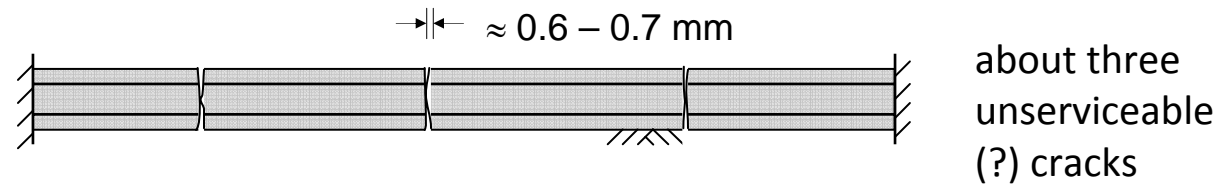
$$p = A_s/A_c$$



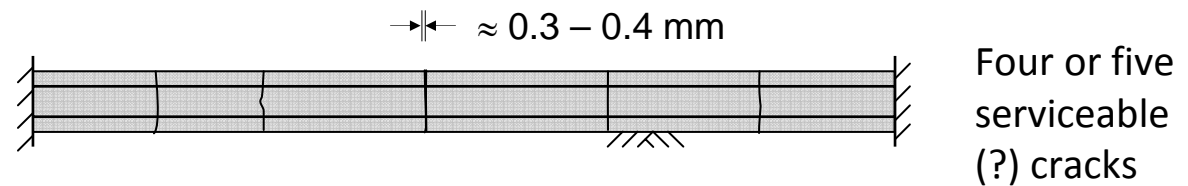
If $p = 0$:



If $p = 0.0035$

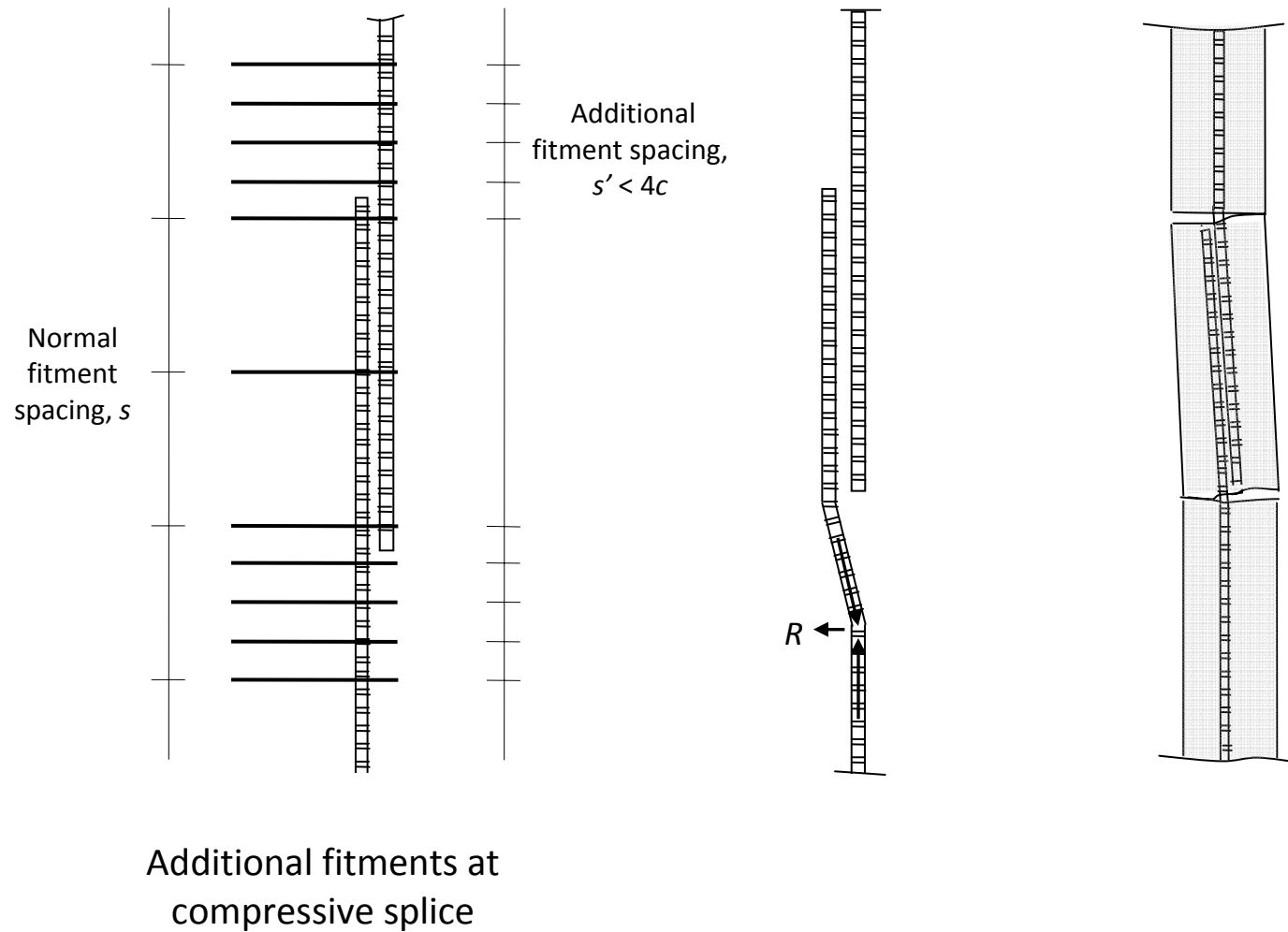


If $p = 0.006$



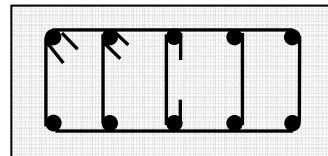
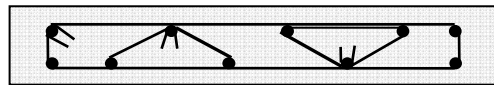
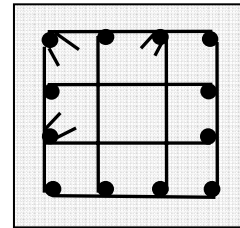
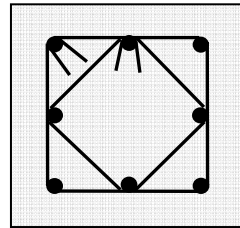
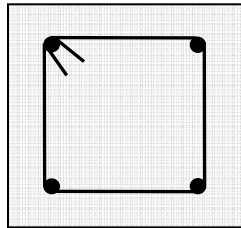
Detailing of columns:

Lapped compressive splices:



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 ≤ 150 mm.

For bundled bars – each bundle must be restrained.

Detailing of columns:

Minimum bar diameters for fitments (AS3600-2009):

Longitudinal bar diameter (mm)	Minimum bar diameter for fitment and helix (mm)
Single bars up to 20	6
Single bars 24 to 28	10
Single bars 28 to 36	12
Single bar 40	16
Bundled bars	12

Maximum spacing of fitments (AS3600-2009):

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

D_c and $15d_b$ for single bars

$0.5D_c$ and $7.5d_b$ for bundled bars

Detailing of Beam-column Connections:

Knee Connections (or two-member connections):

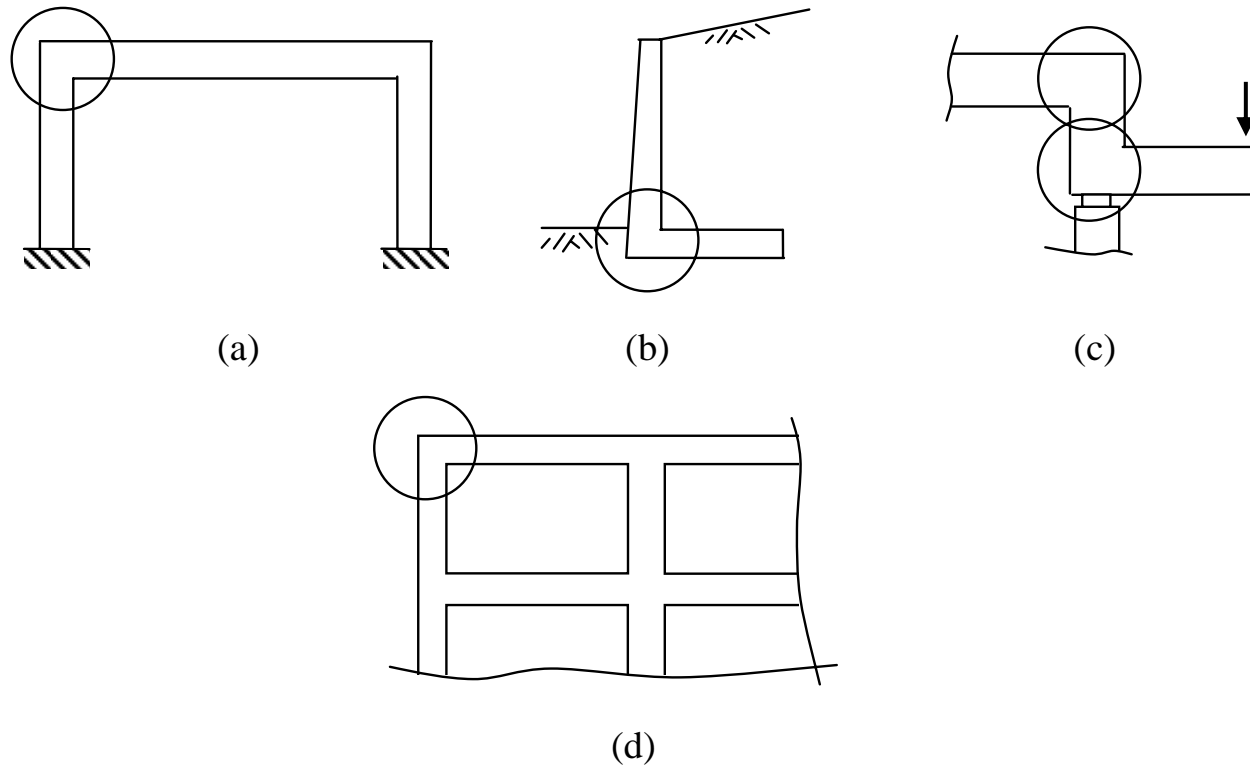
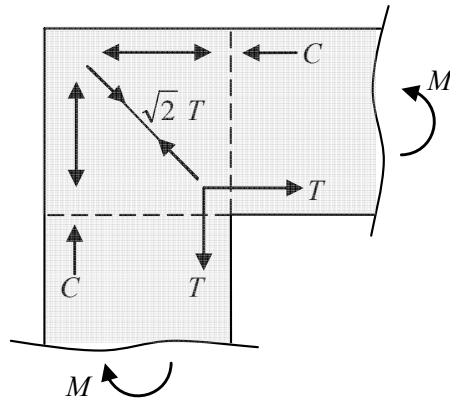


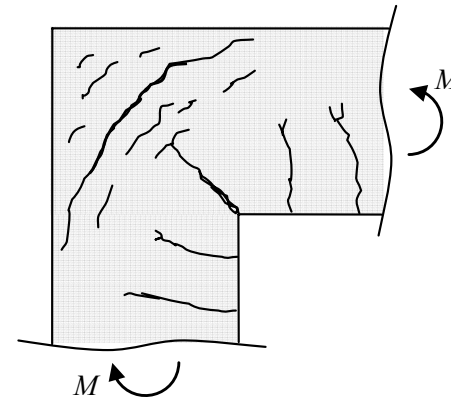
Figure 8.37 Two-member connections.

Detailing of Beam-column Connections:

Knee Connections under “Opening” Moment:

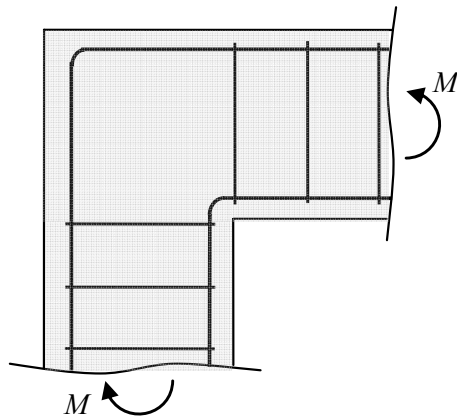


(a) Internal forces

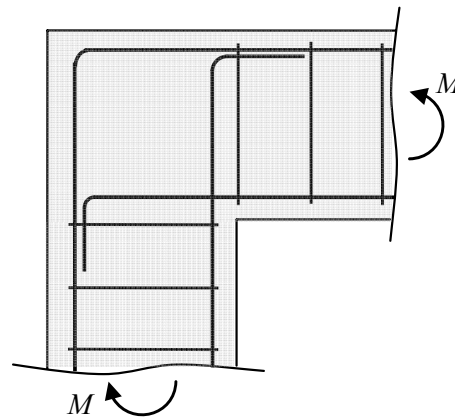


(b) Crack pattern

$$A_{sv} = \frac{\sqrt{2} T}{\phi f_{sy} \cdot f} = \frac{\sqrt{2} A_{st} f_{sy}}{\phi f_{sy} \cdot f}$$



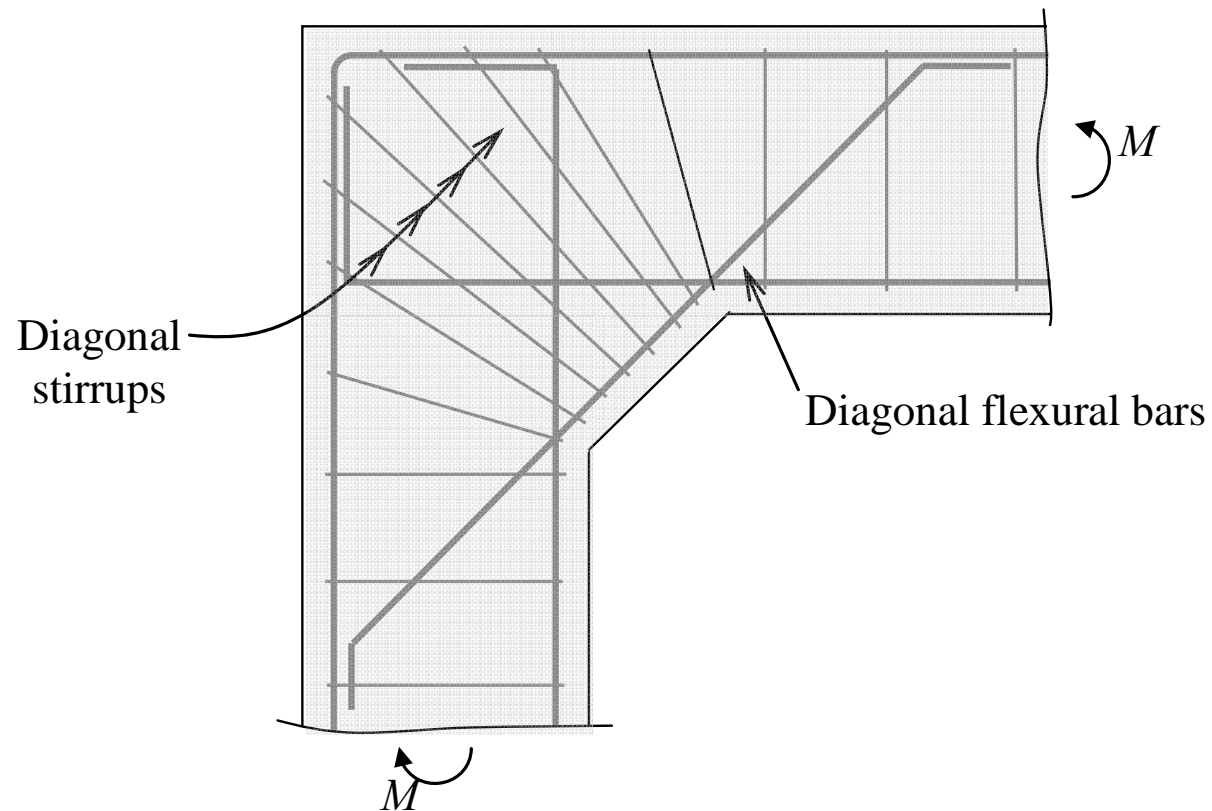
(b) Unsatisfactory



(c) Potentially satisfactory

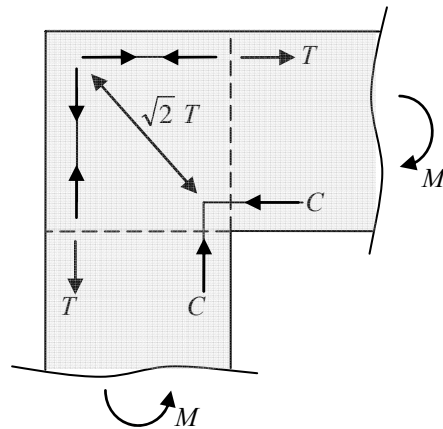
Detailing of Beam-column Connections:

Knee Connections under “Opening” Moment – Suggested detail:

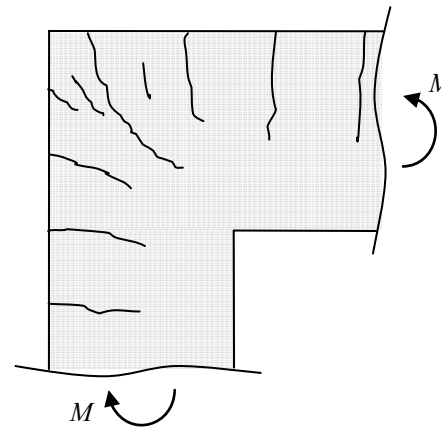


Detailing of Beam-column Connections:

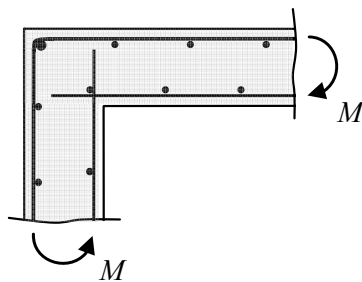
Knee Connections under “Closing” Moment:



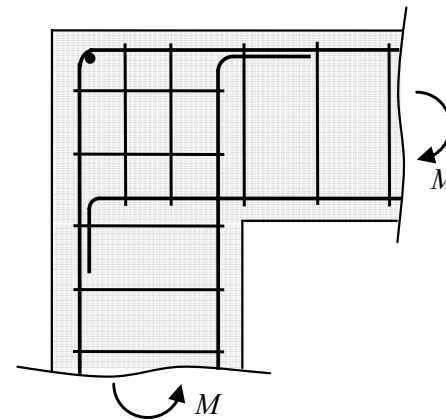
(a) Internal forces



(b) Crack pattern



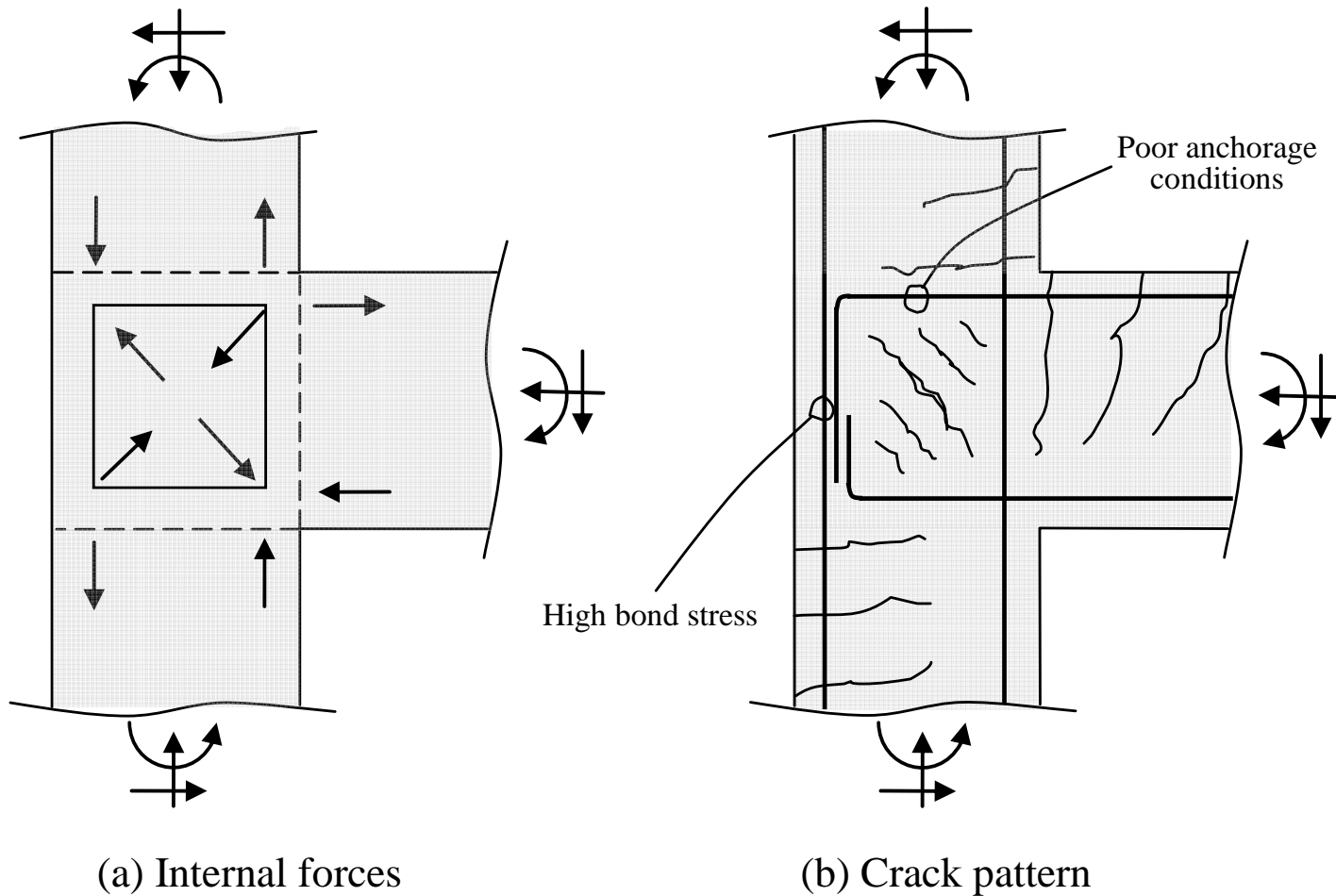
(a) Wall or slab connection (when $p \leq f_{ctf}/f_{sy}$)



(b) Beam to column knee connection

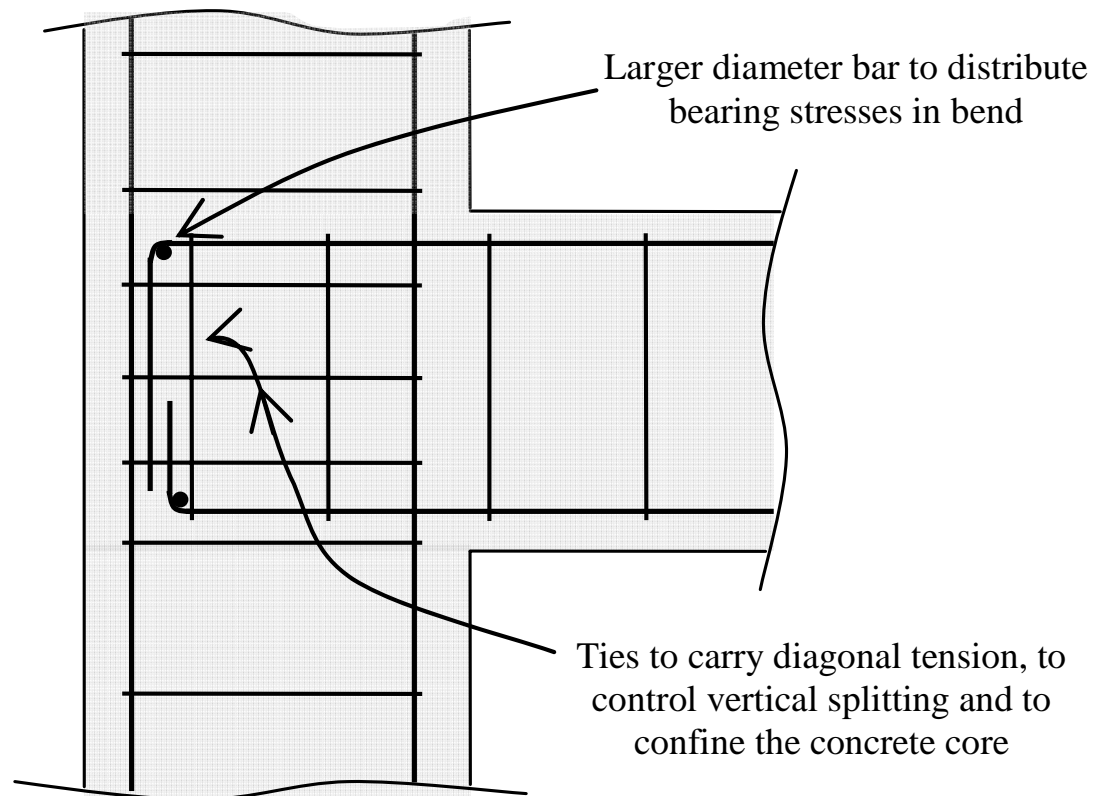
Detailing of Beam-column Connections:

Three-member connections:



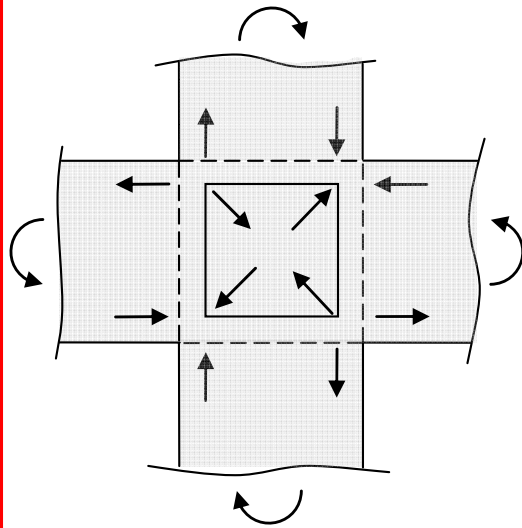
Detailing of Beam-column Connections:

Three-member connections – Reinforcement detail:

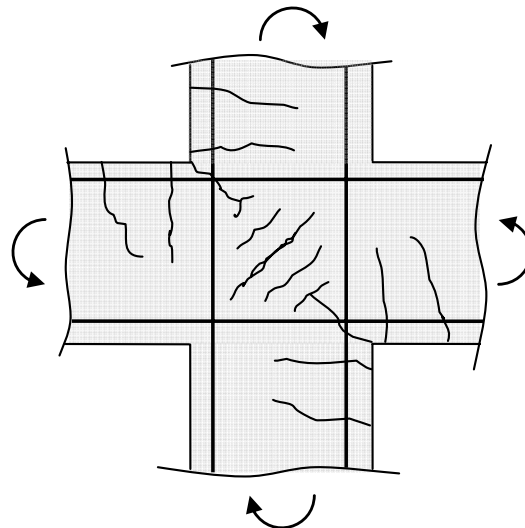


Detailing of Beam-column Connections:

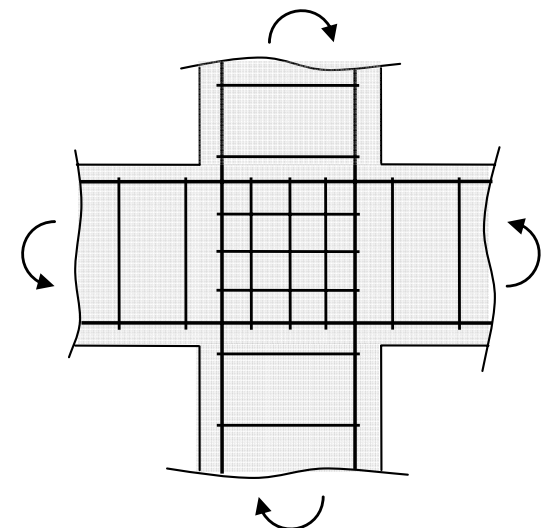
Four-member connections:



(a) Internal forces

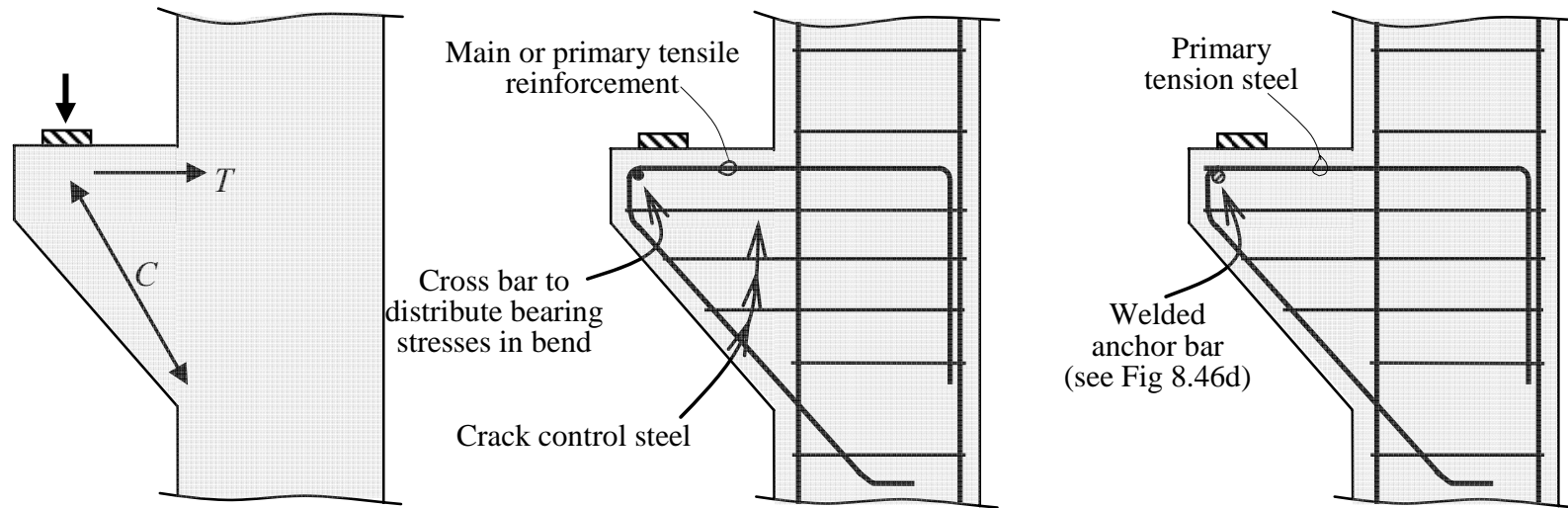


(b) Crack pattern



(c) Reinforcement detail

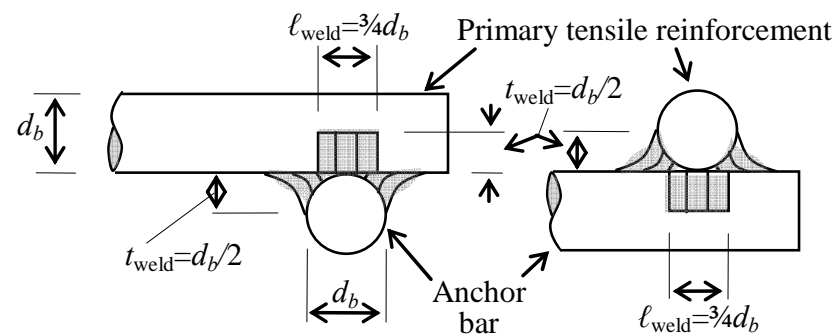
Detailing of Corbels:



(a) Strut-and-tie action

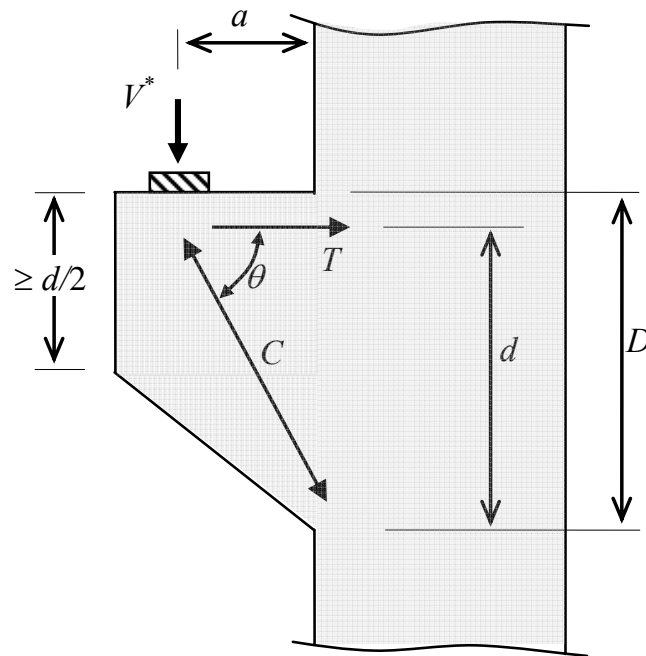
(b) Reinforcement detail

(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: $\phi_{st} C_u = \phi_{st} \beta_s 0.9 f'_c A_c \quad (\phi_{st} = 0.6)$

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

$$A_h \geq A_s / 2$$

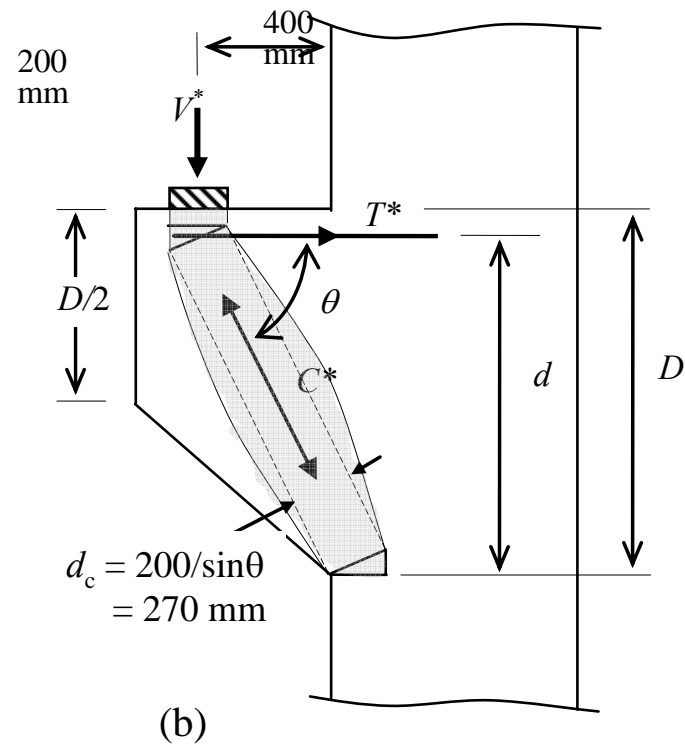
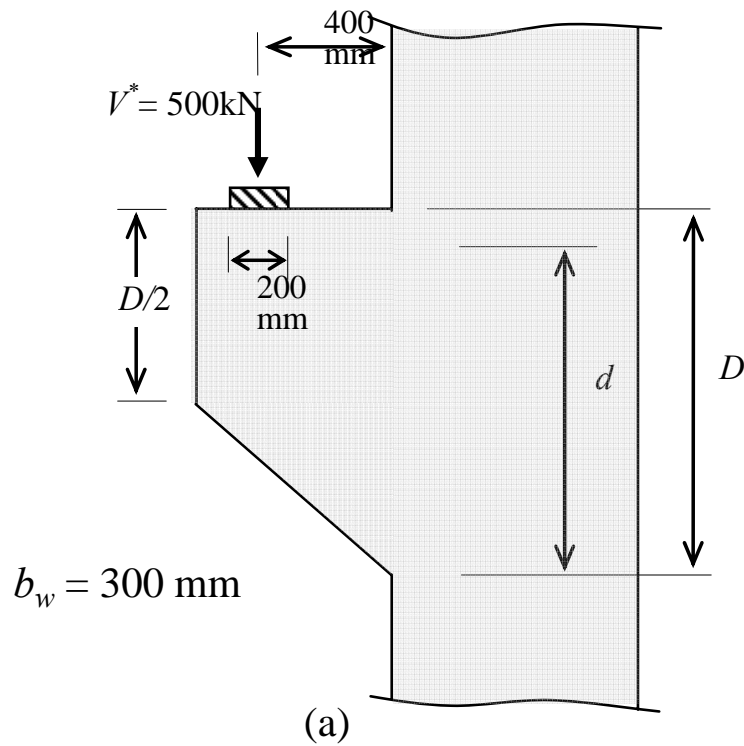
Park and Paulay suggest that a good first estimate of corbel dimensions is obtained from:

$$V^* / b_w d \leq 0.56 \sqrt{f'_c}$$

and ACI318-08 suggests

$$a / d \leq 1.0 \quad \text{and} \quad 0.04 f'_c / f_{sy} \leq A_s / b_w d \leq 0.2 f'_c / f_{sy}$$

Design of Corbels:



$$f'_c = 32 \text{ MPa}$$

$$f_{sy} = 500 \text{ MPa}$$

Cover = 30 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:

$$\tan \theta = \frac{d - 100 \tan(90 - \theta)}{400} \quad \therefore \theta = 47.7^\circ$$

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:

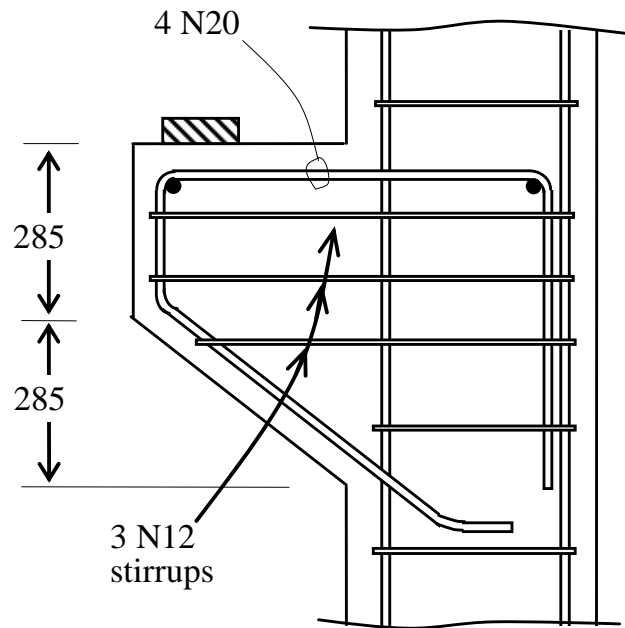
$$\beta_s = \frac{1}{1.0 + 0.66 \cot^2 \theta} = 0.65$$

and

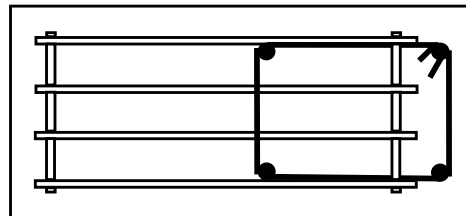
$$\phi_{st} C_u = 0.6 \times 0.65 \times 0.9 \times 32 \times 81150 = 911 \text{ kN}$$

$$> C^* = V^* / \cos \theta = 743 \text{ kN} \quad \therefore \text{OK}$$

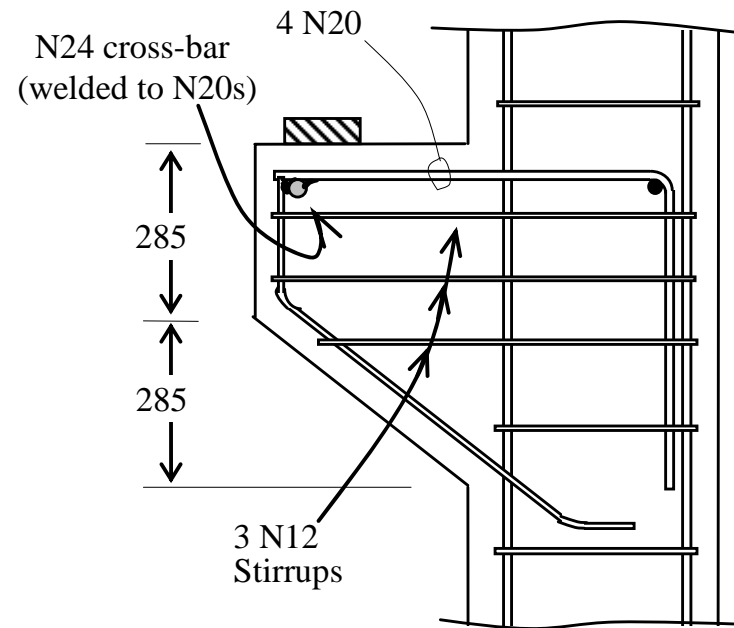
Design of Corbels:



ELEVATION

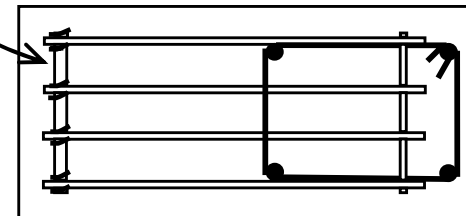


PLAN



ELEVATION

N28 welded cross-bar



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;

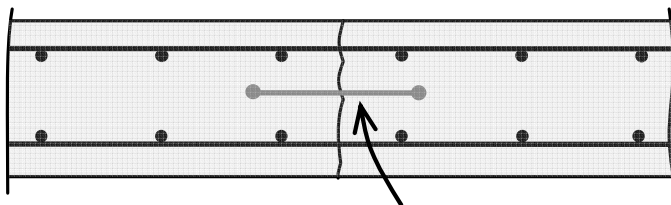
structural joints (such as hinges, pin and roller joints);

shrinkage strips; and

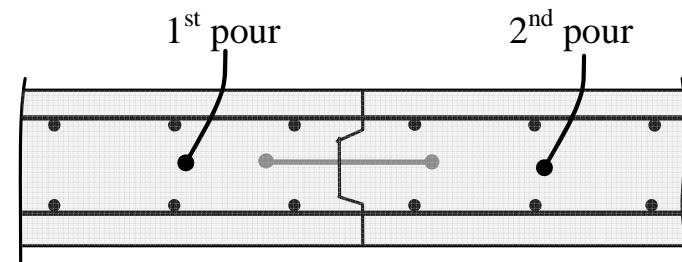
isolation joints.

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

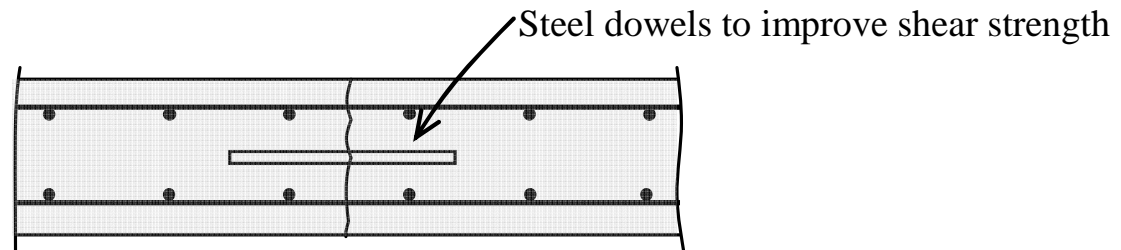
Construction Joints:



(a) Butt joint

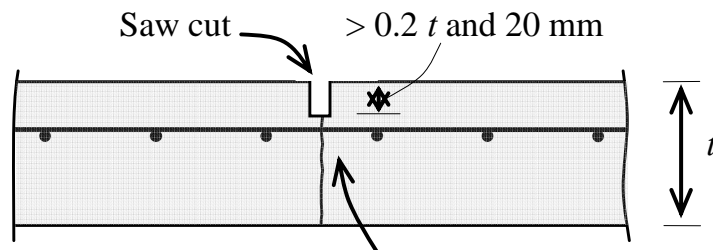


(b) Keyed joint



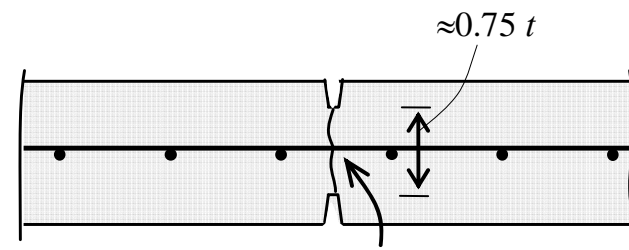
(c) Doweled joint

Control Joints (or Contraction Joints):

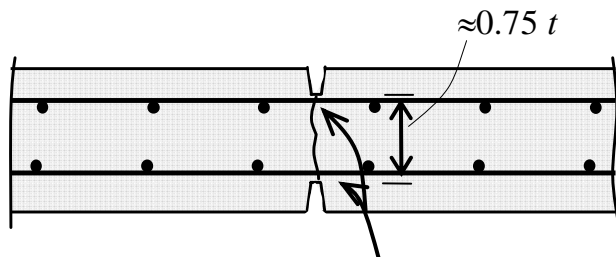


Discontinue every second bar if necessary so that $p < 0.002$

(a) Saw-cut joint in slab on ground

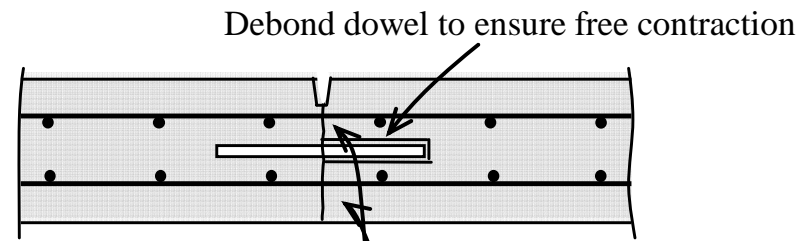


(b) Wall ($t < 200$ mm)



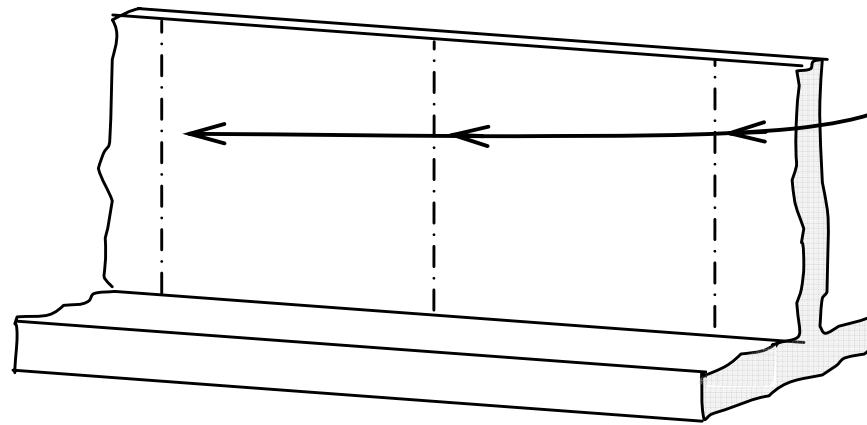
Discontinue every second bar if necessary so that $p < 0.002$

(d) Wall ($t \geq 200$ mm)

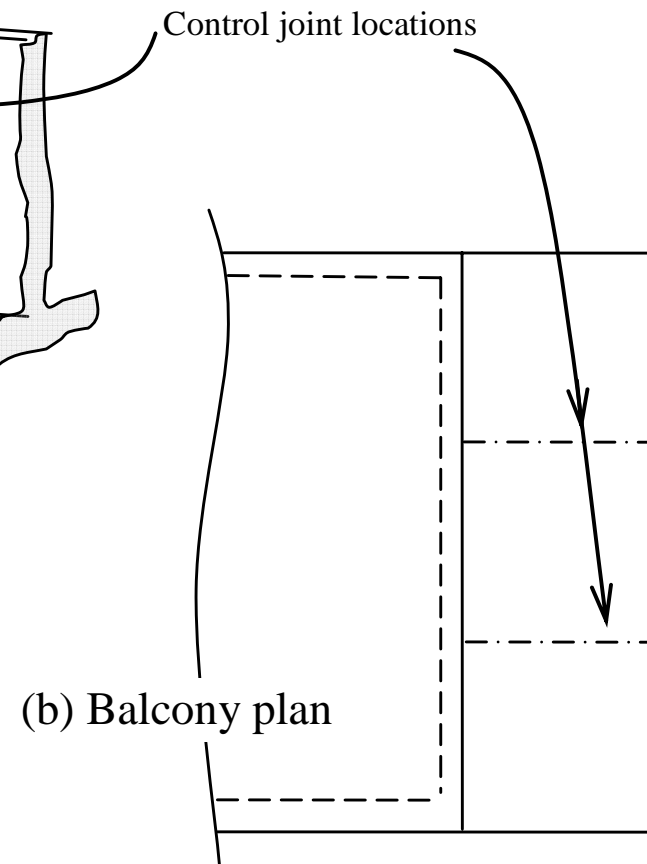


(c) Doweled joint

Typical control joint locations:

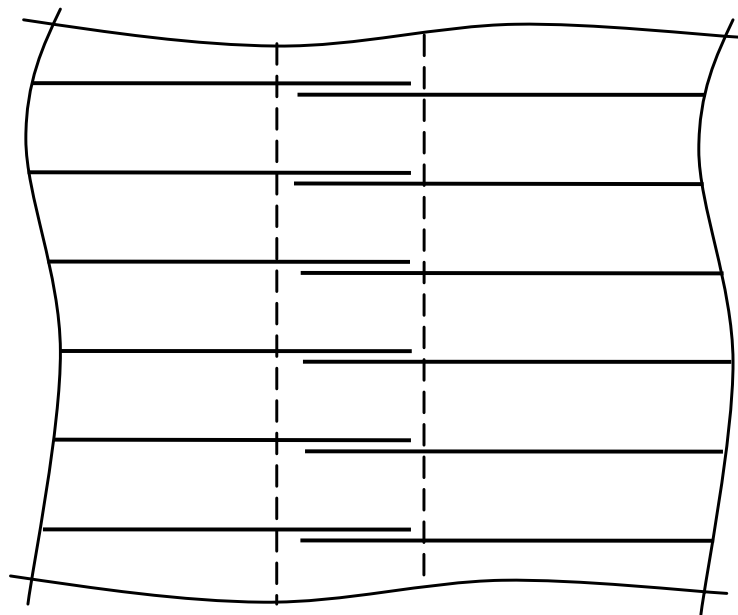


(a) Wall elevation

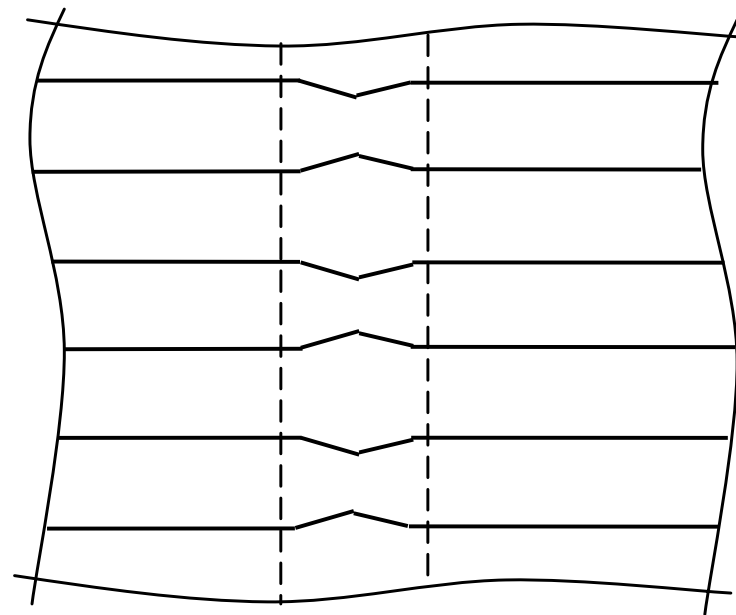


(b) Balcony plan

Alternative shrinkage strip details:

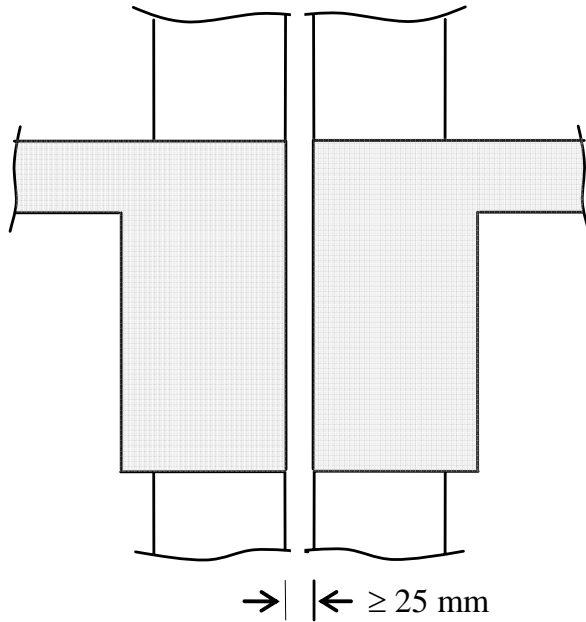


Shrinkage
strip

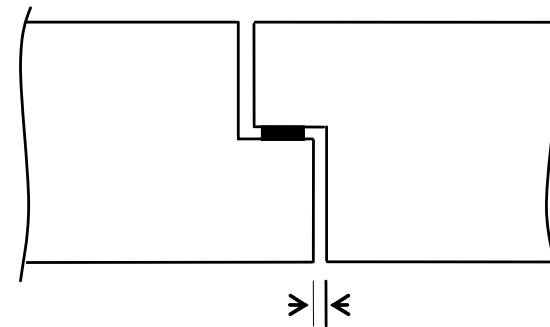


Shrinkage
strip

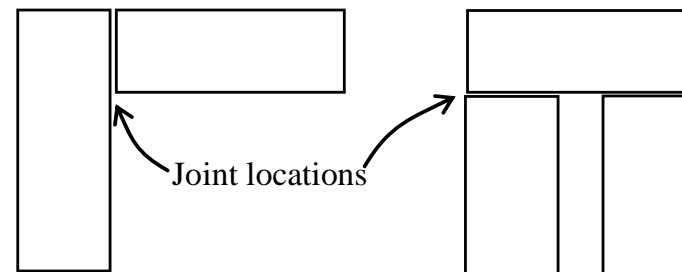
Expansion joint details:



(a) Double column and beams

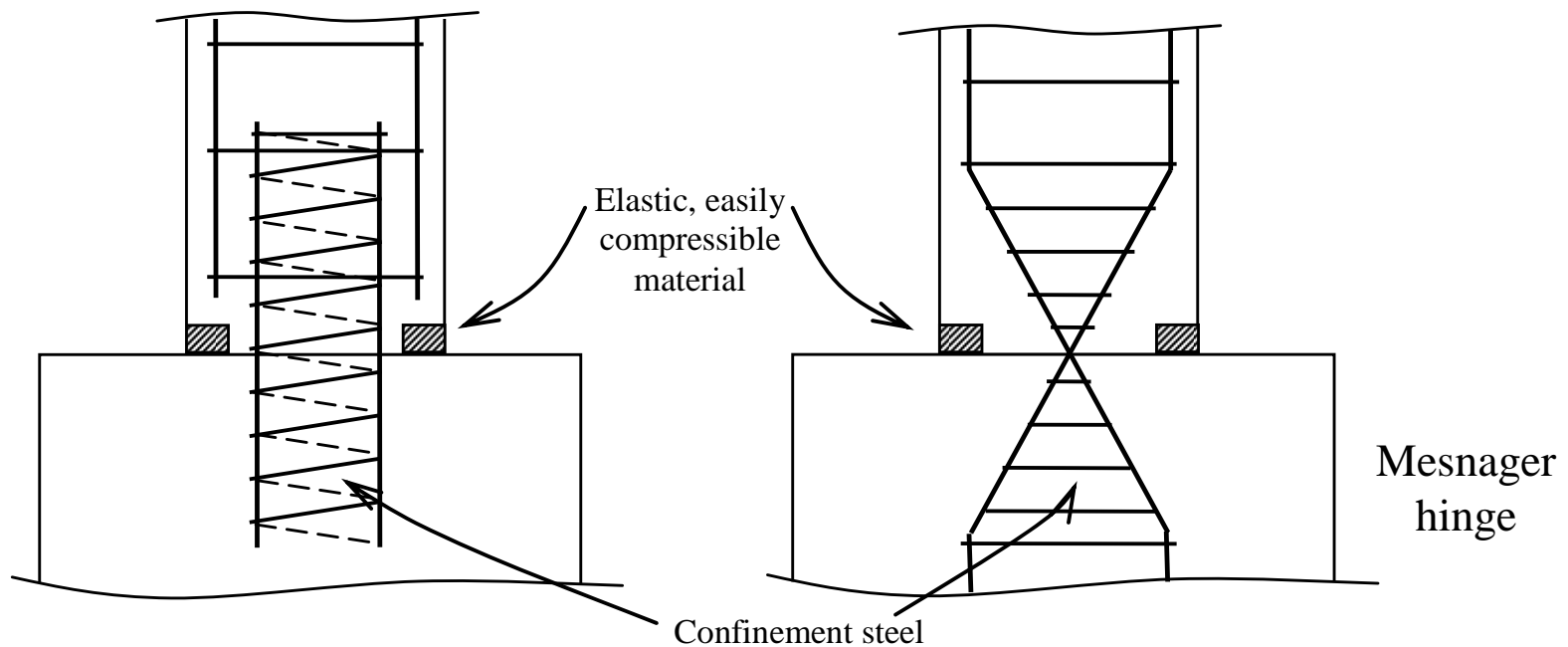


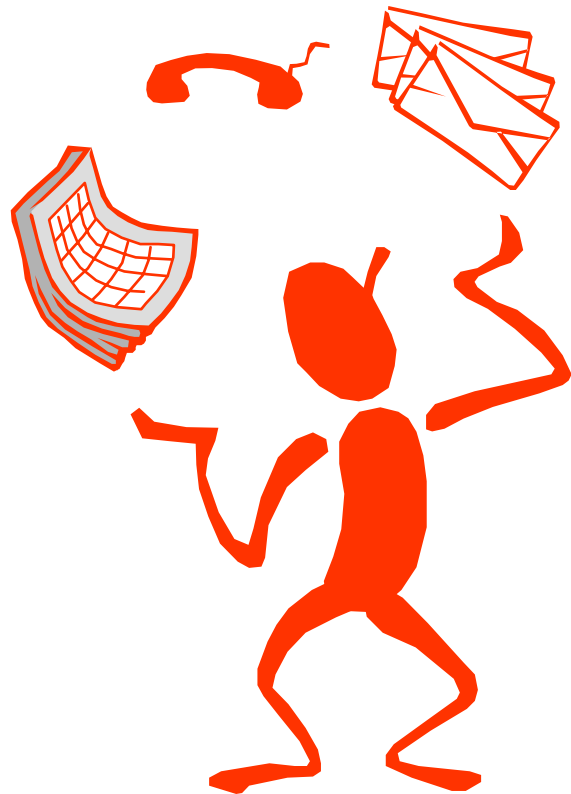
(b) Half joint



(c) Building plans – joint locations

Alternative structural hinge joints at base of a column:





THANKS FOR YOUR ATTENTION

ARE THERE ANY QUESTIONS ?