(5) The rules given in 6.4 are principally formulated for the case of uniformly distributed loading. In special cases, such as footings, the load within the control perimeter adds to the resistance of the structural system, and may be subtracted when determining the design punching shear stress.

6.4.2 Load distribution and basic control perimeter

(1) The basic control perimeter u_1 may normally be taken to be at a distance 2,0d from the loaded area and should be constructed so as to minimise its length (see Figure 6.13).

The effective depth of the slab is assumed constant and may normally be taken as:

$$d_{eff} = \frac{(d_y + d_z)}{2} \quad (6.34)$$

where d_y and d_z are the effective depths of the reinforcement in two orthogonal directions.

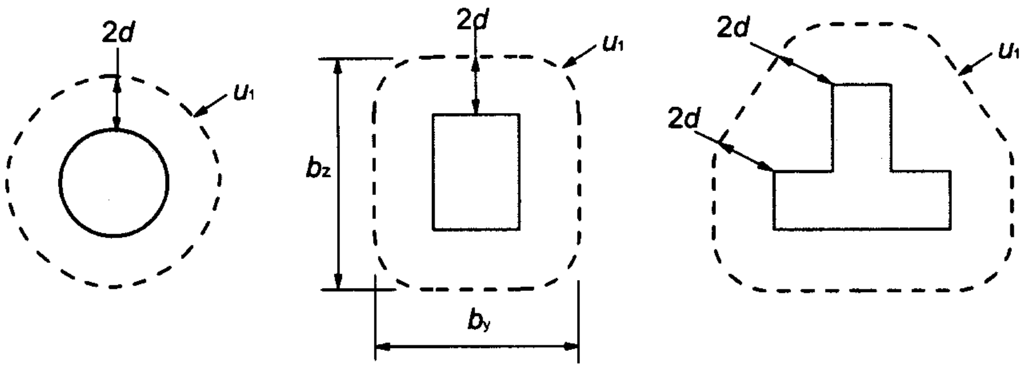


Figure 6.13: Typical basic control perimeters around loaded areas

(2) Control perimeters at a distance less than 2d should be considered where the concentrated force is opposed by a high distributed pressure (e.g. soil pressure on a base), or by the effects of a load or reaction within a distance 2,0 d of the periphery of area of application of the force.

(3) For loaded areas situated near openings, if the shortest distance between the perimeter of the loaded area and the edge of the opening does not exceed 6d, that part of the control perimeter contained between two tangents drawn to the outline of the opening from the centre of the loaded area is considered to be ineffective (see Figure 6.14).

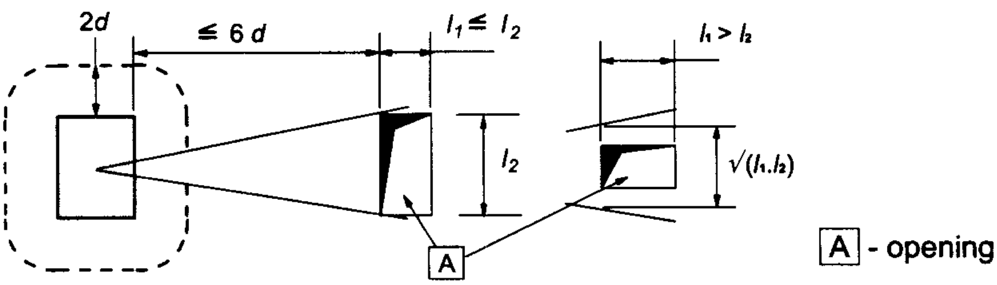


Figure 6.14: Control perimeter near an opening

(4) For a loaded area situated near an edge or a corner, the control perimeter should be taken as shown in Figure 6.15, if this gives a perimeter (excluding the unsupported edges) smaller than that obtained from (1) and (2) above.

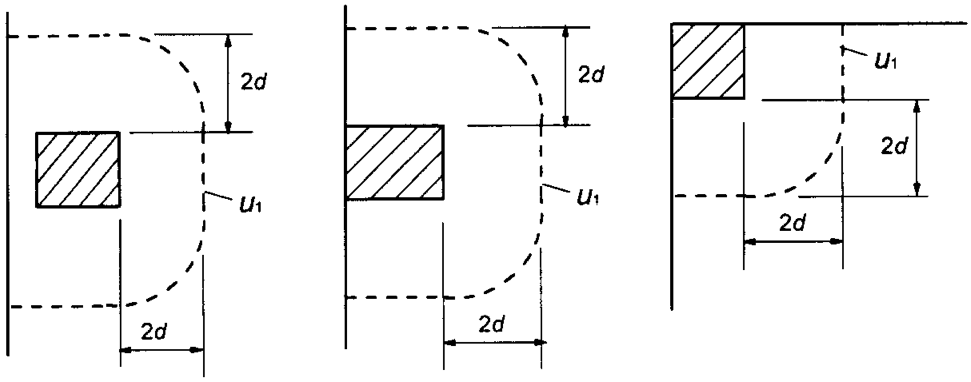


Figure 6.15: Control perimeters for loaded areas close to or at edge or corner

(5) For loaded areas situated near or on an edge or corner, i.e. at a distance smaller than d, special edge reinforcement should always be provided, see 9.3.1.4.

(6) The control section is that which follows the control perimeter and extends over the effective depth d. For slabs of constant depth, the control section is perpendicular to the middle plane of the slab. For slabs or footings of variable depth, the effective depth may be assumed to be the depth at the perimeter of the loaded area as shown in Figure 6.16.

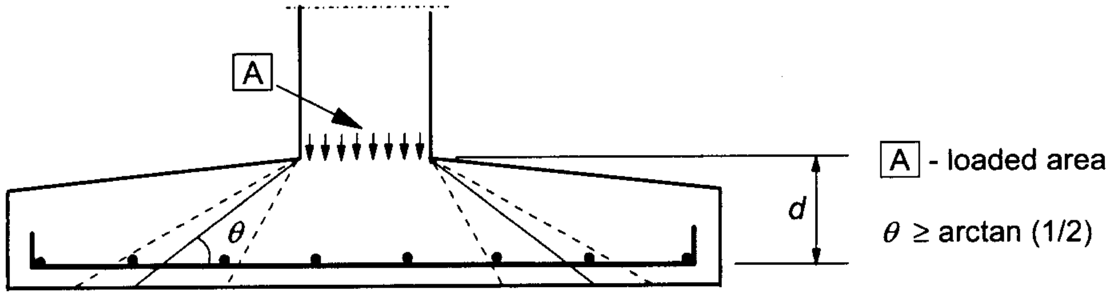


Figure 6.16: Depth of control section in a footing with variable depth

(7) Further perimeters, u_i, inside and outside the control area should have the same shape as the basic control perimeter.

(8) For slabs with circular column heads for which $l_H < 2{,}0h_H$ (see Figure 6.17) a check of the punching shear stresses according to 6.4.3 is only required on the control section outside the column head. The distance of this section from the centroid of the column r_{cont} may be taken as:

$$r_{cont} = 2{,}0d + l_H + 0{,}5c \qquad (6.35)$$

where:

l_H is the distance from the column face to the edge of the column head
c is the diameter of a circular column

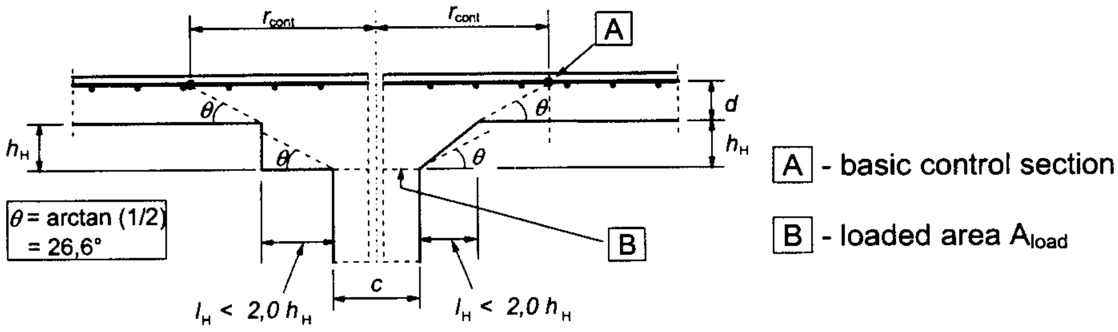


Figure 6.17: Slab with enlarged column head where $l_H < 2{,}0\ h_H$

For a rectangular column with a rectangular head with $l_H < 2{,}0d$ (see Figure 6.17) and overall dimensions l_1 and l_2 ($l_1 = c_1 + 2l_{H1}$, $l_2 = c_2 + 2l_{H2}$, $l_1 \leq l_2$), the value r_{cont} can be taken as the lesser of:

$$r_{cont} = 2{,}0d + 0{,}56\sqrt{l_1 l_2} \qquad (6.36)$$

and

$$r_{cont} = 2{,}0d + 0{,}69\ l_1 \qquad (6.37)$$

(9) For slabs with enlarged column heads where $l_H > 2{,}0h_H$ (see Figure 6.18) the critical sections both within the head and in the slab should be checked.

(10) The provisions of 6.4.2 and 6.4.3 also apply for checks within the column head with d taken as d_H according to Figure 6.18.

(11) The distances from the centroid of the column to the control sections in Figure 6.18 may be taken as:

$$r_{cont,ext} = l_H + 2{,}0d + 0{,}5c \qquad (6.38)$$

$$r_{cont,int} = 2{,}0(d + h_H) + 0{,}5c \qquad (6.39)$$

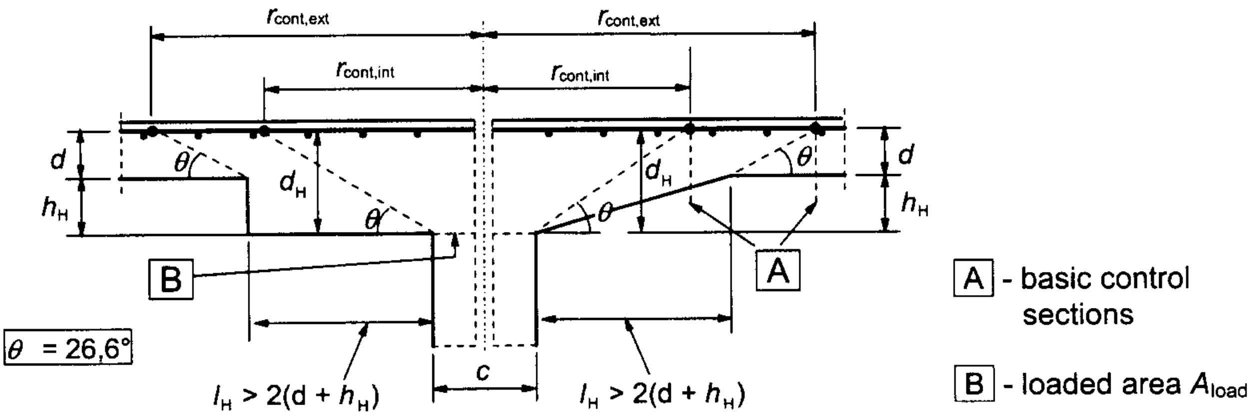


Figure 6.18: Slab with enlarged column head where $l_H > 2{,}0\ (d + h_H)$

6.4.3 Punching shear calculation

(1)P The design procedure for punching shear is based on checks at a series of control sections, which have a similar shape as the basic control section. The following design shear stresses, per unit area along the control sections, are defined:

$v_{Rd,c}$ is the design value of the punching shear resistance of a slab without punching shear reinforcement along the control section considered.

$v_{Rd,cs}$ is the design value of the punching shear resistance of a slab with punching shear reinforcement along the control section considered.

$v_{Rd,max}$ is the design value of the maximum punching shear resistance along the control section considered.

(2) The following checks should be carried out:

(a) At the column perimeter, or the perimeter of the loaded area, the maximum punching shear stress should not be exceeded:

$v_{Ed} < v_{Rd,max}$

(b) Punching shear reinforcement is not necessary if:

$v_{Ed} < v_{Rd,c}$

(c) Where v_{Ed} exceeds the value $v_{Rd,c}$ for the control section considered, punching shear reinforcement should be provided according to 6.4.5.

(3) Where the support reaction is eccentric with regard to the control perimeter, the maximum shear stress should be taken as:

$$v_{Ed} = \beta \frac{V_{Ed}}{u_i d} \qquad (6.40)$$

where

d mean effective depth of the slab, which may be taken as $(d_x + d_y)/2$ where:

d_x, d_y effective depths in the x- and y- directions of the control section

u_i length of the control perimeter being considered

β given by:

$$\beta = 1 + k \frac{M_{Ed}}{V_{Ed}} \cdot \frac{u_1}{W_1} \qquad (6.41)$$

where

u_1 is the length of the basic control perimeter

k is a coefficient dependent on the ratio between the column dimensions c_1 and c_2: its value is a function of the proportions of the unbalanced moment transmitted by uneven shear and by bending and torsion (see Table 6.2).

W_1 corresponds to a distribution of shear as illustrated in Figure 6.19 and is a function of the basic control perimeter u_1:

$$W_1 = \int_0^{u_i} |e| \, dl \qquad (6.42)$$

dl is the elementary length of the perimeter
e is the distance of dl from the axis about which the moment M_{Ed} acts

Table 6.2: Values of *k* for rectangular loaded areas

c_1/c_2	$\leq 0,5$	1,0	2,0	$\geq 3,0$
k	0,45	0,60	0,70	0,80

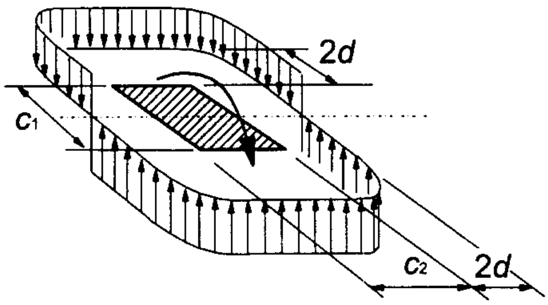


Figure 6.19: Shear distribution due to an unbalanced moment at a slab-internal column connection

For a rectangular column:

$$W_1 = \frac{c_1^2}{2} + c_1 c_2 + 4c_2 d + 16d^2 + 2\pi d c_1 \qquad (6.43)$$

where:

c_1 is the column dimension parallel to the eccentricity of the load
c_2 is the column dimension perpendicular to the eccentricity of the load

For internal circular columns β follows from:

$$\beta = 1 + 0,6\pi \frac{e}{D + 4d} \qquad (6.44)$$

For an internal rectangular column where the loading is eccentric to both axes, the following approximate expression for β may be used:

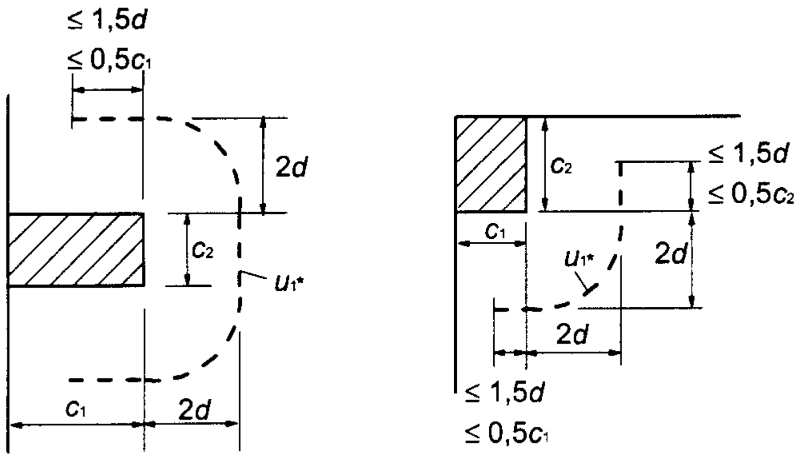
$$\beta = 1 + 1,8\sqrt{\left(\frac{e_y}{b_z}\right)^2 + \left(\frac{e_z}{b_y}\right)^2} \qquad (6.45)$$

where:

e_y and e_z are the eccentricities M_{Ed}/V_{Ed} along y and z axes respectively
b_y and b_z is the dimensions of the control perimeter (see Figure 6.13)
D is the diameter of the circular column.

Note: e_y results from a moment about the z axis and e_z from a moment about the y axis.

(4) For edge column connections, where the eccentricity perpendicular to the slab edge (resulting from a moment about an axis parallel to the slab edge) is toward the interior and there is no eccentricity parallel to the edge, the punching force may be considered to be uniformly distributed along the control perimeter u_{1^*} as shown in Figure 6.20(a).



a) edge column **b) corner column**

Figure 6.20: Equivalent control perimeter u_{1*}

Where there are eccentricities in both orthogonal directions, β may be determined using the following expression:

$$\beta = \frac{u_1}{u_{1*}} + k\frac{u_1}{W_1}e_{par} \tag{6.46}$$

where:

- u_1 is the full control perimeter (see Figure 6.15)
- u_{1*} is the reduced control perimeter (see Figure 6.20(a))
- e_{par} is the eccentricity parallel to the slab edge resulting from a moment about an axis perpendicular to the slab edge.
- k may be determined from Table 6.2 with the ratio c_1/c_2 replaced by $c_1/2c_2$
- W_1 is calculated for the full perimeter u_1 (see Figure 6.13).

For a rectangular column as shown in Figure 6.20(a):

$$W_1 = \frac{c_2^2}{4} + c_1c_2 + 4c_1d + 8d^2 + \pi dc_2 \tag{6.47}$$

If the eccentricity perpendicular to the slab edge is not toward the interior, Expression (6.41) applies. When calculating W_1 the eccentricity e should be measured from the centroid of the control perimeter.

(5) For corner column connections, where the eccentricity is toward the interior of the slab, it is assumed that the punching force is uniformly distributed along the reduced control perimeter u_{1*}, as defined in Figure 6.20(b). The β-value may then be considered as:

$$\beta = \frac{u_1}{u_{1*}} \tag{6.48}$$

If the eccentricity is toward the exterior, Expression (6.41) applies.

(6) For structures where the lateral stability does not depend on frame action between the slabs and the columns, and where the adjacent spans do not differ in length by more than 25%, approximate values for may be used.

Note: Values of β for use in a Country may be found in its National Annex. Recommended values are given in Figure 6.21N.

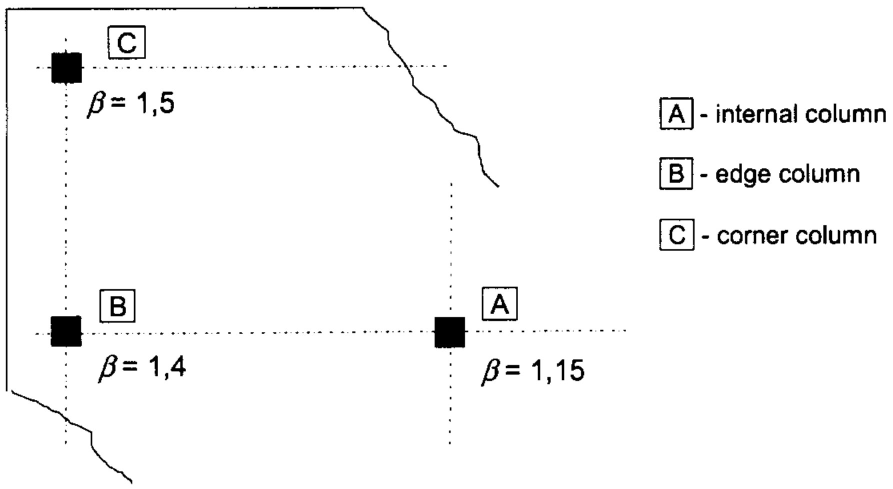


Figure 6.21N: Recommended values for β

(7) Where a concentrated load is applied close to a flat slab column support the resistance enhancement according to 6.2.2 (5) is not valid and should not be included.

(8) The punching shear force V_{Ed} in a foundation slab may be reduced due to the favourable action of the soil pressure.

(9) The vertical component V_{pd} resulting from inclined prestressing tendons crossing the control section may be taken into account as a favourable action where relevant.

6.4.4 Punching shear resistance for slabs or column bases without shear reinforcement

(1) The punching shear resistance of a *slab* should be assessed for the basic control section according to 6.4.2. The design punching shear stress is given by:

$$v_{Rd,c} = C_{Rd,c}\, k\, (100\rho_l\, f_{ck})^{1/3} + 0{,}10\sigma_{cp} \geq (v_{min} + 0{,}10\sigma_{cp}) \qquad (6.49)$$

where:

f_{ck} is in MPa

$$k = 1 + \sqrt{\frac{200}{d}} \leq 2{,}0 \quad d \text{ in mm}$$

$$\rho_l = \sqrt{\rho_{ly} \cdot \rho_{lz}} \leq 0{,}02$$

ρ_{ly}, ρ_{lz} relate to the bonded tension steel in x- and y- directions respectively. The values ρ_{ly} and ρ_{lz} should be calculated as mean values taking into account a slab width equal to the column width plus $3d$ each side.

$\sigma_{cp} = (\sigma_{cy} + \sigma_{cz})/2$

where

σ_{cy}, σ_{cz} are the normal concrete stresses in the critical section in y- and z-directions (MPa, positive if compression):

$$\sigma_{c,y} = \frac{N_{Ed,y}}{A_{cy}} \text{ and } \sigma_{c,z} = \frac{N_{Ed,z}}{A_{cz}}$$

N_{Edy}, N_{Edz} are the longitudinal forces across the full bay for internal columns and the longitudinal force across the control section for edge columns. The force may be from a load or prestressing action.

A_c is the area of concrete according to the definition of N_{Ed}

Note: The values of $C_{Rd,c}$ and v_{min} for use in a Country may be found in its National Annex. The recommended value for $C_{Rd,c}$ is $0{,}18/\gamma_c$ and that for v_{min} is given in Table 6.1N.

(2) The punching resistance of column bases should be verified at control perimeters within $2{,}0d$ from the periphery of the column. The lowest value of resistance found in this way should control the design.

For concentric loading the net applied force is

$$V_{Ed,red} = V_{Ed} - \Delta V_{Ed} \tag{6.50}$$

where:

V_{Ed} is the column load

ΔV_{Ed} is the net upward force within the control perimeter considered i.e. upward pressure from soil minus self weight of base.

$$v_{Ed} = V_{Ed,red}/ud \tag{6.51}$$

$$v_{Rd} = C_{Rd,c}k(100\,\rho f_{ck})^{1/3} \times 2d/a \geq v_{min} \times 2d/a \tag{6.52}$$

where

a is the distance from the periphery of the column to the control perimeter considered

$C_{Rd,c}$ is defined in 6.4.4(1)

v_{min} is defined in 6.4.4(1)

For eccentric loading

$$v_{Ed} = \frac{V_{Ed,red}}{ud}\left[1 + k\frac{M_{Ed}\,u}{V_{Ed,red}W}\right] \tag{6.53}$$

Where k is defined in 6.4.3 (4)

6.4.5 Punching shear resistance of slabs or column bases with shear reinforcement

(1) Where shear reinforcement is required it should be calculated in accordance with Expression (6.54):

$$v_{Rd,cs} = 0{,}75\, v_{Rd,c} + 1{,}5\,(d/s_r)\,A_{sw}\,f_{ywd,ef}\,(1/(u_1 d))\sin\alpha \tag{6.54}$$

where

A_{sw} is the area of one perimeter of shear reinforcement around the column

s_r is the radial spacing of perimeters of shear reinforcement

$f_{ywd,ef}$ is the effective design strength of the punching shear reinforcement, according to $f_{ywd,ef} = 250 + 0,25\, d \leq f_{ywd}$ (MPa.)

d is the mean effective depth of the slabs (mm)

α the angle between the shear reinforcement and the plane of the slab

If a single line of bent-down bars is provided, then the ration d/s_r in Expression (6.52) may be given the value 0,67.

(2) Detailing requirements for punching shear reinforcement are given in 9.4.3.

(3) Adjacent to the column the punching shear resistance is limited to a maximum of:

$$v_{Ed} = \frac{\beta V_{Ed}}{u_0 d} \leq v_{Rd,max} = 0,5\nu\, f_{cd} \tag{6.55}$$

where

u_0 for an interior column u_0 = length of column periphery

for an edge column $u_0 = c_2 + 3d \leq c_2 + 2c_1$

for a corner column $u_0 = 3d \leq c_1 + c_2$

c_1, c_2 are the column dimensions as shown in Figure 6.20

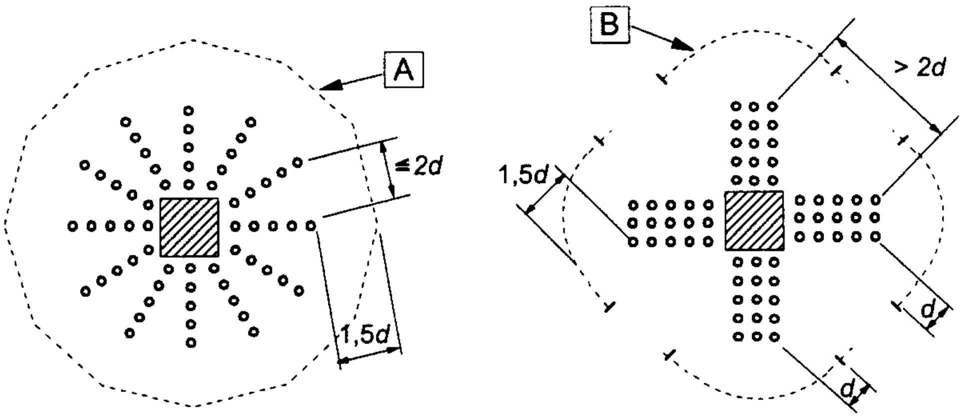
ν see Expression (6.6)

(4) The control perimeter at which shear reinforcement is not required, u_{out} (or $u_{out,ef}$ see Figure 6.22) should be calculated from Expression (6.56):

$$u_{out,ef} = \beta V_{Ed} / (v_{Rd,c}\, d) \tag{6.56}$$

The outermost perimeter of shear reinforcement should be placed at a distance not greater than 1,5d within u_{out} (or $u_{out,ef}$ see Figure 6.22).

(5) For types of shear reinforcement other than links, bent up-bars or mesh, $v_{Rd,cs}$ may be determined by tests.



A Perimeter u_{out} B Perimeter $u_{out,ef}$

Figure 6.22: Control perimeters at internal columns

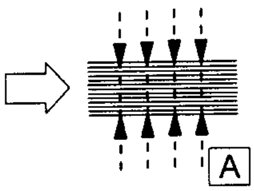
6.5 Design of struts, ties and nodes

6.5.1 General

(1)P Where non-linear strain distribution exists (e.g. supports, near concentrated loads or plain stress) strut-and-tie models may be used (see also 5.6.4).

6.5.2 Struts

(1) The design strength for a discrete concrete strut (e.g. column) may be calculated from Expression (6.57) (see Figure 6.23).



A transverse compressive stress or no transverse stress

Figure 6.23: Design strength of concrete struts without transverse tension

$$\sigma_{Rd,max} = f_{cd} \tag{6.57}$$

It may be appropriate to assume a higher design strength in regions where multi-axial compression exists.

(2) The design strength for notional concrete struts should be reduced in cracked compression zones and, unless a more rigorous approach is used, may be calculated from Expression (6.58) (see Figure 6.24).

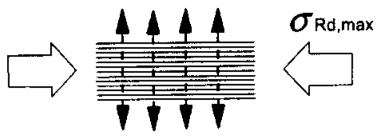


Figure 6.24: Design strength of concrete struts with transverse tension

$$\sigma_{Rd,max} = \nu' f_{cd} \tag{6.58}$$

where

$$\nu' = k(1 - f_{ck}/250) \tag{6.59}$$

Note: The value of k for use in a Country may be found in its National Annex. The recommended value for situations where the cotangent of the angle between the notional strut and any intersecting tie exceeds 2,5 is 0,60.

(3) For struts between directly loaded areas, such as corbels or short deep beams, more accurate calculation methods are given in 6.2.3.

6.5.3 Ties

(1) The design strength of transverse ties and reinforcement should be limited in accordance with 3.2 and 3.3.

(2) Reinforcement should be adequately anchored in the nodes.

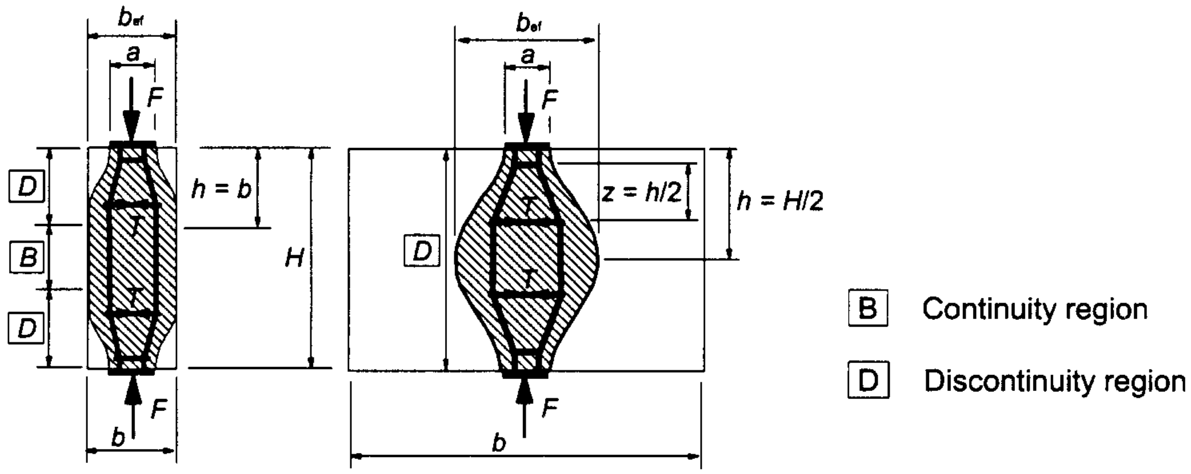
(3) Where smeared nodes (see Figure 6.25a and b) extend over a considerable length of a structure, the reinforcement in the node area should be distributed over the length where the compression trajectories are curved (ties and struts). The tensile force T may be obtained by:

a) for partial discontinuity regions $\left(b \le \frac{h}{2}\right)$, see Figure 6.25 a:

$$T = \frac{1}{4}\frac{b-a}{b}F \tag{6.60}$$

b) for full discontinuity regions $(b \ge b_{ef})$, see Figure 6.25 b:

$$T = \frac{1}{4}\left(1-0{,}7\frac{a}{h}\right)F \tag{6.61}$$



$b_{ef} = b$ $b_{ef} = 0{,}5H + 0{,}65a$; $a \le H$

a) Partial discontinuity **b) Full discontinuity**

Figure 6.25: Transverse tensile forces in a compression field with concentrated nodes

6.5.4 Nodes

(1)P The rules of this section also apply to regions where concentrated forces are transferred in a member and which are not designed by the strut-and-tie method.

(2)P The forces acting at nodes shall be in equilibrium. Transverse tensile forces perpendicular to an in-plane node shall be considered.

(3)P Reinforcement resisting nodal forces shall be adequately anchored.

(4) The dimensioning and detailing of concentrated nodes are critical in determining their load-bearing resistance. Concentrated nodes may develop, e.g. where point loads are applied, at supports, in anchorage zones with concentration of reinforcement or prestressing tendons, at bends in reinforcing bars, and at connections and corners of members.

(5) The design values for the compressive stresses within nodes may be determined by:

a) in compression nodes where no ties are anchored at the node (see Figure 6.26)

$$\sigma_{Rd,max} = 1,0\ \nu' f_{cd} \tag{6.62}$$

where $\sigma_{Rd,max}$ is the maximum of $\sigma_{Rd,1}$, $\sigma_{Rd,2}$, and $\sigma_{Rd,3}$.

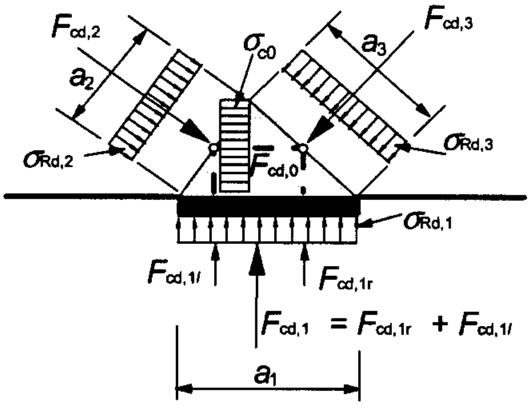


Figure 6.26: Compression node without ties

b) in compression - tension nodes with anchored ties provided in one direction (see Figure 6.27),

$$\sigma_{Rd,max} = 0,85\nu' f_{cd} \tag{6.63}$$

where $\sigma_{Rd,max}$ is the maximum of $\sigma_{Rd,1}$ and $\sigma_{Rd,2}$.

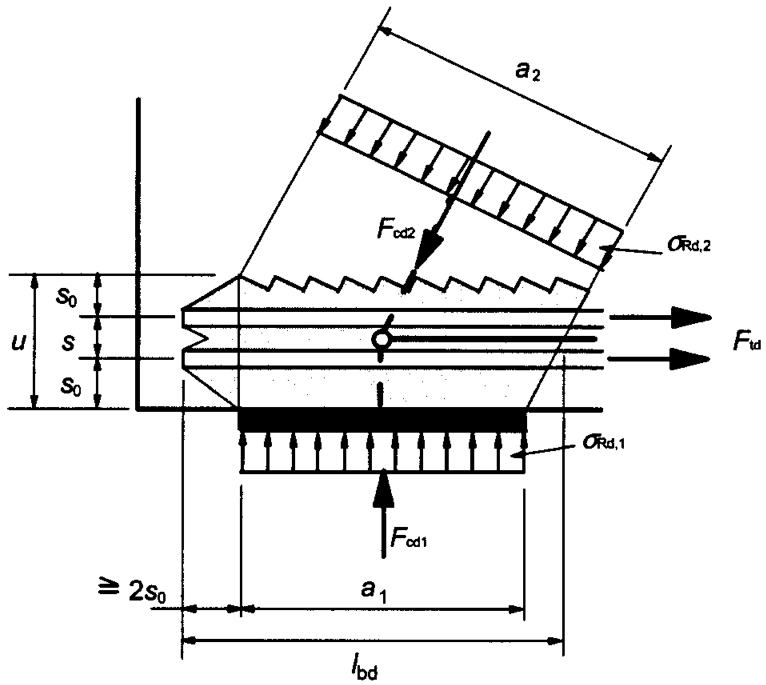


Figure 6.27: Compression tension node with reinforcement provided in one direction

c) in compression - tension nodes with anchored ties provided in more than one direction (see Figure 6.28),

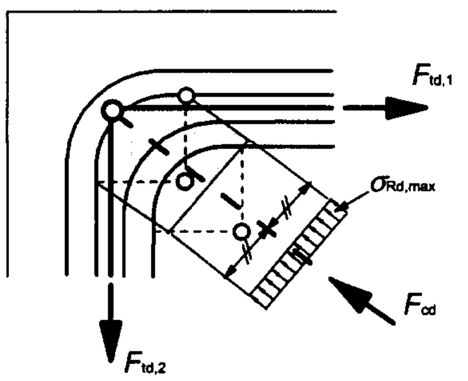


Figure 6.28: Compression tension node with reinforcement provided in two directions

$$\sigma_{Rd,max} = 0{,}75\ \nu f_{cd} \qquad (6.64)$$

(6) Under the conditions listed below, the design compressive stress values given in (5)P may be increased by up to10% where at least one of the following apply:

- triaxial compression is assured,
- all angles between struts and ties are ≥ 55°,
- the stresses applied at supports or at point loads are uniform, and the node is confined by stirrups,
- the reinforcement is arranged in multiple layers,
- the node is reliably confined by means of bearing arrangement or friction.

(7) Triaxially compressed nodes may be checked according to Expression (3.24) and (3.25) with $\sigma_{Rd,max} \leq 3\nu f_{cd}$ if for all three directions of the struts the distribution of load is known.

(8) The anchorage of the reinforcement in compression-tension nodes starts at the beginning of the node, e.g. in case of a support anchorage starting at its inner face (see Figure 6.27). The anchorage length should extend over the entire node length. In certain cases, the reinforcement may also be anchored behind the node. For anchorage and bending of reinforcement, see sections 8.4 to 8.6.

(9) In-plane compression nodes at the junction of three struts may be verified in accordance with Figure 6.26. The maximum average principal node stresses (σ_{c0}, σ_{c1}, σ_{c2}, σ_{c3}) should be checked in accordance with (5)P a). Normally the following may be assumed: $F_1/a_1 = F_2/a_2 = F_3/a_3$ resulting in $\sigma_c = \sigma_{c2} = \sigma_{c3} = \sigma_{co}$.

(10) Nodes at reinforcement bends may be analysed in accordance with Figure 6.28. The average stresses in the struts should be checked in accordance with 6.5.4 (5)P. The diameter of the mandrel should be checked in accordance with 8.4.

6.6 Anchorages and laps

(1)P The design bond stress is limited to a value depending on the surface characteristics of the reinforcement, the tensile strength of the concrete and confinement of surrounding concrete. This depends on cover, transverse reinforcement and transverse pressure.

(2) The length necessary for developing the required tensile force in an anchorage or lap is calculated on the basis of a constant bond stress.

(3) Application rules for the design and detailing of anchorages and laps are given in 8.4 to 8.8.

6.7 Partially loaded areas

(1)P For partially loaded areas, local crushing (see below) and transverse tension forces (see 6.5) shall be considered.

(2) For a uniform distribution of load on an area A_{c0} (see Figure 6.26) the concentrated resistance force may be determined as follows:

$$F_{Rdu} = A_{c0} \cdot f_{cd} \cdot \sqrt{A_{c1}/A_{c0}} \leq 3{,}0 \cdot f_{cd} \cdot A_{c0} \qquad (6.65)$$

where:
A_{c0} is the loaded area,
A_{c1} is the maximum design distribution area with a similar shape to A_{c0}

(3) The design distribution area A_{c1} required for the resistance force F_{Rdu} should correspond to the following conditions:
- The height for the load distribution in the load direction should correspond to the conditions given in Figure 6.29
- the centre of the design distribution area A_{c1} should be on the line of action of the centre of the load area A_{c0}.
- If there is more than one compression force acting on the concrete cross section, the designed distribution areas should not overlap.

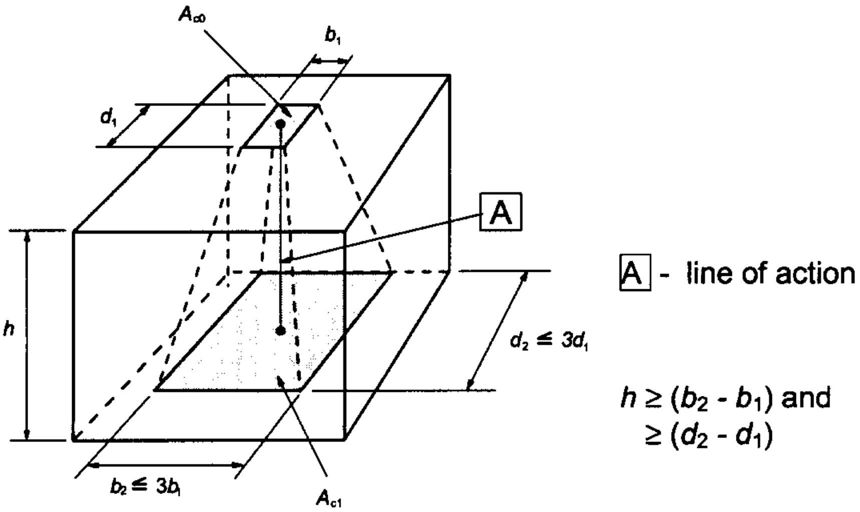


Figure 6.29: Design distribution for partially loaded areas

The value of F_{Rdu} should be reduced if the load is not uniformly distributed on the area A_{c0} or if high shear forces exist.

6.8 Fatigue

6.8.1 Verification conditions

(1)P The resistance of structures to fatigue shall be verified in special cases. This verification shall be performed separately for concrete and steel.

(2) A fatigue verification is necessary for structures and structural components which are subjected to regular load cycles (e.g. crane-rails, bridges exposed to high traffic loads).

6.8.2 Internal forces and stresses for fatigue verification

(1)P The stress calculation shall be based on the assumption of cracked cross sections neglecting the tensile strength of concrete but satisfying compatibility of strains.

(2)P The effect of different bond behaviour of prestressing and reinforcing steel shall be taken into account by increasing the stress range in the reinforcing steel calculated under the assumption of perfect bond by the factor:

$$\eta = \frac{A_S + A_P}{A_S + A_P\sqrt{\xi(\phi_S / \phi_P)}} \qquad (6.66)$$

where:

A_s is the area of reinforcing steel
A_P is the area of prestressing tendon or tendons
ϕ_S is the largest diameter of reinforcement
ϕ_P is the diameter or equivalent diameter of prestressing steel
$\phi_P = 1,6\sqrt{A_P}$ for bundles
$\phi_P = 1,75\ \phi_{wire}$ for single 7 wire strands
$\phi_P = 1,20\ \phi_{wire}$ for single 3 wire strands
ξ is the ratio of bond strength between bonded tendons and ribbed steel in concrete. The value is subject to the relevant European Technical Approval. In the absence of this the values given in Table 6.3 may be used.

Table 6.3: Ratio of bond strength, ξ, between tendons and reinforcing steel

prestressing steel	ξ		
	pre-tensioned	bonded, post-tensioned	
		≤ C50/60	≥ C55/67
smooth bars and wires	Not applicable	0,3	0,15
strands	0,6	0,5	0,25
indented wires	0,7	0,6	0,3
ribbed bars	0,8	0,7	0,35

(3) In shear design the inclination of the compressive struts θ_{fat} may be calculated in accordance with Expression (6.67).

$$\tan\theta_{fat} = \sqrt{\tan\theta} \leq 1{,}0 \quad (6.67)$$

where:

θ is the angle of concrete compression struts to the beam axis assumed in ULS design (see 6.2.3)

6.8.3 Combination of actions

(1)P For the calculation of the stress ranges the action shall be divided into non-cycling and fatigue-inducing cyclic actions.

(2)P The basic combination of the non-cyclic load is equivalent to the definition of the frequent combination for SLS:

$$E_d = E\{G_{k,j};P;\psi_{1,1}Q_{k,1};\psi_{2,i}Q_{k,i}\}\ j \geq 1;\ i > 1 \quad (6.68)$$

The combination of actions in bracket { }, (called the basic combination), can be expressed as:

$$\sum_{j\geq 1} G_{k,j}\ "+"\ P\ "+"\ \psi_{1,1}Q_{k,1}\ "+"\ \sum_{i>1}\psi_{2,i}Q_{k,i} \quad (6.69)$$

Note: $Q_{k,1}$ and $Q_{k,i}$ are non-cyclic, non-permanent actions

(3)P The cyclic action shall be combined with the unfavourable basic combination:

$$E_d = E\{\{G_{k,j};P;\psi_{1,1}Q_{k,1};\psi_{2,i}Q_{k,i}\}\ Q_{fat}\}\ j \geq 1;\ i > 1 \quad (6.70)$$

The combination of actions in bracket { }, (called the basic combination plus the cyclic action), can be expressed as:

$$\left(\sum_{j\geq 1} G_{k,j}\ "+"\ P\ "+"\ \psi_{1,1}Q_{k,1}\ "+"\ \sum_{i>1}\psi_{2,i}Q_{k,i}\right)\ "+"\ Q_{fat} \quad (6.71)$$

where:

Q_{fat} relevant fatigue load (e.g. traffic load as defined in EN 1991 or other cyclic load)

6.8.4 Verification procedure for reinforcing and prestressing steel

(1) The damage of a single load amplitude $\Delta\sigma$ may be determined by using the corresponding S-N curves (Figure 6.30) for reinforcing and prestressing steel. the applied load should be multiplied by γ_F and γ_{ED}. be The resisting stress range at N^* cycles $\Delta\sigma_{Rsk}$ obtained should be divided by the safety factor $\gamma_{S,fat}$.

Note 1: The values of γ_F and of γ_{ED} for use in a Country may be found in its National Annex. The recommended value for both is 1,0.

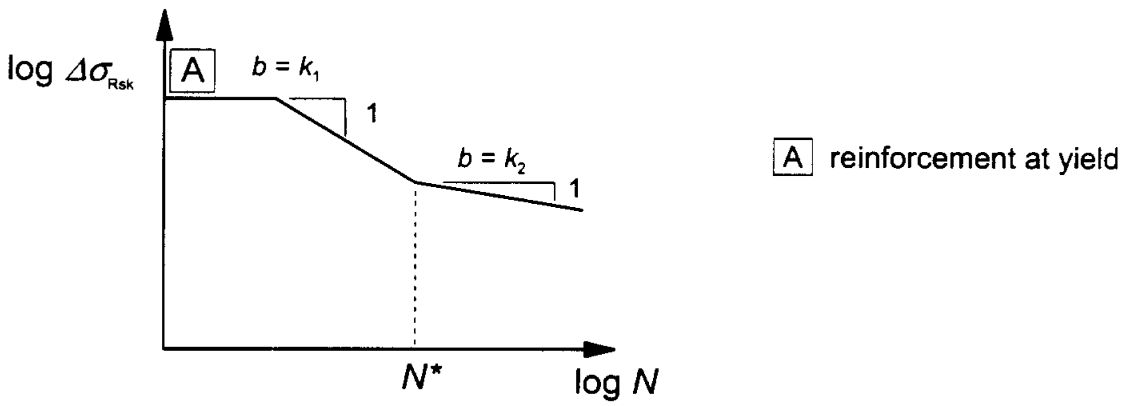


Figure 6.30: Shape of the characteristic fatigue strength curve (S-N-curves for reinforcing and prestressing steel)

Note 2: The values of parameters for reinforcing steels S-N curves for use in a Country may be found in its National Annex. The recommended values are given in Table 6.4N and 6.5N which apply for reinforcing and prestressing steel respectively.

Table 6.4N: Parameters for S-N curves for reinforcing steel

Type of reinforcement		**stress exponent**		**$\Delta\sigma_{Rsk}$ (MPa)**
	N^*	**k_1**	**k_2**	**at N^* cycles**
Straight and bent bars[1]	10^6	5	9	162,5
Welded bars and wire fabrics[2]	10^7	3	5	58,5
Splicing devices [2]	10^7	3	5	35

Note 1: Values for $\Delta\sigma_{Rsk}$ are those for straight bars. Values for bent bars should be obtained using a reduction factor $\zeta = 0,35 + 0,026\ D/\phi$.
where:
D diameter of the mandrel
ϕ bar diameter

Table 6.5N: Parameters for S-N curves of prestressing steel

S-N curve of prestressing steel used for		**stress exponent**		**$\Delta\sigma_{Rsk}$ (MPa)**
	N^*	**k_1**	**k_2**	**at N^* cycles**
pre-tensioning	10^6	5	9	185
post-tensioning				
– single strands in plastic ducts	10^6	5	9	185
– straight tendons or curved tendons in plastic ducts	10^6	5	10	150
– curved tendons in steel ducts	10^6	5	7	120
– splicing devices[1]	10^6	5	5	80

Note 1: Unless other S-N curves can be justified by test results or documented by the supplier.

(2) For multiple amplitudes the effects of damage can be added by using the Palmgren-Miner Rule. Hence, the fatigue damage factor D_{Ed} of steel caused by the relevant fatigue loads shall satisfy the condition:

$$D_{Ed} = \sum_i \frac{n(\Delta\sigma_i)}{N(\Delta\sigma_i)} < 1 \tag{6.72}$$

where:
$n(\Delta\sigma_i)$ is the applied number of cycles for a stress range $\Delta\sigma_i$
$N(\Delta\sigma_i)$ is the resisting number of cycles for a stress range $\Delta\sigma_i$

(3)P If prestressing or reinforcing steel is exposed to fatigue loads, the calculated stresses shall not exceed the design yield strength of the steel.

(4) The proportionality limit should be verified by tensile tests for the steel used.

(5) When the rules of 6.8 are used to evaluate the remaining life of existing structures, or to assess the need for strengthening, once corrosion has started the stress range can be determined by reducing the stress exponent k_2 for straight and bent bars.

Note: The value of k_2 for use in a Country may be found in its National Annex. The recommended values is 5.

(6)P The stress range of welded bars shall never exceed the stress range of straight and bent bars.

6.8.5 Verification using damage equivalent stress range

(1)P Instead of an explicit verification of the operational strength according to 6.8.4 the fatigue verification of standard cases with known loads (railway and road bridges) may also be performed as follows:

- by damage equivalent stress ranges for steel according to 6.8.5 (3)
- damage equivalent compression stresses for concrete according to 6.8.7

(2) EN 1992-2 gives relevant fatigue loading models and procedures for the calculation of the equivalent stress range $\Delta\sigma_{S,equ}$ for superstructures of road and railway bridges. In the equivalent stress range the real operational loading is condensed to a single amplitude at N^* cycles.

(3) For reinforcing or prestressing steel and splicing devices adequate fatigue resistance shall be assumed if the Expression (6.73) is satisfied:

$$\gamma_f \cdot \gamma_{Sd} \cdot \Delta\sigma_{S,equ}(N^*) \leq \frac{\Delta\sigma_{Rsk}(N^*)}{\gamma_{s,fat}} \tag{6.73}$$

where:

$\Delta\sigma_{Rsk}(N^*)$ is the stress range at N^* cycles from the appropriate S-N curves given in Figure 6.30.

Note: See also Tables 6.4N and 6.5N.

$\Delta\sigma_{S,equ}(N^*)$ is the damage equivalent stress range for different types of reinforcement and considering the number of loading cycles N^*.

6.8.6 Other verifications

(1) Adequate fatigue resistance may be assumed for unwelded reinforcing bars under tension, if the stress range under frequent cyclic load combined with the basic combination is $\Delta\sigma_S \leq 70$ MPa.

For welded reinforcing bars under tension adequate fatigue resistance may be assumed if the stress range under frequent load combination is $\Delta\sigma_S \leq 35$ MPa.

(2) Where welded joints or splicing devices are used, no tension should exist in the concrete section within 200 mm of the prestressing tendons or reinforcing steel under the frequent load combination together with a reduction factor of 0,9 for the mean value of prestressing force, $P_{m,}$

6.8.7 Verification of concrete using damage equivalent stress

(1) A satisfactory fatigue resistance may be assumed for concrete under compression, if the following condition is fulfilled:

$$S_{cd,max,equ} + 0{,}43\sqrt{(1 - R_{equ})} \leq 1 \tag{6.74}$$

where:

$$R_{equ} = \frac{S_{cd,min,equ}}{S_{cd,max,equ}} \tag{6.75}$$

$$S_{cd,min,equ} = \frac{\sigma_{cd,min,equ}}{f_{cd,fat}} \tag{6.76}$$

$$S_{cd,max,equ} = \frac{\sigma_{cd,max,equ}}{f_{cd,fat}} \tag{6.77}$$

where :

$f_{cd,fat}$ is the design fatigue strength of concrete according to 6.7.7
$\sigma_{cd,max,equ}$ is the upper stress of the ultimate amplitude for
$\sigma_{cd,min,equ}$ is the lower stress of the ultimate amplitude for $N = 10^6$ cycles

Note: The value of N ($\leq 10^6$ cycles) for use in a Country may be found in its National Annex. The recommended value is $N = 10^6$ cycles.

$$f_{cd,fat} = 0{,}85\beta_{cc}(t_0)f_{cd}\left(1 - \frac{f_{ck}}{250}\right) \tag{6.78}$$

where:

$\beta_{cc}(t_0)$ coefficient for concrete strength at first load application (see 3.1.2 (6))
t_0 time of first loading of concrete in days

(2) The fatigue verification for concrete under compression may be assumed, if the following condition is satisfied:

$$\frac{\sigma_{c,max}}{f_{cd,fat}} \le 0{,}5 + 0{,}45\frac{\sigma_{c,min}}{f_{cd,fat}} \tag{6.79}$$

$$\le 0{,}9 \text{ for } f_{ck} \le 50 \text{ MPa}$$
$$\le 0{,}8 \text{ for } f_{ck} > 50 \text{ MPa}$$

where:

$\sigma_{c,max}$ is the maximum compressive stress at a fibre under the frequent load combination (compression measured positive)

$\sigma_{c,min}$ is the minimum compressive stress at the same fibre where $\sigma_{c,max}$ occurs. If $\sigma_{c,min}$ > 0 (tension), then $\sigma_{c,min}$ = 0.

(3) Expression (6.79) also applies to the compression struts of members subjected to shear. In this case the concrete strength $f_{cd,fat}$ should be reduced by the effectiveness factor ν (see Expression (6.6)).

(4) For members not requiring design shear reinforcement for the ultimate limit state it may be assumed that the concrete resists fatigue due to shear effects where the following apply:

- for $\frac{V_{Ed,min}}{V_{Ed,max}} \ge 0$:

$$\frac{|V_{Ed,max}|}{|V_{Rd,ct}|} \le 0{,}5 + 0{,}45\frac{|V_{Ed,min}|}{|V_{Rd,ct}|}\begin{cases} \le 0{,}9 \text{ up to C50/60} \\ \le 0{,}8 \text{ greater than C55/67} \end{cases} \tag{6.80}$$

- for $\frac{V_{Ed,min}}{V_{Ed,max}} < 0$:

$$\frac{|V_{Ed,max}|}{|V_{Rd,ct}|} \le 0{,}5 - \frac{|V_{Ed,min}|}{|V_{Rd,ct}|} \tag{6.81}$$

where:

$V_{Ed,max}$ is the design value of the maximum applied shear force under frequent load combination

$V_{Ed,min}$ is the design value of the minimum applied shear force under frequent load combination in the cross-section where $V_{Ed,max}$ occurs

$V_{Rd,ct}$ is the design value for shear-resistance according to (6.2.a).

SECTION 7 SERVICEABILITY LIMIT STATES

7.1 General

(1)P This section covers the common serviceability limit states. These are:

- stress limitation (see 7.2)
- crack control (see 7.3)
- deflection control (see 7.4)

Other limit states (such as vibration) may be of importance in particular structures but are not covered in this Standard.

7.2 Stresses

(1)P Compressive stresses in the concrete shall be limited in order to avoid longitudinal cracks, micro-cracks or high levels of creep, where they could result in unacceptable effects on the function of the structure.

(2)P Stresses in the reinforcement shall be limited in order to avoid inelastic strain, to avoid unacceptable cracking or deformation.

(3) In the calculation of stresses and deflections, cross sections should be assumed to be uncracked provided that the flexural tensile stress does not exceed $f_{ct,eff}$. The value of $f_{ct,eff}$ may be taken as f_{ctm} or $f_{ctm,fl}$ provided that the calculation for minimum tension reinforcement is also based on the same value. For the purposes of calculating crack widths and tension stiffening f_{ctm} should be used.

7.3 Cracking

7.3.1 General considerations

(1)P Cracking shall be limited to an extent that will not impair the proper functioning or durability of the structure or cause its appearance to be unacceptable.

(2)P Cracking is normal in reinforced concrete structures subject to bending, shear, torsion or tension resulting from either direct loading or restraint of imposed deformations.

(3) Cracks may also arise from other causes such as plastic shrinkage or expansive chemical reactions within the hardened concrete. Such cracks may be unacceptably large but their avoidance and control lie outside the scope of this Section.

(4) Cracks may be permitted to form without any attempt to control their width, provided they do not impair the functioning of the structure.

(5) A limiting calculated crack width, w_{max}, taking into account of the proposed function and nature of the structure and the costs of limiting cracking, should be established.

Note: The value of w_{max} for use in a Country may be found in its National Annex. The recommended values for relevant exposure classes are given in Table 7.1N.

Table 7.1N Recommended values of w_{max}

Exposure Class	Reinforced members and prestressed members with unbonded tendons	Prestressed members with bonded tendons
	Quasi-permanent load combination	Frequent load combination
X0, XC1	0,4[1]	0,2
XC2, XC3, XC4	0,3	0,2[2]
XD1, XD2, XS1, XS2, XS3		Decompression

Note 1: For X0, XC1 exposure classes, crack width has no influence on durability and this limit is set to guarantee acceptable appearance. In the absence of appearance conditions this limit may be relaxed.
Note 2: For these exposure classes, in addition, decompression should be checked under the quasi-permanent combination of loads.

In the absence of specific requirements (e.g. water-tightness), it may be assumed that limiting the calculated crack widths to the values of w_{max} given in Table 7.1N, under the quasi-permanent combination of loads, will generally be satisfactory for reinforced concrete members in buildings with respect to appearance and durability.

The durability of prestressed members may be more critically affected by cracking. In the absence of more detailed requirements, it may be assumed that limiting the calculated crack widths to the values of w_{max} given in Table 7.1N, under the frequent combination of loads, will generally be satisfactory for prestressed concrete members. The decompression limit requires that all parts of the tendons or duct lie at least 25 mm within concrete in compression.

(6) For members with only unbonded tendons, the requirements for reinforced concrete elements apply. For members with a combination of bonded and unbonded tendons requirements for prestressed concrete members with bonded tendons apply.

(7) Special measures may be necessary for members subjected to exposure class XD3. The choice of appropriate measures will depend upon the nature of the aggressive agent involved.

(8) When using strut-and-tie models oriented according to stress trajectories from elastic analysis, it is possible to use the forces in the ties to obtain the corresponding steel stresses to estimate the crack width.

(9) Crack widths may be calculated according to 7.3.4. A simplified alternative is to limit the bar size or spacing according to 7.3.3.

7.3.2 Minimum reinforcement areas

(1)P If crack control is required, a minimum amount of bonded reinforcement is required to control cracking in areas where tension is expected. The amount may be estimated from equilibrium between the tensile force in concrete just before cracking and the tensile force in reinforcement at yielding or at a lower stress if necessary to limit the crack width.

(2) Unless a more rigorous calculation shows lesser areas to be adequate, the required minimum areas of reinforcement may be calculated as follows. In profiled cross sections like T-beams and box girders, minimum reinforcement should be determined for the individual parts of the section (webs, flanges).

$$A_{s,min}\sigma_s = k_c\, k\, f_{ct,eff}\, A_{ct} \tag{7.1}$$

where:

$A_{s,min}$ is the area of reinforcing steel within tensile zone

A_{ct} is the area of concrete within tensile zone. The tensile zone is that part of the section which is calculated to be in tension just before formation of the first crack

σ_s is the absolute value of the maximum stress permitted in the reinforcement immediately after formation of the crack. This may be taken as the yield strength of the reinforcement, f_{yk}. A lower value may, however, be needed to satisfy the crack width limits according to the maximum bar size (Table 7.2) or the maximum bar spacing (Table 7.3)

$f_{ct,eff}$ is the the mean value of the tensile strength of the concrete effective at the time when the cracks may first be expected to occur:

$f_{ct,eff} = f_{ctm}$ or lower, ($f_{ctm}(t)$), if cracking is expected earlier than 28 days

k is the coefficient which allows for the effect of non-uniform self-equilibrating stresses, which lead to a reduction of restraint forces
= 1,0 for webs with $h \leq 300$ mm or flanges with widths less than 300 mm
= 0,65for webs with $h \geq 800$ mm or flanges with widths greater than 800 mm
intermediate values may be interpolated

k_c is a coefficient which takes account of the nature of the stress distribution within the section immediately prior to cracking and of the change of the lever arm:

For pure tension:
$k_c = 1,0$

For bending or bending combined with axial forces:

- For rectangular sections and webs of box sections and T-sections:

$$k_c = 0,4 \cdot \left[1 - \frac{\sigma_c}{k_1(h/h^*)f_{ct,eff}}\right] \leq 1 \tag{7.2}$$

- For flanges of box sections and T-sections:

$$k_c = 0,9\frac{F_{cr}}{A_{ct}f_{ct,eff}} \geq 0,5 \tag{7.3}$$

where

σ_c is the mean stress of the concrete acting on the part of the section under consideration:

$$\sigma_c = \frac{N_{Ed}}{bh} \tag{7.4}$$

N_{Ed} is the axial force at the serviceability limit state acting on the part of the cross-section under consideration (compressive force positive). N_{Ed} should be determined considering the characteristic values of prestress and axial forces under the relevant combination of actions

h^* $h^* = h$ for $h < 1{,}0$ m
$h^* = 1{,}0$ m for $h \geq 1{,}0$ m

k_1 is a coefficient considering the effects of axial forces on the stress distribution:
$k_1 = 1{,}5$ if N_{Ed} is a compressive force
$k_1 = \dfrac{2h^*}{3h}$ if N_{Ed} is a tensile force

- For flanges of box sections and T-sections:

$$k_c = 0{,}9 \frac{F_{cr}}{A_{ct} f_{ct,eff}} \geq 0{,}5 \qquad (7.3)$$

where

F_{cr} is the absolute value of the tensile force within the flange immediately prior to cracking due to the cracking moment calculated with $f_{ct,eff}$

(3) Bonded tendons in the tension zone may be assumed to contribute to crack control within a distance ≤ 150 mm from the centre of the tendon. This may be taken into account by adding the term $\xi_1 A_p \Delta\sigma_p$ to the left hand side of Expression (7.1),

where

A_p is the area of pre or post-tensioned tendons within the $A_{c,eff}$

ξ_1 is the adjusted ratio of bond strength taking into account the different diameters of prestressing and reinforcing steel:

$$= \sqrt{\xi \cdot \frac{\phi_s}{\phi_p}} \qquad (7.5)$$

ξ ratio of bond strength of prestressing and reinforcing steel, according to Table 6.2 in 6.8.2.

ϕ_s largest bar diameter of reinforcing steel

ϕ_p equivalent diameter of tendon according to 6.8.2

If only prestressing steel is used to control cracking, $\xi_1 = \sqrt{\xi}$.

$\Delta\sigma_p$ Stress variation in prestress tendons from the state of zero strain of the concrete at the same level

(4) In prestressed members no minimum reinforcement is required in sections where, under the characteristic combination of loads and the characteristic value of prestress, the concrete remains in compression.

7.3.3 Control of cracking without direct calculation

(1) For reinforced or prestressed slabs in buildings subjected to bending without significant axial tension, specific measures to control cracking are not necessary where the overall depth does not exceed 200 mm and the provisions of 9.3 have been applied.

(2) Where the minimum reinforcement given by 7.3.2 is provided, crack widths are not likely to be excessive if:

- for cracking caused dominantly by restraint, the bar sizes given in Table 7.2 are not exceeded where the steel stress is the value obtained immediately after cracking (i.e. σ_s in Expression (7.1)).
- for cracks caused mainly by loading, either the provisions of Table 7.2 or the provisions of Table 7.3 are complied with. The steel stress should be calculated on the basis of a cracked section under the relevant combination of actions.

For pre-tensioned concrete, where crack control is mainly provided by tendons with direct bond, Tables 7.2 and 7.3 may be used with a stress equal to the total stress minus prestress. For post-tensioned concrete, where crack control is provided mainly by ordinary reinforcement, the tables may be used with the stress in this reinforcement calculated with the effect of prestressing forces included.

Table 7.2 Maximum bar diameters ϕ^*_s for crack control

Steel stress* [MPa]	Maximum bar size [mm] w_k=0,4 mm	w_k=0,3 mm	w_k=0,2 mm
160	40	32	25
200	32	25	16
240	20	16	12
280	16	12	8
320	12	10	6
360	10	8	5
400	8	6	4
450	6	5	-

* **Note:** Under the relevant combinations of actions

The maximum bar diameter may be modified as follows:

$$\phi_s = \phi^*_s\,(f_{ct,eff}/2,9)\,\frac{k_c h_{cr}}{2\,(h-d)} \quad \text{Bending (at least part of section in compression)} \tag{7.6}$$

$$\phi_s = \phi^*_s(f_{ct,eff}/2,9)\,\frac{k_c h_{cr}}{(h-d)} \quad \text{Tension (all of section under tensile stress)} \tag{7.7}$$

where:

ϕ_s is the adjusted maximum bar diameter
ϕ^*_s is the maximum bar size given in the Table 7.2
h is the overall depth of the section
h_{cr} is the depth of the tensile zone immediately prior to cracking, considering the characteristic values of prestress and axial forces under the quasi-permanent combination of actions
d is the effective depth to the centroid of the outer layer of reinforcement

Table 7.3 Maximum bar spacing for crack control

Steel stress * [MPa]	Maximum bar spacing [mm] w_k=0,4 mm	w_k=0,3 mm	w_k=0,2 mm
160	300	300	200
200	300	250	150
240	250	200	100
280	200	150	50
320	150	100	-
360	100	50	-

* **Note:** Under the relevant combinations of actions

(3) Beams with a total depth of 1000 mm or more, where the main reinforcement is concentrated in only a small proportion of the depth, should be provided with additional skin reinforcement to control cracking on the side faces of the beam. This reinforcement should be evenly distributed between the level of the tension steel and the neutral axis and should be located within the links. The area of the skin reinforcement should not be less than the amount obtained from 7.3.2 (2) taking k as 0,5 and σ_s as f_{yk}. The spacing and size of suitable bars may be obtained from Table 7.2 or 7.3 assuming pure tension and a steel stress of half the value assessed for the main tension reinforcement.

(4) It should be noted that there are particular risks of large cracks occurring at sections where there are sudden changes of stress, e.g.

- at changes of section
- near concentrated loads
- sections where bars are curtailed
- areas of high bond stress, particularly at the ends of laps

Care should be taken at such sections to minimise the stress changes wherever possible. However, the rules for crack control given above will normally ensure adequate control at these points provided that the rules for detailing reinforcement given in Sections 8 and 9 are observed.

Cracking due to tangential action effects may be assumed to be adequately controlled if the detailing rules given in 9.2.2, 9.2.3, 9.3.2 and 9.4.4.3 are observed.

7.3.4 Calculation of crack widths

(1) The characteristic crack, w_k, width may be obtained from the relation:

$$w_k = s_{r,max} (\varepsilon_{sm} - \varepsilon_{cm}) \qquad (7.8)$$

where

$s_{r,max}$ is the maximum crack spacing

ε_{sm} is the mean strain in the reinforcement under the relevant combination of loads, including the effect of imposed deformations and taking into account the effects of tension stiffening. Only the additional tensile strain beyond zero strain in the concrete is considered

ε_{cm} is the mean strain in concrete between cracks

(2) $\varepsilon_{sm} - \varepsilon_{cm}$ may be calculated from the expression:

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_s - k_t \dfrac{f_{ct,eff}}{\rho_{p,eff}}\left(1 + \alpha_e \rho_{p,eff}\right)}{E_s} \geq 0,6\frac{\sigma_s}{E_s} \qquad (7.9)$$

where:

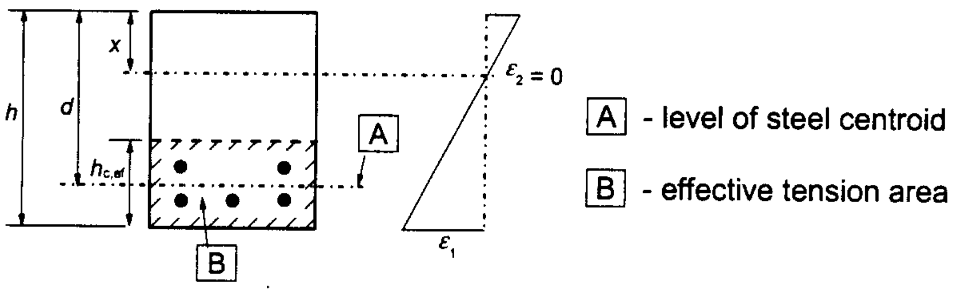
σ_s is the stress in the tension reinforcement assuming a cracked section. For pretensioned members, σ_s may be replaced by σ_s-σ_p where σ_s is the total stress and σ_p is the prestress.

α_e is theratio E_s/E_{cm}

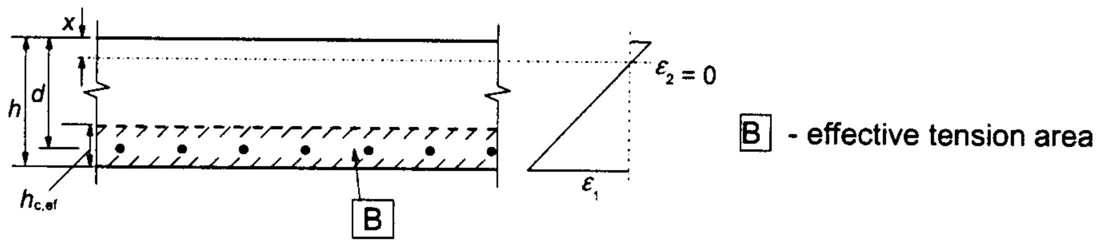
$$\rho_{p,eff} = \frac{A_s + \xi_1^2 A_p}{A_{c,eff}} \qquad (7.10)$$

$A_{c,eff}$ is the effective tension area. $A_{c,eff}$ is the area of concrete surrounding the tension reinforcement of depth, $h_{c,ef}$, where $h_{c,ef}$ is the lesser of 2,5(h-d), (h-x)/3 or h/2 (see Figure 7.1)

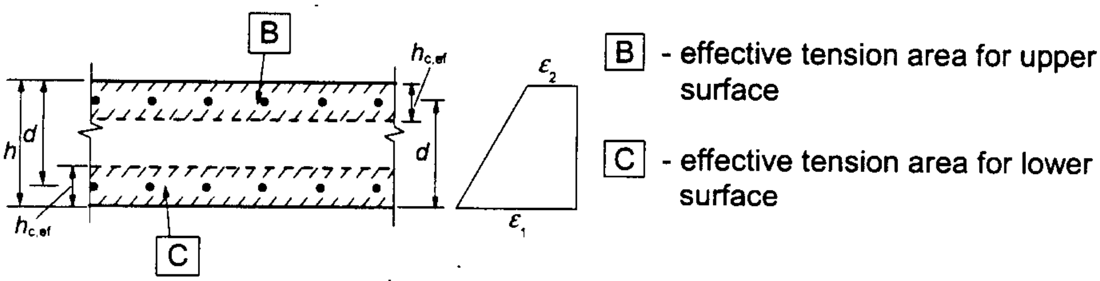
k_t is a factor dependent on the duration of the load
k_t = 0,6 for short term loading
k_t = 0,4 for long term loading



a) Beam



b) Slab



c) Member in tension

Figure 7.1: Effective tension area (typical cases)

(3) In situations where bonded reinforcement is fixed at reasonably close centres within the tension zone (spacing $\leq 5(c+\phi/2)$, the maximum final crack spacing may be calculated from Expression (7.11):

$$s_{r,max} = 3{,}4c + 0{,}425 k_1 k_2 \phi / \rho_{p,eff} \qquad (7.11)$$

where:

ϕ is the bar diameter. Where a mixture of bar diameters is used in a section, an equivalent diameter, ϕ_{eq}, should be used. For a section with n_1 bars of diameter ϕ_1 and n_2 bars of diameter ϕ_2, the following expression should be used

$$\phi_{eq} = \frac{n_1\phi_1^2 + n_2\phi_2^2}{n_1\phi_1 + n_2\phi_2} \qquad (7.12)$$

c is the cover to the reinforcement

k_1 is a coefficient which takes account of the bond properties of the bonded reinforcement:
= 0,8 for high bond bars
= 1,6 for bars with an effectively plain surface (e.g. prestressing tendons)

k_2 is a coefficient which takes account of the distribution of strain:
= 0,5 for bending
= 1,0 for pure tension
For cases of eccentric tension or for local areas, intermediate values of k_2 should be used which may be calculated from the relation:

$$k_2 = (\varepsilon_1 + \varepsilon_2)/2\varepsilon_1 \qquad (7.13)$$

Where ε_1 is the greater and ε_2 is the lesser tensile strain at the boundaries of the section considered, assessed on the basis of a cracked section

Where the spacing of the bonded reinforcement exceeds $5(c+\phi/2)$ (see Figure 7.2) or where there is no bonded reinforcement within the tension zone, an upper bound to the crack width may be found by assuming a maximum crack spacing:

$$s_{rmax} = 1.3\ (h - x) \qquad (7.14)$$

(4) Where the angle between the axes of principal stress and the direction of the reinforcement, for members reinforced in two orthogonal directions, is significant (>15°), then the crack spacing $s_{r,max}$ may be calculated from the following expression:

$$s_{r,max} = \frac{1}{\dfrac{\cos\theta}{s_{r,max,y}} + \dfrac{\sin\theta}{s_{r,max,z}}} \qquad (7.15)$$

where:

θ is the angle between the reinforcement in the y direction and the direction of the principal tensile stress

$s_{rmax,y}$ $s_{rmax,z}$ is the crack spacings calculated in the y and z directions respectively, according to 7.3.4 (3)

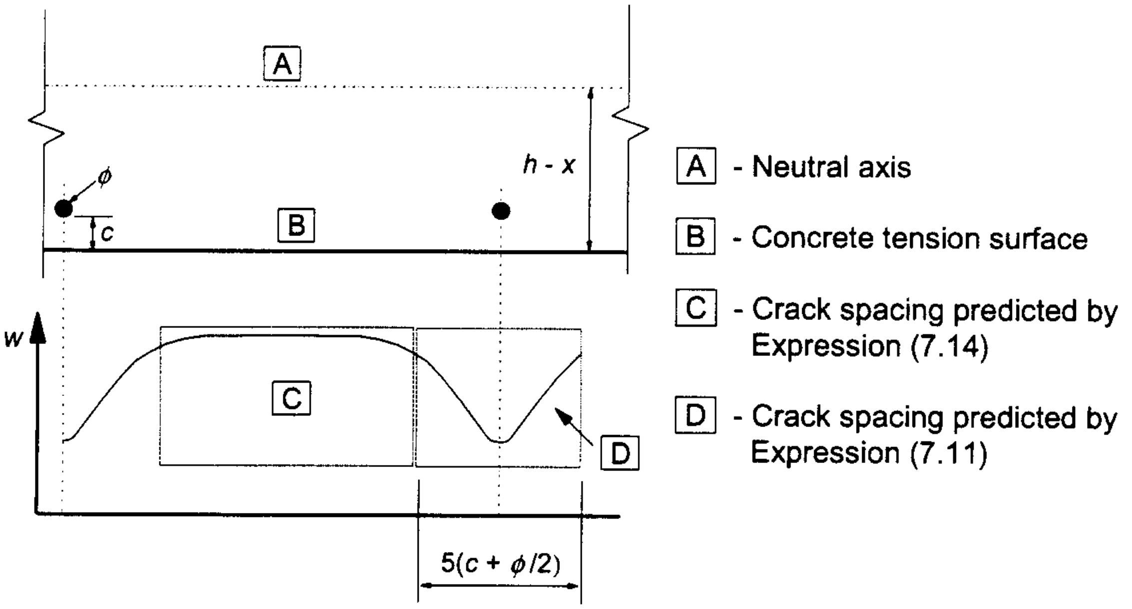


Figure 7.2: Crack width, w, at concrete surface relative to distance from bar

(5) For walls subjected to early thermal contraction where the horizontal steel area, A_s does not fulfil the requirements of 7.3.2 and where the bottom of the wall is restrained by a previously cast base, $s_{r,max}$ may be assumed to be equal to 1,3 times the height of the wall.

(6) Simplified methods of calculating crack width may be used provided they are based on the properties given in this Standard and substantiated by tests.

7.4 Deflections

7.4.1 General considerations

(1)P The deformation of a member or structure shall not be such that it adversely affects its proper functioning or appearance.

(2) Appropriate limiting values of deflection taking into account the nature of the structure, of the finishes, partitions and fixings and upon the function of the structure should be established.

(3) Deformations should not exceed those that can be accommodated by other connected elements such as partitions, glazing, cladding, services or finishes. In some cases limitation may be required to ensure the proper functioning of machinery or apparatus supported by the structure, or to avoid ponding on flat roofs.

Note: The limiting deflections given in (5) and (6) below are derived from ISO 4356 and should generally result in satisfactory performance of buildings such as dwellings, offices, public buildings or factories. Care should be taken to ensure that the limits are appropriate for the particular structure considered and that that there are no special requirements. Further information on deflections and limiting values may be obtained from ISO 4356.

(4) The appearance and general utility of the structure may be impaired when the calculated sag of a beam, slab or cantilever subjected to quasi-permanent loads exceeds span/250. The

sag is assessed relative to the supports. Pre-camber may be used to compensate for some or all of the deflection but any upward deflection incorporated in the formwork should not generally exceed span/250.

(5) Deflections that could damage adjacent parts of the structure should be limited. For the deflection after construction, span/500 is normally an appropriate limit for quasi-permanent loads. Other limits may be considered, depending on the sensitivity of adjacent parts.

(6) The limit state of deformation may be checked by either:
- by limiting the span/depth ratio, according to 7.4.2 or
- by comparing a calculated deflection, according to 7.4.3, with a limit value

The actual deformations may differ from the estimated values, particularly if the values of applied moments are close to the cracking moment. The differences will depend on the dispersion of the material properties, on the environmental conditions, on the load history, on the restraints at the supports, ground conditions, etc.

7.4.2 Cases where calculations may be omitted

(1)P Generally, it is not necessary to calculate the deflections explicitly as simple rules, for example limits to span/depth ratio may be formulated, which will be adequate for avoiding deflection problems in normal circumstances. More rigorous checks are necessary for members which lie outside such limits, or where deflection limits other than those implicit in simplified methods are appropriate.

(2) Provided that reinforced concrete beams or slabs in buildings are dimensioned so that they comply with the limits of span to depth ratio given in this clause, their deflections may be considered as not exceeding the limits set out in 7.4.1 (5) and (6). The limiting span/depth ratio may be estimated using Expressions (7.16.a) and (7.16.b) and multiplying this by correction factors to allow for the type of reinforcement used and other variables. No allowance has been made for any pre-camber in the derivation of these Expressions.

$$\frac{l}{d} = K\left[11 + 1{,}5\sqrt{f_{ck}}\,\frac{\rho_0}{\rho} + 3{,}2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] \quad \text{if } \rho \le \rho_0 \qquad (7.16.a)$$

$$\frac{l}{d} = K\left[11 + 1{,}5\sqrt{f_{ck}}\,\frac{\rho_0}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_0}}\right] \quad \text{if } \rho > \rho_0 \qquad (7.16.b)$$

where:
- l/d is the limit span/depth
- K is the factor to take into account the different structural systems
- ρ_0 is the reference reinforcement ratio = $\sqrt{f_{ck}}\, 10^{-3}$
- ρ is the required tension reinforcement ratio at mid-span to resist the moment due to the design loads (at support for cantilevers)
- ρ' is the required compression reinforcement ratio at mid-span to resist the moment due to design loads (at support for cantilevers)
- f_{ck} is in MPa units

Expressions (7.16.a) and (7.16.b) have been derived on the assumption that the steel stress, under the appropriate design service load at a cracked section at the mid-span of a beam or slab or at the support of a cantilever, is 310 MPa, (corresponding roughly to f_{yk} = 500 MPa). Where other stress levels are used, the values obtained using Expression (7.16) should be multiplied by $310/\sigma_s$. It will normally be conservative to assume that:

$$310 / \sigma_s = 500 /(f_{yk} A_{s,req} / A_{s,prov}) \tag{7.17}$$

where:

σ_s is the tensile steel stress at mid-span (at support for cantilevers) under the design service load

$A_{s,prov}$ is the area of steel provided at this section

$A_{s,req}$ is the area of steel required at this section for ultimate limit state

For flanged sections where the ratio of the flange breadth to the rib breadth exceeds 3, the values of l/d given by Expression (7.16) should be multiplied by 0,8.

For beams and slabs, other than flat slabs, with spans exceeding 7 m, which support partitions liable to be damaged by excessive deflections, the values of l/d given by Expression (7.16) should be multiplied by $7 / l_{eff}$ (l_{eff} in metres).
For flat slabs where the greater span exceeds 8,5 m, and which support partitions liable to be damaged by excessive deflections, the values of l/d given by Expression (7.16) should be multiplied by $8,5 / l_{eff}$ (l_{eff} in metres).

Note: Values of K for use in a Country may be found in its National Annex. Recommended values of K are given in Table 7.4N. Values obtained using Expression (7.16) for common cases (C30, σ_s = 310 MPa, different structural systems and reinforcement ratios ρ = 0,5 % and ρ = 1,5 %) are also given.

Table 7.4N: Basic ratios of span/effective depth for reinforced concrete members without axial compression

Structural System	K	Concrete highly stressed ρ = 1,5%	Concrete lightly stressed ρ = 0,5%
Simply supported beam, one- or two-way spanning simply supported slab	1,0	14	20
End span of continuous beam or one-way continuous slab or two-way spanning slab continuous over one long side	1,3	18	26
Interior span of beam or one-way or two-way spanning slab	1,5	20	30
Slab supported on columns without beams (flat slab) (based on longer span)	1,2	17	24
Cantilever	0,4	6	8

Note 1: The values given have been chosen to be generally conservative and calculation may frequently show that thinner members are possible.
Note 2: For 2-way spanning slabs, the check should be carried out on the basis of the shorter span. For flat slabs the longer span should be taken.
Note 3: The limits given for flat slabs correspond to a less severe limitation than a mid-span deflection of span/250 relative to the columns. Experience has shown this to be satisfactory.

The values given by Expression (7.16) and Table 7.4N have been derived from results of a parametric study made for a series of beams or slabs simply supported with rectangular cross section, using the general approach given in 7.4.3. Different values of concrete strength class and a 500 MPa characteristic yield strength were considered. For a given area of tension reinforcement the ultimate moment was calculated and the quasi-permanent load was assumed as 50% of the corresponding design load. The span/depth limits obtained satisfied the limiting deflection given in 7.4.1(5).

7.4.3 Checking deflections by calculation

(1)P Where a calculation is deemed necessary, the deformations shall be calculated under load conditions which are appropriate to the purpose of the check.

(2)P The calculation method adopted shall represent the true behaviour of the structure under relevant actions to an accuracy appropriate to the objectives of the calculation.

(3) Members which are not expected to be loaded above the level which would cause the tensile strength of the concrete to be exceeded anywhere within the member should be considered to be uncracked. Members which are expected to crack should behave in a manner intermediate between the uncracked and fully cracked conditions and, for members subjected mainly to flexure, an adequate prediction of behaviour is given by Expression (7.18):

$$\alpha = \zeta \alpha_{II} + (1 - \zeta)\alpha_{I} \qquad (7.18)$$

where

α is the deformation parameter considered which may be, for example, a strain, a curvature, or a rotation. (As a simplification, α may also be taken as a deflection - see (6) below)

α_I, α_{II} are the values of the parameter calculated for the uncracked and fully cracked conditions respectively

ζ is a distribution coefficient (allowing for tensioning stiffening at a section) given by Expression (7.19):

$$\zeta = 1 - \beta \left(\frac{\sigma_{sr}}{\sigma_s} \right)^2 \qquad (7.19)$$

ζ = 0 for uncracked sections

β is a coefficient taking account of the influence of the duration of the loading or of repeated loading on the average strain
= 1,0 for a single short-term loading
= 0,5 for sustained loads or many cycles of repeated loading

σ_s is the stress in the tension reinforcement calculated on the basis of a cracked section

σ_{sr} is the stress in the tension reinforcement calculated on the basis of a cracked section under the loading conditions causing first cracking

Note: σ_{sr}/σ_s may be replaced by M_{cr}/M for flexure or N_{cr}/N for pure tension, where M_{cr} is the cracking moment and N_{cr} is the cracking force.

(4) Deformations due to loading may be assessed using the tensile strength and the effective modulus of elasticity of the concrete.

Table 3.1 indicates the range of likely values for tensile strength. In general, the best estimate of the behaviour will be obtained if f_{ctm} is used. Where it can be shown that there are no axial tensile stresses (e.g. those caused by shrinkage or thermal effects) the flexural tensile strength, $f_{ctm,fl}$, (see 3.1.8) may be used.

(5) For loads with a duration causing creep, the total deformation including creep may be calculated by using an effective modulus of elasticity for concrete according to Expression (7.20):

$$E_{c,eff} = \frac{E_{cm}}{1+\varphi(\infty,t_0)} \tag{7.20}$$

where:

$\varphi(\infty,t_0)$ is the creep coefficient relevant for the load and time interval (see 3.1.3)

(6) Shrinkage curvatures may be assessed using Expression (7.21):

$$\frac{1}{r_{cs}} = \varepsilon_{cs}\alpha_e \frac{S}{I} \tag{7.21}$$

where:

$1/r_{cs}$ is the curvature due to shrinkage
ε_{cs} is the free shrinkage strain (see 3.1.4)
S is the first moment of area of the reinforcement about the centroid of the section
I second moment of area of the section
α_e is the effective modular ratio
$\alpha_e = E_s / E_{c,eff}$

S and I should be calculated for the uncracked condition and the fully cracked condition, the final curvature being assessed by use of Expression (7.17).

(7) The most rigorous method of assessing deflections using the method given in (3) above is to compute the curvatures at frequent sections along the member and then calculate the deflection by numerical integration. In most cases it will be acceptable to compute the deflection twice, assuming the whole member to be in the uncracked and fully cracked condition in turn, and then interpolate using Expression (7.17).

(8) Simplified methods of calculating deflection may be used provided they are based on the properties given in this Standard and substantiated by tests.

SECTION 8 DETAILING OF REINFORCEMENT - GENERAL

8.1 General

(1)P The rules given in this Section apply to ribbed reinforcement, mesh and prestressing tendons subjected predominantly to static loading. They are applicable for normal buildings and bridges. They may not be sufficient for:
- elements subjected to dynamic loading caused by seismic effects or machine vibration, impact loading and
- to elements incorporating specially painted, epoxy or zinc coated bars.

Additional rules are provided for large diameter bars.

(2)P The requirements concerning minimum concrete cover shall be satisfied (see 4.4.1.2).

(3) For lightweight aggregate concrete, supplementary rules are given in Section 11.

(4) Rules for structures subjected to fatigue loading are given in 6.8.

8.2 Spacing of bars

(1)P The spacing of bars shall be such that the concrete can be placed and compacted satisfactorily for the development of adequate bond.

(2) The clear distance (horizontal and vertical) between individual parallel bars or horizontal layers of parallel bars should be not less than the maximum of bar diameter, (d_g + 5 mm) or 20 mm where d_g is the maximum size of aggregate.

(3) Where bars are positioned in separate horizontal layers, the bars in each layer should be located vertically above each other. There should be sufficient space between the resulting columns of bars to allow access for vibrators and good compaction of the concrete.

(4) Lapped bars may be allowed to touch one another within the lap length. See 8.7 for more details.

8.3 Permissible mandrel diameters for bent bars

(1)P The minimum diameter to which a bar is bent shall be such as to avoid bending cracks in the bar, and to avoid failure of the concrete inside the bend of the bar.

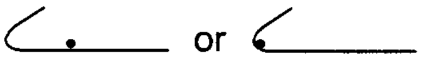
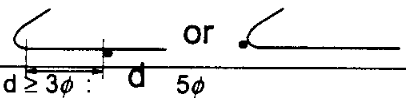
Note: The values of the mandrel diameter for use in a Country may be found in its National Annex. The recommend minimum values are given in Table 8.1N. These values may be used without causing concrete failure if one of the following conditions is fulfilled (ϕ is the diameter of bent bar):
- the anchorage of the bar does not require a length more than 5ϕ past the end of the bend
- there is a cross bar placed inside the bend with a diameter at least that of the bent bar.

Table 8.1N: Minimum mandrel diameter to avoid damage to reinforcement

a) for bars and wire

Bar diameter	Minimum mandrel diameter for bends, hooks and loops (see Figure 8.1)
$\phi \leq 16$ mm	4ϕ
$\phi > 16$ mm	7ϕ

b) for welded bent reinforcement and mesh bent after welding

Minimum mandrel diameter	
	
5ϕ	$d \geq 3\phi$: 5ϕ $d < 3\phi$ or welding within the curved zone: 20ϕ
Note: The mandrel size for welding within the curved zone may be reduced to 5ϕ where the welding is carried out in accordance with prEN ISO 17660 Annex B	

(2) The mandrel diameter need not be checked to avoid concrete failure if the following conditions exist:

- the anchorage of the bar does not require a length more than 5ϕ past the end of the bend;
- the bar is not positioned at the edge (plane of bend close to concrete face) and there is a cross bar diameter $\geq \phi$ inside the bend.

Otherwise the mandrel diameter, ϕ_m, should be increased in accordance with Expression (8.1)

$$\phi_m \geq F_{bt}((1/a_b)+1/(2\phi)) / f_{cd} \tag{8.1}$$

where:

F_{bt} is the tensile force from ultimate loads in a bar or group of bars in contact at the start of a bend

a_b for a given bar (or group of bars in contact) is the half the centre-to-centre distance between bars (or groups of bars) perpendicular to the plane of the bend. For a bar or group of bars adjacent to the face of the member, a_b should be taken as the cover plus $\phi/2$

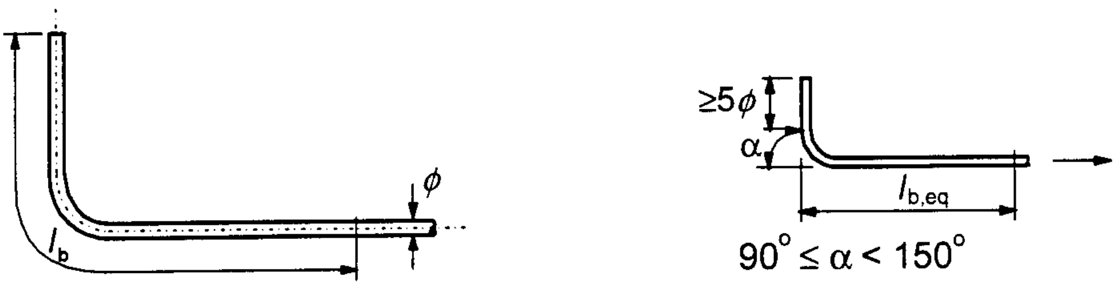
The value of f_{cd} should not be taken greater than that for concrete class C55/67.

8.4 Anchorage of longitudinal reinforcement

8.4.1 General

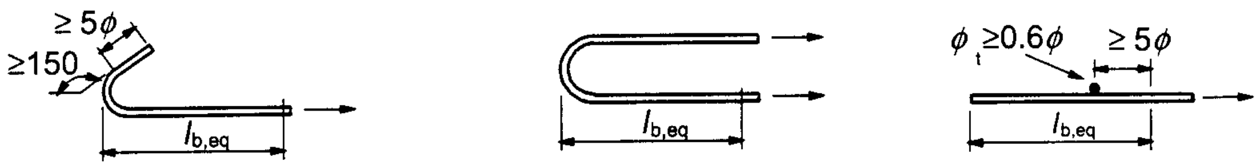
(1)P Reinforcing bars, wires or welded mesh fabrics shall be so anchored that the bond forces are safely transmitted to the concrete avoiding longitudinal cracking or spalling. Transverse reinforcement shall be provided if necessary.

(2) Methods of anchorage are shown in Figure 8.1.



a) **Basic anchorage length, l_b, for any shape measured along the centreline**

b) **Equivalent anchorage length for standard bend**



c) **Equivalent anchorage length for standard hook**

d) **Equivalent anchorage length for standard loop**

e) **Welded transverse bar**

Figure 8.1: Methods of anchorage other than by a straight bar

(3) Bends and hooks do not contribute to compression anchorages.

(4) Concrete failure inside bends should be prevented by complying with 8.3 (3).

(5) Where mechanical devices are used the test requirements should be in accordance with the relevant product standard or a European Technical Approval.

(6) For the transmission of prestressing forces to the concrete, see 8.10.

8.4.2 Ultimate bond stress

(1)P The ultimate bond stress shall be such that there is an adequate safety margin against bond failure.

(2) The design value of the ultimate bond stress, f_{bd}, for ribbed bars may be taken as:

$$f_{bd} = 2{,}25\, \eta_1\, \eta_2\, f_{ctd} \qquad (8.2)$$

where:

f_{ctd} is the design value of concrete tensile strength according to 3.1.6 (2)P. Due to the increasing brittleness of higher strength concrete, $f_{ctk,0,05}$ should be limited here to the value for C60, unless it can be verified that the average bond strength increases above this limit

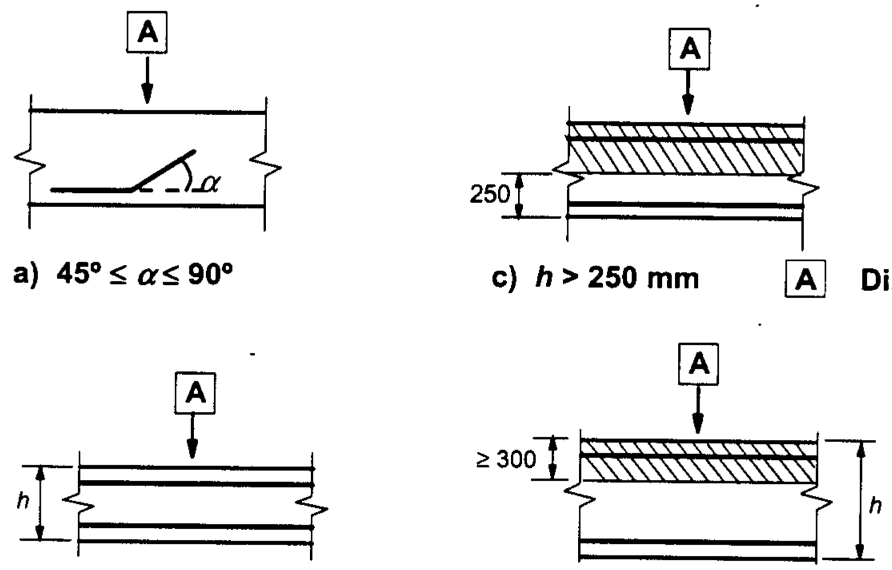
η_1 is a coefficient related to the quality of the bond condition and the position of the bar during concreting (see Figure 8.2):
η_1 = 1,0 when 'good' conditions are obtained and
η_1 = 0,7 for all other cases and for bars in structural elements built with slip-forms,

unless it can be shown that 'good' bond conditions exist

η_2 is related to the bar diameter:

η_2 = 1,0 for $\phi \leq 32$ mm

η_2 = (132 - ϕ)/100 for $\phi > 32$ mm



a) $45° \leq \alpha \leq 90°$

b) $h \leq 250$ mm

c) $h > 250$ mm

d) $h > 600$ mm

a) & b) 'good' bond conditions for all bars

c) & d) unhatched zone – 'good' bond conditions
hatched zone – 'poor' bond conditions

Figure 8.2: Description of bond conditions

8.4.3 Basic anchorage length

(1)P The calculation of the required anchorage length shall take into consideration the type of steel and bond properties of the bars.

(2) The basic required anchorage length, $l_{b,rqd}$, for anchoring the force $A_s.f_{yd}$ in a bar assuming constant bond stress equal to f_{bd} follows from:

$$l_{b,rqd} = (\phi / 4)\,(\sigma_{sd} / f_{bd}) \tag{8.3}$$

Where σ_{sd} is the design stress of the bar at the position from where the anchorage is measured from at the ultimate limit state

Values for f_{bd} are given in 8.4.2.

(3) For bent bars the anchorage length, l_b , should be measured along the centre-line of the bar (see Figure 8.1a).

(4) Where pairs of wires/bars form welded fabrics the diameter, ϕ, in Expression (8.3) should be replaced by the equivalent diameter $\phi_n = \phi\sqrt{2}$.

8.4.4 Design anchorage length

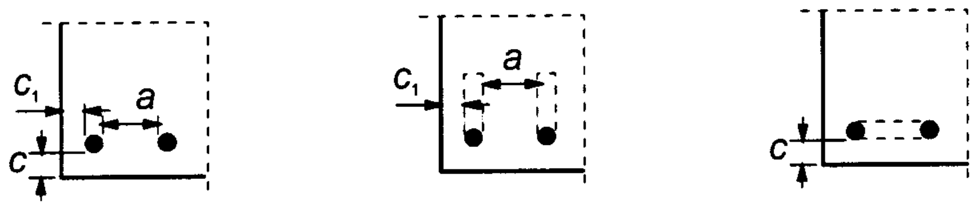
(1) The design anchorage length, $l_{bd,}$:

$$l_{bd} = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_4\ \alpha_5\ l_{b,rqd} \geq l_{b,min} \tag{8.4}$$

where α_1, α_2, α_3, α_4 and α_5 are coefficients given in Table 8.2:

α_1 is for the effect of the form of the bars assuming adequate cover (see Figure 8.1).

α_2 is for the effect of concrete cover (see Figure 8.3)



a) Straight bars
c_d = min (a/2, c_1, c)

b) Bent or hooked bars
c_d = min (a/2, c_1)

c) Looped bars
$c_d = c$

Figure 8.3: Values of c_d for beams and slabs

α_3 is for the effect of confinement by transverse reinforcement

α_4 is for the influence of one or more welded transverse bars ($\phi_t > 0{,}6\phi$) along the design anchorage length l_{bd} (see also 8.6)

α_5 is for the effect of the pressure transverse to the plane of splitting along the design anchorage length

The product $(\alpha_2\alpha_3\alpha_5) \geq 0{,}7$

$l_{b,rqd}$ is taken from Expression (8.3)
$l_{b,min}$ is the minimum anchorage length if no other limitation is applied:
- for anchorages in tension: $l_{b,min} > \max\{0{,}3 l_{b,rqd};\ 10\phi;\ 100\ \text{mm}\}$ (8.6)
- for anchorages in compression: $l_{b,min} > \max\{0{,}6 l_{b,rqd};\ 10\phi;\ 100\ \text{mm}\}$ (8.7)

(2) As a simplified alternative to 8.4.4 (1) the tension anchorage of certain shapes shown in Figure 8.1 may be provided as an equivalent anchorage length, $l_{b,eq}$. $l_{b,eq}$ is defined in this figure and may be taken may be taken as:
- $\alpha_1\, l_{b,rqd}$ for shapes shown in Figure 8.1b to 8.1d (see 8.4.4 for values of α_1)
- $\alpha_4\, l_{b,rqd}$ for shapes shown in Figure 8.1e (see 8.4.4 for values of α_4).

where
α_1 and α_4 are defined in 8.4.4
$l_{b,rqd}$ is calculated from Expression (8.3)

Table 8.2: Values of α_1, α_2, α_3, α_4 and α_5 coefficients

Influencing factor	Type of anchorage	Reinforcement bar	
		In tension	In compression
Shape of bars	Straight	$\alpha_1 = 1,0$	$\alpha_1 = 1,0$
	Other than straight (see Figure 8.1 (b), (c) and (d) & Figure 8.3)	$\alpha_1 = 0,7$ if $c_d > 3\phi$ otherwise $\alpha_1 = 1,0$ (see Figure 8.3 for values of c_d)	$\alpha_1 = 1,0$
Concrete cover	Straight	$\alpha_2 = 1 - 0,15\,(c_d - \phi)/\phi$ $\geq 0,7$ $\leq 1,0$	$\alpha_2 = 1,0$
	Other than straight (see Figure 8.1 (b), (c) and (d))	$\alpha_2 = 1 - 0,15\,(c_d - 3\phi)/\phi$ $\geq 0,7$ $\leq 1,0$ (see Figure 8.3 for values of c_d)	$\alpha_2 = 1,0$
Confinement by transverse reinforcement not welded to main reinforcement	All types	$\alpha_3 = 1 - K\lambda$ $\geq 0,7$ $\leq 1,0$	$\alpha_3 = 1,0$
Confinement by welded transverse reinforcement*	All types, position and size as specified in Figure 8.1 (e)	$\alpha_4 = 0,7$	$\alpha_4 = 0,7$
Confinement by transverse pressure	All types	$\alpha_5 = 1 - 0,04p$ $\geq 0,7$ $\leq 1,0$	-

where:

λ $= (\Sigma A_{st} - \Sigma A_{st,min})/ A_s$

ΣA_{st} cross-sectional area of the transverse reinforcement along the design anchorage length l_{bd}

$\Sigma A_{st,min}$ cross-sectional area of the minimum transverse reinforcement $= 0,25\ A_s$ for beams and 0 for slabs

A_s area of a single anchored bar with maximum bar diameter

K values shown in Figure 8.4

p transverse pressure [MPa] at ultimate limit state along l_{bd}

* See also 8.6: For direct supports l_{bd} may be taken less than $l_{b,min}$ provided that there is at least one transverse wire welded within the support. This should be at least 15 mm from the face of the support.

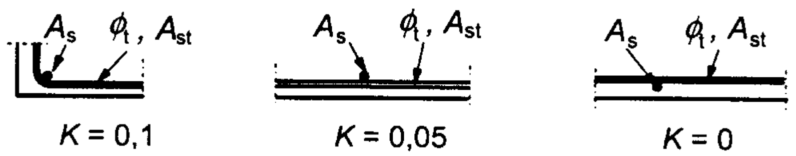


Figure 8.4: Values of K for beams and slabs

8.5 Anchorage of links and shear reinforcement

(1) The anchorage of links and shear reinforcement should normally be effected by means of bends and hooks, or by welded transverse reinforcement. A bar should be provided inside a hook or bend.

(2) The anchorage should comply with Figure 8.5. Welding should be carried out in accordance with EN ISO 17660 and have a welding capacity in accordance with 8.6 (2).

Note: For definition of the bend angles see Figure 8.1.

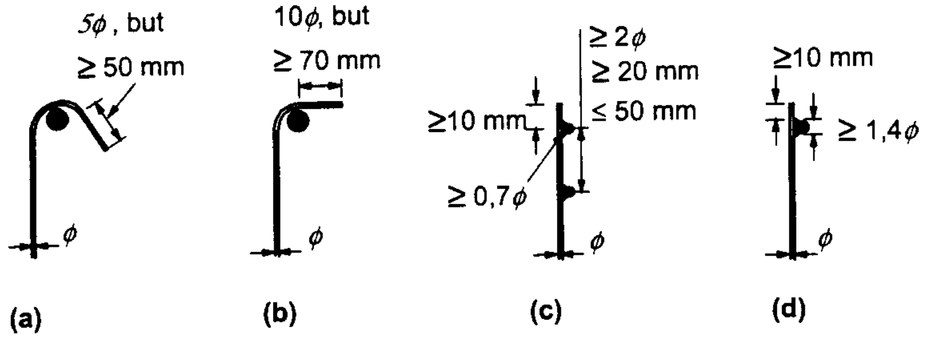


Figure 8.5: Anchorage of links

8.6 Anchorage by welded bars

(1) Additional anchorage to that of 8.4 and 8.5 may be obtained by transverse welded bars (see Figure 8.6) bearing on the concrete. The quality of the welded joints should be shown to be adequate.

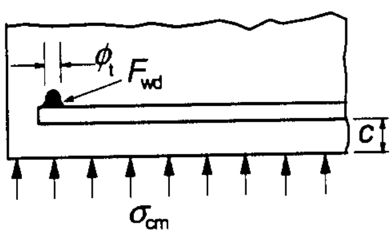


Figure 8.6: Welded transverse bar as anchoring device

(2) The anchorage capacity of one welded transverse bar (diameter 14 mm- 32 mm), welded on the inside of the main bar, is F_{btd}. σ_{sd} in Expression (8.3) may then be reduced by F_{btd}/A_s, where A_s is the area of the bar.

Note: The value of F_{btd} for use in a Country may be found in its National Annex. The recommended value is determined from:

$$F_{btd} = l_{td}\ \phi_t\ \sigma_{td} \text{ but not greater than } F_{wd} \quad (8.8N)$$

where:

F_{wd} is the design shear strength of weld (specified as a factor times $A_s\ f_{yd}$; say 0.5 $A_s\ f_{yd}$ where A_s is the cross-section of the anchored bar and f_{yd} is its design yield strength)

l_{td} is the design length of transverse bar: $l_{td} = 1{,}16\ \phi_t\ (f_{yd}/\sigma_{td})^{0,5} \leq l_t$

l_t is the length of transverse bar, but not more than the spacing of bars to be anchored
ϕ_t is the diameter of transverse bar
σ_{td} is the concrete stress; $\sigma_{td} = (f_{ctd} + \sigma_{cm})/y \leq 3\, f_{cd}$
σ_{cm} is the compression in the concrete perpendicular to both bars (mean value, positive for compression)
y is a function: $y = 0{,}015 + 0{,}14\, e^{(-0{,}18x)}$
x is a function accounting for the geometry: $x = 2\,(c/\phi_t) + 1$
c is the concrete cover perpendicular to both bars

(3) If two bars of the same size are welded on opposite sides of the bar to be anchored, the capacity given by Expression (8.8) may be doubled provided that the cover to the outer bar is in accordance with Section 4.

(4) If two bars are welded to the same side with a minimum spacing of 3ϕ, the capacity should be multiplied by a factor of 1,41.

(5) For steel grade B500 and nominal bar diameters of 12 mm and less, the anchorage capacity of a welded cross bar is mainly dependent on the design strength of the welded joint. The anchorage capacity of a welded cross bar for sizes of maximum 12 mm may be calculated as follows:

$$F_{btd} = F_{wd} \leq 16\, A_s\, f_{cd}\, \phi_t \,/\, \phi_l \qquad (8.9)$$

where:
F_{wd} design shear strength of weld (see Expression (8.8))
ϕ_t nominal diameter of transverse bar: $\phi_t \leq 12$ mm
ϕ_l nominal diameter of bar to anchor: $\phi_l \leq 12$ mm

If two welded cross bars with a minimum spacing of ϕ_t are used, the anchorage length given by Expression (8.4) should be multiplied by a factor of 1,41.

8.7 Laps and mechanical couplers

8.7.1 General

(1)P Forces are transmitted from one bar to another by:
- lapping of bars, with or without bends or hooks;
- welding;
- mechanical devices assuring load transfer in tension-compression or in compression only.

8.7.2 Laps

(1)P The detailing of laps between bars shall be such that:
- the transmission of the forces from one bar to the next is assured;
- spalling of the concrete in the neighbourhood of the joints does not occur;
- large cracks which affect the performance of the structure do not occur.

(2) Laps:
- between bars should normally be staggered and not located in areas of high stress. Exceptions are given in (4) below;
- at any one section should normally be arranged symmetrically.

(3) The arrangement of lapped bars should comply with Figure 8.7:
- the clear transverse distance between two lapped bars should not be greater than 4ϕ or 50 mm, otherwise the lap length should be increased by a length equal to the clear space where it exceeds 4ϕ or 50 mm;
- the longitudinal distance between two adjacent laps should not be less than 0,3 times the lap length, l_0;
- In case of adjacent laps, the clear distance between adjacent bars should not be less than 2ϕ or 20 mm.

(4) When the provisions comply with (3) above, the permissible percentage of lapped bars in tension may be 100% where the bars are all in one layer. Where the bars are in several layers the percentage should be reduced to 50%.

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

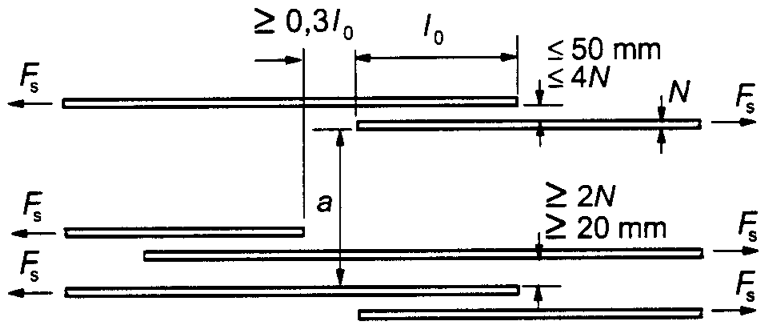


Figure 8.7: Adjacent laps

8.7.3 Lap length

(1) The design lap length is:

$$l_0 = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_5\ \alpha_6\ l_{b,rqd}\ A_{s,req} / A_{s,prov} \geq l_{0,min} \tag{8.10}$$

where:

$l_{b,rqd}$ is calculated from Expression (8.3)

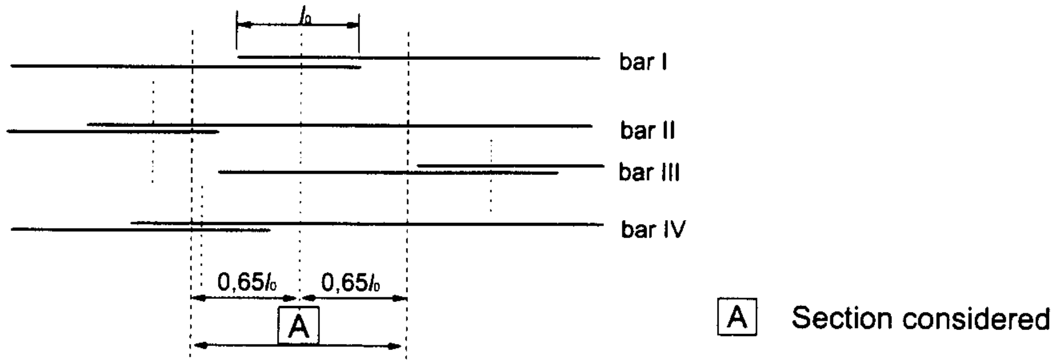
$$l_{0,min} > \max\{0,3\ \alpha_6\ l_{b,rqd};\ 15\phi,\ 200\text{ mm}\} \tag{8.11}$$

Values of α_1, α_2, α_3 *and* α_5 may be taken from Table 8.2; however, for the calculation of α_3, $\Sigma A_{st,min}$ should be taken as $1,0A_s$, with A_s = area of one lapped bar.

α_6 = $(\rho_1/25)^{0,5}$ but not exceeding 1,5, where ρ_1 is the percentage of reinforcement lapped within 0,65 l_0 from the centre of the lap length considered (see Figure 8.8). Values of α_6 are given in Table 8.3.

Table 8.3: Values of the coefficient α_6

Percentage of lapped bars relative to the total cross-section area	< 25%	33%	50%	>50%
α_6	1	1,15	1,4	1,5
Note: Intermediate values may be determined by interpolation.				



Example: Bars II and III are outside the section being considered: % = 50 and α_6 =1,4

Figure 8.8: Percentage of lapped bars in one section

8.7.4 Transverse reinforcement in the lap zone

8.7.4.1 Transverse reinforcement for bars in tension

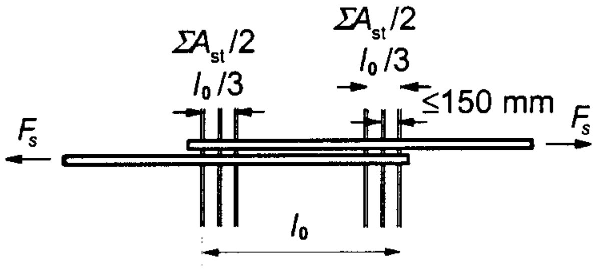
(1) Transverse reinforcement is required in the lap zone to resist transverse tension forces.

(2) Where the diameter, ϕ, of the lapped bars is less than 20 mm, or the percentage of lapped bars in any one section is less than 25%, then any transverse reinforcement or links necessary for other reasons may be assumed sufficient for the transverse tensile forces without further justification.

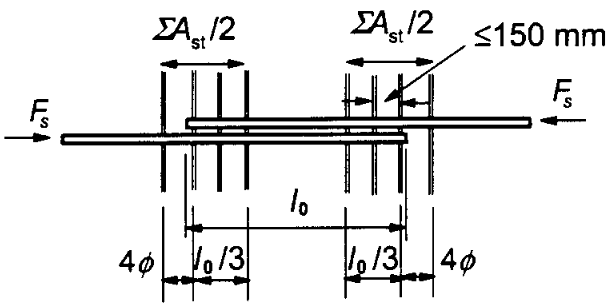
(3) Where the diameter, ϕ, of the lapped bars is greater than or equal to 20 mm, the transverse reinforcement should have a total area, A_{st} (sum of all legs parallel to the layer of the spliced reinforcement) of not less than the area A_s of one spliced bar ($\Sigma A_{st} \geq 1,0A_s$). It should be placed perpendicular to the direction of the lapped reinforcement and between that and the surface of the concrete.

If more than 50% of the reinforcement is lapped at one point and the distance, a, between adjacent laps at a section is $\leq 10\phi$ (see Figure 8.7) transverse bars should be formed by links or U bars anchored into the body of the section.

(4) The transverse reinforcement provided for (3) above should be positioned at the outer sections of the lap as shown in Figure 8.9(a).



a) bars in tension



b) bars in compression

Figure 8.9: Transverse reinforcement for lapped splices

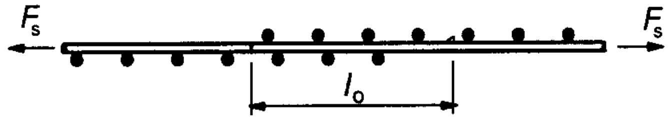
8.7.4.2 Transverse reinforcement for bars permanently in compression

(1) In addition to the rules for bars in tension one bar of the transverse reinforcement should be placed outside each end of the lap length and within 4ϕ of the ends of the lap length (Figure 8.9b).

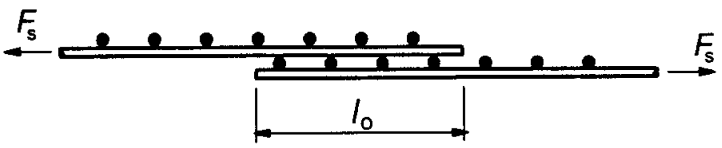
8.7.5 Laps for welded mesh fabrics made of ribbed wires

8.7.5.1 Laps of the main reinforcement

(1) The splices can be made either by intermeshing or by layering of the fabrics (Figure 8.10).



a) intermeshed fabric (longitudinal section)



b) layered fabric (longitudinal section)

Figure 8.10: Lapping of welded fabric

(2) Where fatigue loads occur, intermeshing should be adopted

(3) For intermeshed fabric, the lapping arrangements for the main longitudinal bars should conform with 8.7.2. Any favourable effects of the transverse bars should be ignored: thus taking $\alpha_3 = 1{,}0$.

(4) For layered fabric, the laps of the main reinforcement should generally be situated in zones where the calculated stress in the reinforcement at ultimate limit state is not more than 80% of the design strength.

(5) Where condition (4) above is not fulfilled, the effective depth of the steel for the calculation of bending resistance in accordance with 6.1 should apply to the layer furthest from the tension face. In addition, when carrying out a crack-verification next to the end of the lap, the steel stress used in Tables 7.2 and 7.3 should be increased by 25% due to the discontinuity at the ends of the laps,.

(6) The percentage of the main reinforcement, which may be lapped in any one section, should comply with the following:

For intermeshed fabric, the values given in Table 8.3 are applicable.

For layered fabric the permissible percentage of the main reinforcement that may be spliced by lapping in any section, depends on the specific cross-section area of the welded fabric provided $(A_s/s)_{prov}$, where s is the spacing of the wires:

- 100% if $(A_s/s)_{prov} \leq 1200$ mm^2/m
- 60% if $(A_s/s)_{prov} > 1200$ mm^2/m.

The joints of the multiple layers should be staggered by at least $1{,}3l_0$ (l_0 is determined from 8.7.3).

(7) Additional transverse reinforcement is not necessary in the lapping zone.

8.7.5.2 Laps of secondary or distribution reinforcement

(1) All secondary reinforcement may be lapped at the same location.

The minimum values of the lap length l_0 are given in Table 8.4; at least two transverse bars should be within the lap length (one mesh).

Table 8.4: Required lap lengths for secondary layered fabric

Diameter of wires (mm)	Lap lengths
$\phi \leq 6$	$\geq$ 150 mm; at least 1 wire pitch within the lap length
$6 < \phi \leq 8{,}5$	$\geq$ 250 mm; at least 2 wire pitches
$8{,}5 < \phi \leq 12$	$\geq$ 350 mm; at least 2 wire pitches

8.8 Additional rules for large diameter bars

(1) For bars with a diameter larger than ϕ_{large} the following rules supplement those given in 8.4 and 8.7.

Note: The value of ϕ_{large} for use in a Country may be found in its National Annex. The recommended value is 32 mm

(2) When such large diameter bars are used, crack control may be achieved either by using surface reinforcement (see 9.2.4) or by calculation (see 7.3.4).

(3) Splitting forces are higher and dowel action is greater with the use of large diameter bars. Such bars should be anchored with mechanical devices. If anchored as straight bars, links should be provided as confining reinforcement.

(4) Generally large diameter bars should not be lapped. Exceptions include sections with a minimum dimension 1,0 m or where the stress is not greater than 80% of the design ultimate strength.

(5) Transverse reinforcement, additional to that for shear, should be provided in the anchorage zones where transverse compression is not present.

(6) For straight anchorage lengths (see Figure 8.11 for the notation used) the additional reinforcement referred to in (5) above should not be less than the following:

- in the direction parallel to the tension face:

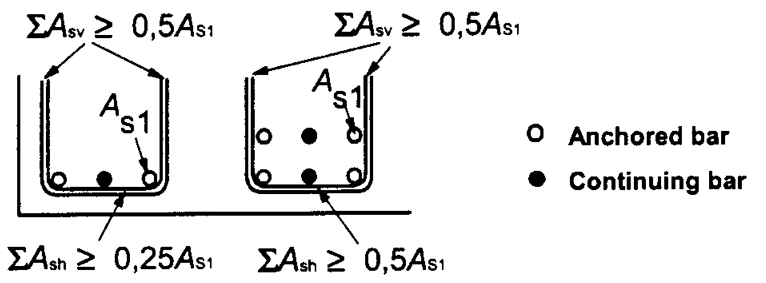
$$A_{sh} = 0,25\, A_s\, n_1 \tag{8.12}$$

- in the direction perpendicular to the tension face:

$$A_{sv} = 0,25\, A_s\, n_2 \tag{8.13}$$

where:
A_s is the cross sectional area of an anchored bar,
n_1 is the number of layers with bars anchored at the same point in the member
n_2 is the number of bars anchored in each layer.

(7) The additional transverse reinforcement should be uniformly distributed in the anchorage zone and the spacing of bars should not exceed 5 times the diameter of the longitudinal reinforcement.



Example: In the left hand case n_1 = 1, n_2 = 2 and in the right hand case n_1 = 2, n_2 = 2

Figure 8.11: Additional reinforcement in an anchorage for large diameter bars where there is no transverse compression.

(8) For surface reinforcement, 9.2.4 applies, but the area of surface reinforcement should not be less than 0,01 $A_{ct,ext}$ in the direction perpendicular to large diameter bars, and 0,02 $A_{ct,ext}$ parallel to those bars.

8.9 Bundled bars

8.9.1 General

(1) Unless otherwise stated, the rules for individual bars also apply for bundles of bars. In a bundle, all the bars should be of the same characteristics (type and grade). Bars of different sizes may be bundled provided that the ratio of diameters does not exceed 1,7.

(2) In design, the bundle is replaced by a notional bar having the same sectional area and the same centre of gravity as the bundle. The equivalent diameter, ϕ_n of this notional bar is such that:

$$\phi_n = \phi \sqrt{n_b} \leq 55 \text{ mm} \tag{8.14}$$

where
n_b is the number of bars in the bundle, which is limited to:
$n_b \leq 4$ for vertical bars in compression and for bars in a lapped joint,
$n_b \leq 3$ for all other cases.

(3) For a bundle, the rules given in 8.2 for spacing of bars apply. The equivalent diameter, ϕ_n, should be used but the clear distance between bundles should be measured from the actual external contour of the bundle of bars. The concrete cover should be measured from the actual external contour of the bundles and should not be less than ϕ_n.

(4) Where two touching bars are positioned one above the other, and where the bond conditions are good, such bars need not be treated as a bundle.

8.9.2 Anchorage of bundles of bars

(1) Bundles of bars in tension may be curtailed over end and intermediate supports. Bundles with an equivalent diameter < 32 mm may be curtailed near a support without the need for staggering bars. Bundles with an equivalent diameter ≥ 32 mm which are anchored near a support should be staggered in the longitudinal direction as shown in Figure 8.12.

(2) Where individual bars are anchored with a staggered distance greater than 1,3 $l_{b,rqd}$ (where $l_{b,rqd}$ is based on the bar diameter), the diameter of the bar may be used in assessing l_{bd} (see Figure 8.12). Otherwise the equivalent diameter of the bundle, ϕ_n, should be used.

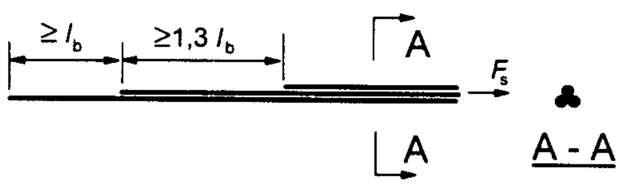


Figure 8.12: Anchorage of widely staggered bars in a bundle

(3) For compression anchorages bundled bars need not be staggered. For bundles with an equivalent diameter ≥ 32 mm, at least four links having a diameter ≥ 12 mm should be provided at the ends of the bundle. A further link should be provided just beyond the end of the curtailed bar.

8.9.3 Lapping bundles of bars

(1) The lap length should be calculated in accordance with 8.7.3 using ϕ_n (from 8.9.1 (2)) as the equivalent diameter of bar.

(2) For bundles which consist of two bars with an equivalent diameter < 32 mm the bars may be lapped without staggering individual bars. In this case the equivalent bar size should be used to calculate l_0.

(3) For bundles which consist of two bars with an equivalent diameter ≥ 32 mm or of three bars, individual bars should be staggered in the longitudinal direction by at least $1{,}3l_0$ as shown in Figure 8.13. For this case the diameter of a single bar may be used to calculate l_0. Care should be taken to ensure that there are not more than four bars in any lap cross section.

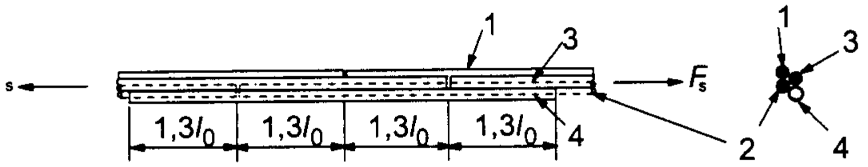


Figure 8.13: Lap joint in tension including a fourth bar

8.10 Prestressing tendons

8.10.1 Arrangement of prestressing tendons and ducts

8.10.1.1 General

(1)P The spacing of ducts or of pre-tensioned tendons shall be such as to ensure that placing and compacting of the concrete can be carried out satisfactorily and that sufficient bond can be attained between the concrete and the tendons.

8.10.1.2 Pre-tensioned tendons

(1) The minimum clear horizontal and vertical spacing of individual pre-tensioned tendons should be in accordance with that shown in Figure 8.14. Other layouts may be used provided that test results show satisfactory ultimate behaviour with respect to:
- the concrete in compression at the anchorage
- the spalling of concrete
- the anchorage of pre-tensioned tendons
- the placing of the concrete between the tendons.

Consideration should also be given to durability and the danger of corrosion of the tendon at the end of elements.

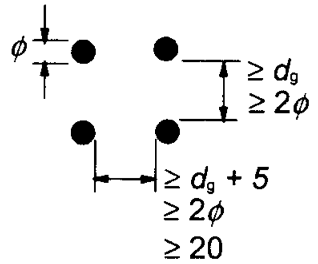


Figure 8.14: Minimum clear spacing between pre-tensioned tendons.

(2) Bundling of tendons should not occur in the anchorage zones, unless placing and compacting of the concrete can be carried out satisfactorily and sufficient bond can be attained between the concrete and the tendons.

8.10.1.3 Post-tension ducts

(1)P The ducts for post-tensioned tendons shall be located and constructed so that:
- the concrete can be safely placed without damaging the ducts;
- the concrete can resist the forces from the ducts in the curved parts during and after stressing;
- no grout will leak into other ducts during grouting process.

(2) Bundled ducts for post-tensioned members, should not normally be bundled except in the case of a pair of ducts placed vertically one above the other.

(3) The minimum clear spacing between ducts should be in accordance with that shown in Figure 8.15.

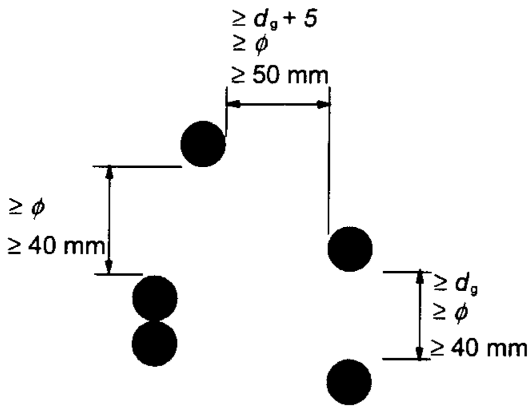


Figure 8.15 Minimum clear spacing between ducts

8.10.2 Anchorage of pre-tensioned tendons

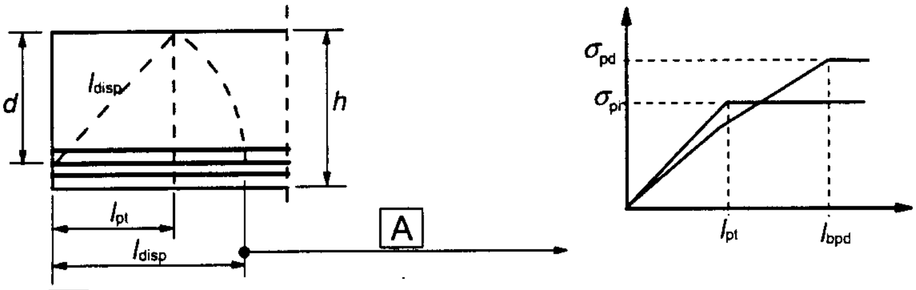
8.10.2.1 General

(1) In anchorage regions for pre-tensioned tendons, the following length parameters should be considered, see Figure 8.16:

a) Transmission length, l_{pt}, over which the prestressing force (P_0) is fully transmitted to the concrete; see 8.10.2.2 (2),

b) Dispersion length, l_{disp} over which the concrete stresses gradually disperse to a linear distribution across the concrete section; see 8.10.2.2 (4),

c) Anchorage length, l_{bpd}, over which the tendon force F_{pd} in the ultimate limit state is fully anchored in the concrete; see 8.10.2.3 (4) and (5).



A - **Linear stress distribution in member cross-section**

Figure 8.16: Transfer of prestress in pretensioned elements; length parameters

8.10.2.2 Transfer of prestress

(1) At release of tendons, the prestress may be assumed to be transferred to the concrete by a constant bond stress f_{bpt}, where:

$$f_{bpt} = \eta_{p1}\,\eta_1\,f_{ctd}(t) \tag{8.15}$$

where:

η_{p1} is a coefficient that takes into account the type of tendon and the bond situation at release
η_{p1} = 2,7 for indented wires
η_{p1} = 3,2 for 7-wire strands

η_1 = 1,0 for good bond conditions (see 8.4.2)
= 0,7 otherwise, unless a higher value can be justified with regard to special circumstances in execution

$f_{ctd}(t)$ is the design tensile value of strength at time of release; $f_{ctd}(t) = \alpha_{ct} \cdot 0{,}7 \cdot f_{ctm}(t) / \gamma_c$ (see also 3.1.2 (8) and 3.1.6 (2)P)

Note: Values of η_{p1} for types of tendons other than those given above may be used subject to the relevant product standard or a European Technical Approval

(2) The basic value of the transmission length, l_{pt}, is given by:

$$l_{pt} = \alpha_1 \alpha_2 \phi \sigma_{pm0} / f_{bpt} \tag{8.16}$$

where:

α_1 = 1,0 for gradual release
= 1,25 for sudden release

α_2 = 0,25 for tendons with circular cross section
= 0,19 for 7-wire strands

ϕ is the nominal diameter of tendon

σ_{pm0} is the tendon stress just after release

(3) The design value of the transmission length should be taken as the less favourable of two values, depending on the design situation:

$$l_{pt1} = 0{,}8\ l_{pt} \qquad (8.17)$$

or

$$l_{pt2} = 1{,}2\ l_{pt} \qquad (8.18)$$

Note: Normally the lower value is used for verifications of local stresses at release, the higher value for ultimate limit states (shear, anchorage etc.).

(4) Concrete stresses may be assumed to have a linear distribution outside the dispersion length, see Figure 8.17:

$$l_{disp} = \sqrt{l_{pt}^2 + d^2} \qquad (8.19)$$

(5) Alternative build-up of prestress may be assumed, if adequately justified and if the transmission length is modified accordingly.

8.10.2.3 Anchorage of tensile force for the ultimate limit state

(1) The anchorage of tendons should be checked in sections where the concrete tensile stress exceeds $f_{ctk,0,05}$. The tendon force should be calculated for a cracked section, including the effect of shear according to 6.2.3 (6); see also 9.2.1.3. Where the concrete tensile stress is less than $f_{ctk,0,05}$, no anchorage check is necessary.

(2) The bond strength for anchorage in the ultimate limit state is:

$$f_{bpd} = \eta_{p2}\ \eta_1\ f_{ctd} \qquad (8.20)$$

where:

η_{p2} is a coefficient that takes into account the type of tendon and the bond situation at anchorage

η_{p2} = 1,4 for indented wires or

η_{p2} = 1,2 for 7-wire strands

η_1 is as defined in 8.10.2.1 (1)

Note : Values of η_{p2} for types of tendons other than those given above may be used subject to a European Technical Approval.

(3) Due to increasing brittleness with higher concrete strength, $f_{ctk,0,05}$ should here be limited to the value for C60, unless it can be verified that the average bond strength increases above this limit.

(4) The total anchorage length for anchoring a tendon with stress σ_{pd} is:

$$l_{bpd} = l_{pt2} + \alpha_2\phi(\sigma_{pd} - \sigma_{pm\infty})/f_{bpd} \qquad (8.21)$$

where

l_{pt2} is the upper design value of transmission length, see 8.10.2.1 (3)

α_2 as defined in 8.10.2.1 (2)

σ_{pd} is the tendon stress corresponding to the force described in (1)
$\sigma_{pm\infty}$ is the prestress after all losses

(5) Tendon stresses in the anchorage zone are illustrated in Figure 8.17.

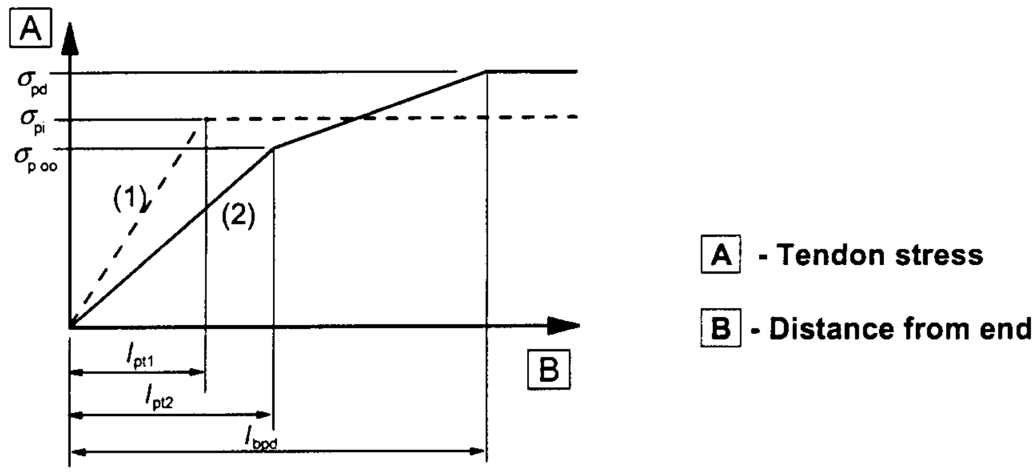


Figure 8.17 Stresses in the anchorage zone of pre-tensioned members: (1) at release of tendons, (2) at ultimate limit state

(6) In case of combined ordinary and pre-tensioned reinforcement, the anchorage capacities of both may be added.

8.10.3 Anchorage zones of post-tensioned members

(1) The design of anchorage zones should be in accordance with the application rules given in this section and those in 6.5.3.

(2) When considering the effects of the prestress as a concentrated force on the anchorage zone, the design value of the prestressing tendons should be in accordance with 2.4.2.2 (3) and the lower characteristic tensile strength of the concrete should be used.

(3) The bearing stress behind anchorage plates should be checked in accordance with the relevant European Technical Approval.

(4) Tensile forces due to concentrated forces should be assessed by a strut and tie model, or other appropriate representation (see 6.5). Reinforcement should be detailed assuming that it acts at its design strength. If the stress in this reinforcement is limited to 300 MPa no check of crackwidths is necessary.

(5) As a simplification the prestressing force may be assumed to disperse at an angle of spread 2β (see Figure 8.18), starting at the end of the anchorage device, where β may be assumed to be arc tan 2/3.

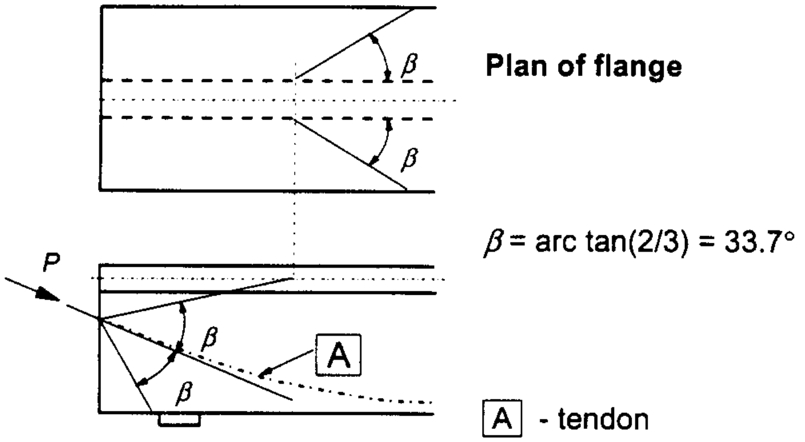


Figure 8.18 Dispersion of prestress

8.10.4 Anchorages and couplers for prestressing tendons

(1)P The anchorage devices used for post-tensioned tendons shall be in accordance with those specified for the prestressing system, and the anchorage lengths in the case of pre-tensioned tendons shall be such as to enable the full design strength of the tendons to be developed, taking account of any repeated, rapidly changing action effects.

(2)P Where couplers are used they shall be in accordance with those specified for the prestressing system and shall be so placed - taking account of the interference caused by these devices - that they do not affect the bearing capacity of the member and that any temporary anchorage which may be needed during construction can be introduced in a satisfactory manner.

(3) Calculations for local effects in the concrete and for the transverse reinforcement should be made in accordance with 6.5 and 8.10.3.

(4) In general, couplers should be located away from intermediate supports.

(5) The placing of couplers on 50% or more of the tendons at one cross-section should be avoided unless it can be shown that a higher percentage will not cause more risk to the safety of the structure.

8.10.5 Deviators

(1)P A deviator shall satisfy the following requirements:
- withstand both longitudinal and transverse forces that the tendon applies to it and transmit these forces to the structure;
- ensure that the radius of curvature of the prestressing tendon does not cause any overstressing or damage to it.

(2)P In the deviation zones the tubes forming the sheaths shall be able to sustain the radial pressure and longitudinal movement of the prestressing tendon, without damage and without impairing its proper functioning.

(3)P The radius of curvature of the tendon in a deviation zone shall be in accordance with EN 10138 and appropriate European Technical Approvals.

(4) Designed tendon deviations up to an angle of 0,01 radians may be permitted without using a deviator. The forces developed by the change of angle using a deviator in accordance with the relevant European Technical Approval should be taken into account in the design calculations.

SECTION 9 DETAILING OF MEMBERS AND PARTICULAR RULES

9.1 General

(1)P The requirements for safety, serviceability and durability are satisfied by following the rules given in this section in addition to the general rules given elsewhere.

(2) The detailing of members should be consistent with the design models adopted.

(3) Minimum areas of reinforcement are given in order to prevent a brittle failure, wide cracks and also to resist forces arising from restrained actions.

Note: The rules given in this section are mainly applicable to reinforced concrete buildings.

9.2 Beams

9.2.1 Longitudinal reinforcement

9.2.1.1 Minimum and maximum reinforcement areas

(1) The minimum area of longitudinal tension reinforcement should not be taken as less than $A_{s,min}$.

Note 1: See also 7.3 for area of longitudinal tension reinforcement to control cracking.

Note 2: The value of $A_{s,min}$ for beams for use in a Country may be found in its National Annex. The recommended value is given in the following:

$$A_{s,min} = 0{,}26\frac{f_{ctm}}{f_{yk}}b_t d \quad \text{but not less than } 0{,}0013 b_t d \qquad (9.1N)$$

Where:

b_t denotes the mean width of the tension zone; for a T-beam with the flange in compression, only the width of the web is taken into account in calculating the value of b_t.

f_{ctm} should be determined with respect to the relevant strength class according to Table 3.1.

Alternatively, for secondary elements, where some risk of brittle failure may be accepted, $A_{s,min}$ may be taken as 1,2 times the area required in ULS verification.

(2) Sections containing less reinforcement than $A_{s,min}$ should be considered as unreinforced (see Section 12).

(3) The cross-sectional area of tension or compression reinforcement should not exceed $A_{s,max}$ outside lap locations.

Note: The value of $A_{s,max}$ for beams for use in a Country may be found in its National Annex. The recommended value is $0{,}04A_c$.

(4) For members prestressed with permanently unbonded tendons or with external prestressing cables, it should be verified that the ultimate bending capacity is larger than the flexural cracking moment. A capacity of 1,15 times the cracking moment is sufficient.

9.2.1.2 Other detailing arrangements

(1) In monolithic construction, even when simple supports have been assumed in design, the section at supports should be designed for a bending moment arising from partial fixity of at least β_1 of the maximum bending moment in the span.

Note 1: The value of β_1 for beams for use in a Country may be found in its National Annex. The recommended value is 0,15.

Note 2: The minimum area of longitudinal reinforcement section defined in 9.2.1.1 (1) applies.

(2) At intermediate supports of continuous beams, the total area of tension reinforcement A_s of a flanged cross-section should be spread over the effective width of flange (see 5.3.2). Part of it may be concentrated over the web width (See Figure 9.1).

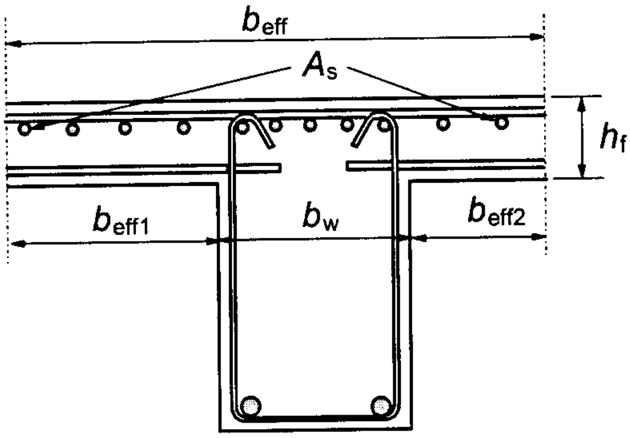


Figure 9.1: Placing of tension reinforcement in flanged cross-section.

(3) Any compression longitudinal reinforcement (diameter ϕ) which is included in the resistance calculation should be held by transverse reinforcement with spacing not greater than 15ϕ.

9.2.1.3 Curtailment of longitudinal tension reinforcement

(1) Sufficient reinforcement should be provided at all sections to resist the envelope of the acting tensile force, including the effect of inclined cracks in webs and flanges.

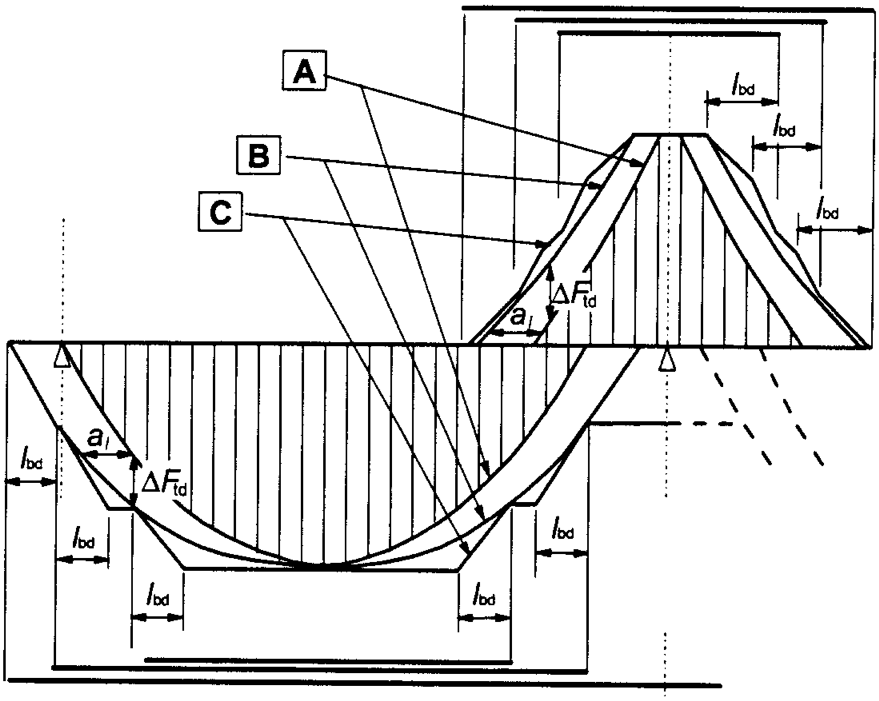
(2) For members with shear reinforcement the additional tensile force, F_{td}, should be calculated according to 6.2.3 (6). For members without shear reinforcement F_{td} may be estimated by shifting the moment curve a distance $a_l = d$ according to 6.2.2 (5). This "shift rule" may also be used as an alternative for members with shear reinforcement, where:

$$a_l = z\,(\cot\theta - \cot\alpha)/2 \quad \text{(symbols defined in 6.2.3)} \tag{9.2}$$

The additional tensile force is illustrated in Figure 9.2.

(3) The resistance of bars within their anchorage lengths may be taken into account, assuming a linear variation of force, see Figure 9.2. As a conservative simplification this contribution may be ignored.

(4) The anchorage length of a bent-up bar which contributes to the resistance to shear should be not less than 1,3 l_{bd} in the tension zone and 0,7 l_{bd} in the compression zone. It is measured from the point of intersection of the axes of the bent-up bar and the longitudinal reinforcement.



A - Envelope of $M_{Ed}/z + N_{Ed}$ B - acting tensile force F_s C - resisting tensile force F_{Rs}

Figure 9.2: Illustration of the curtailment of longitudinal reinforcement, taking into account the effect of inclined cracks and the resistance of reinforcement within anchorage lengths

9.2.1.4 Anchorage of bottom reinforcement at an end supports

(1) The area of bottom reinforcement provided at supports with little or no end fixity assumed in design, should be at least β_2 of the area of steel provided in the span.

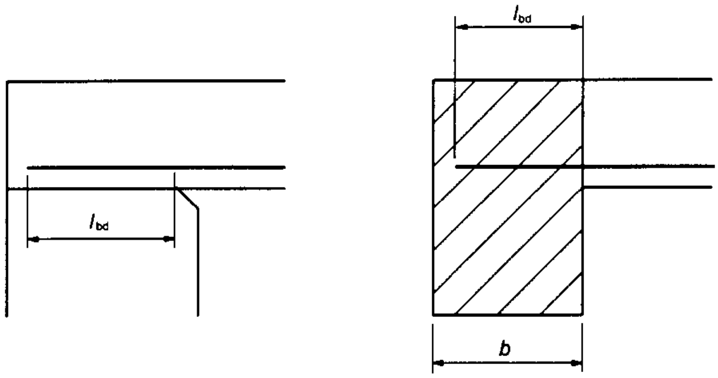
Note: The value of β_2 for beams for use in a Country may be found in its National Annex. The recommended value is 0,25.

(2) The tensile force to be anchored may be determined according to 6.2.3 (6) (members with shear reinforcement) including the contribution of the axial force if any, or according to the shift rule:

$$F_E = |V_{Ed}| \cdot a_l / z + N_{Ed} \tag{9.3}$$

where N_{Ed} is the axial force, to be added to or subtracted from the tensile force; a_l see 9.2.1.3 (2).

(3) The anchorage length is l_{bd} according to 8.4.4, measured from the line of contact between beam and support. Transverse pressure may be taken into account for direct support. See Figure 9.3.



a) Direct support: Beam supported by wall or column **b) Indirect support:** Beam intersecting another supporting beam

Figure 9.3: Anchorage of bottom reinforcement at end supports

9.2.1.5 Anchorage of bottom reinforcement at intermediate supports

(1) The area of reinforcement given in 9.2.1.4 (1) applies.

(2) The anchorage length should not be less than 10ϕ (for straight bars) or not less than the diameter of the mandrel (for hooks and bends with bar diameters at least equal to 16 mm) or twice the diameter of the mandrel (in other cases) (see Figure 9.4 (a)). These minimum values are normally valid but a more refined analysis may be carried out in accordance with 6.6.

(3) The reinforcement required to resist possible positive moments (e.g. settlement of the support, explosion, etc.) should be specified in contract documents. This reinforcement should be continuous which may be achieved by means of lapped bars (see Figure 9.4 (b) or (c)).

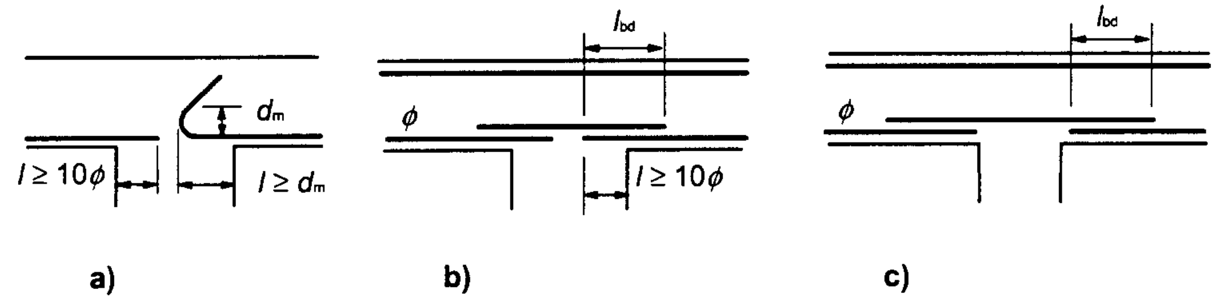


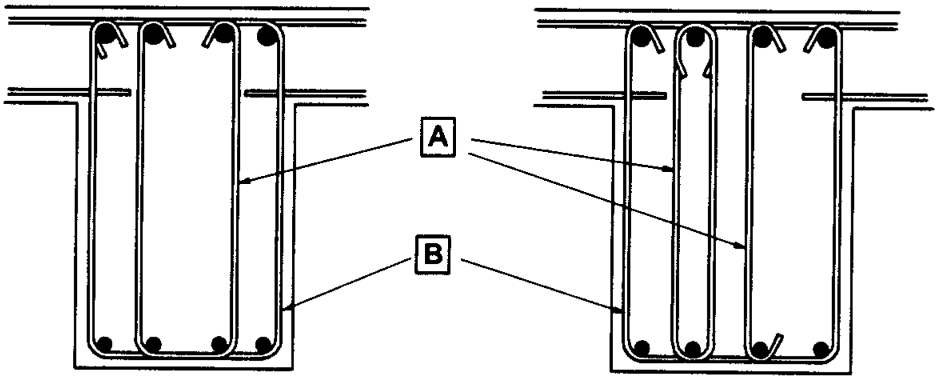
Figure 9.4: Anchorage at intermediate supports

9.2.2 Shear reinforcement

(1) The shear reinforcement should form an angle α of between 45° and 90° to the longitudinal axis of the structural element.

(2) The shear reinforcement may consist of a combination of:
- links enclosing the longitudinal tension reinforcement and the compression zone (see Figure 9.5);
- bent-up bars;

- cages, ladders, etc. which are cast in without enclosing the longitudinal reinforcement but are properly anchored in the compression and tension zones.



A Inner link alternatives B Enclosing link

Figure 9.5: Examples of shear reinforcement

(3) Links should be effectively anchored. A lap joint on the leg near the surface of the web is permitted provided that the link is not required to resist torsion.

(4) At least β_3 of the necessary shear reinforcement should be in the form of links.

Note: The value of β_3 for beams for use in a Country may be found in its National Annex. The recommended value is 0, 5.

(5) The ratio of shear reinforcement is given by Expression (9.4):

$$\rho_w = A_{sw} / (s \cdot b_w \cdot \sin\alpha) \tag{9.4}$$

where:

ρ_w is the shear reinforcement ratio
ρ_w should not be less than $\rho_{w,min}$
A_{sw} is the area of shear reinforcement within length s
s is the spacing of the shear reinforcement measured along the longitudinal axis of the member
b_w is the breadth of the web of the member
α is the angle between the shear reinforcement and the longitudinal axis (see (1) above)

Note: The value of $\rho_{w,min}$ for beams for use in a Country may be found in its National Annex. The recommended value is given Expression (9.5N)

$$\rho_{w,min} = (0{,}08\sqrt{f_{ck}})/f_{yk} \tag{9.5N}$$

(6) The maximum longitudinal spacing between shear assemblies should not exceed $s_{l,max}$.

Note: The value of $s_{l,max}$ for use in a Country may be found in its National Annex. The recommended value is given by Expression (9.6N)

$$s_{max} = 0{,}75d\,(1 + \cot\alpha) \tag{9.6N}$$

where α is the inclination of the shear reinforcement to the longitudinal axis of the beam.

(7) The maximum longitudinal spacing of bent-up bars should not exceed $s_{b,max}$:

Note: The value of $s_{b,max}$ for use in a Country may be found in its National Annex. The recommended value is given by Expression (9.7N)

$$s_{b,max} = 0,6\, d\, (1 + \cot \alpha) \tag{9.7N}$$

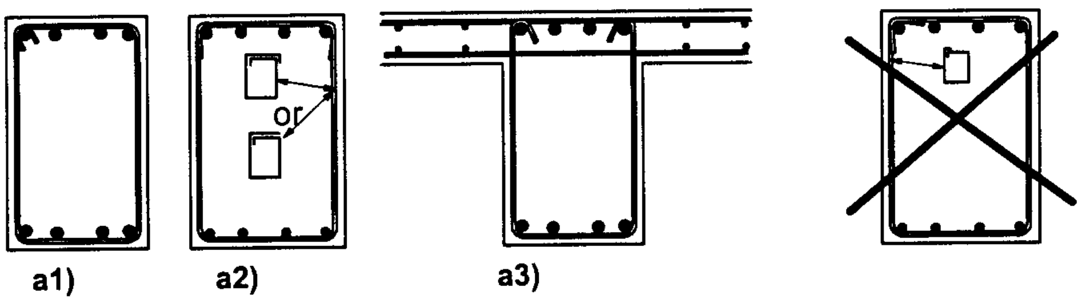
(8) The transverse spacing of the legs in a series of shear links should not exceed $s_{t,max}$:

Note: The value of $s_{t,max}$ for use in a Country may be found in its National Annex. The recommended value is given by Expression (9.8N)

$$s_{t,max} = 0,75d \leq 600 \text{ mm} \tag{9.8N}$$

9.2.3 Torsion reinforcement

(1) The torsion links should be closed and be anchored by means of laps or hooked ends, see Figure 9.6, and should form an angle of 90° with the axis of the structural element.



a) recommended shapes **b) not recommended shape**

Note: The second alternative for a2) should have a full lap length along the top.

Figure 9.6: Examples of shapes for torsion links

(2) The provisions of 9.2.2 (5) and (6) are generally sufficient to provide the minimum torsion links required.

(3) The longitudinal spacing of the torsion links should not exceed u / 8 (see 6.3.2, Figure 6.11, for the notation), or the requirement in 9.2.2 (6) or the lesser dimension of the beam cross-section.

(4) The longitudinal bars should be so arranged that there is at least one bar at each corner, the others being distributed uniformly around the inner periphery of the links, with a spacing not greater than 350 mm.

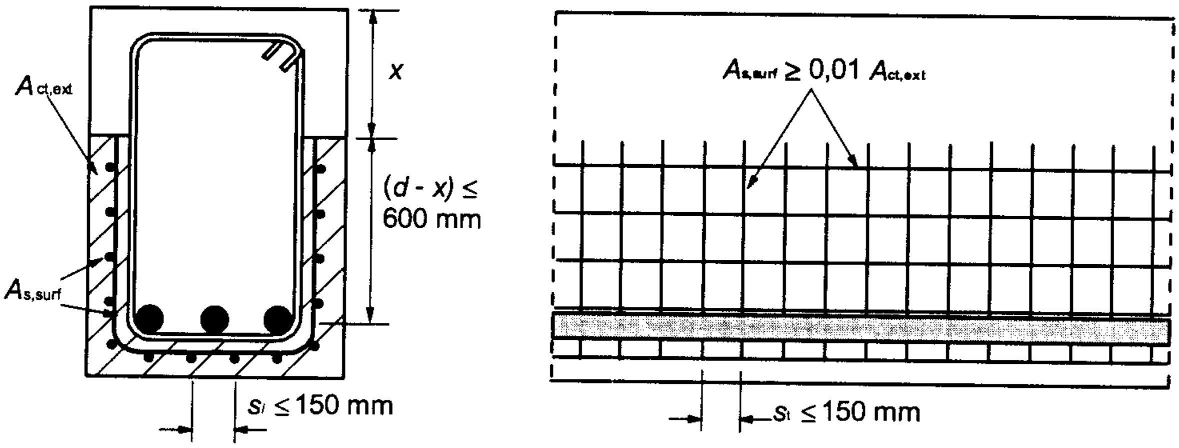
9.2.4 Surface reinforcement

(1) It may be necessary to provide surface reinforcement either to control cracking or to ensure adequate resistance to spalling of the cover.

(2) Surface reinforcement to resist spalling should be used where the main reinforcement is made up of:

- bars with diameter greater than 32 mm or
- bundled bars with equivalent diameter greater than 32 mm (see 8.8)

The surface reinforcement should consist of wire mesh or small diameter bars, and be placed outside the links as indicated in Figure 9.7.



x is the depth of the neutral axis at ULS

Figure 9.7: Example of surface reinforcement

(3) The area of surface reinforcement $A_{s,surf}$ should be not less than $A_{s,surfmin}$ in the two directions parallel and orthogonal to the tension reinforcement in the beam,

Note: The value of $A_{s,surfmin}$ for use in a Country may be found in its National Annex. The recommended value is 0,01 $A_{ct,ext}$, where $A_{ct,ext}$ is the area of the tensile concrete external to the links (see Figure 9.7).

(4) Where the cover to reinforcement is greater than 70 mm, for enhanced durability similar surface reinforcement should be used, with an area of 0,005 $A_{ct,ext}$ in each direction.

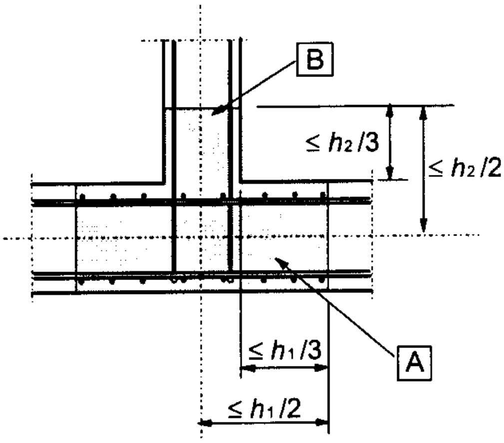
(5) The minimum cover needed for the surface reinforcement is given in 4.4.1.2.

(6) The longitudinal bars of the surface reinforcement may be taken into account as longitudinal bending reinforcement and the transverse bars as shear reinforcement provided that they meet the requirements for the arrangement and anchorage of these types of reinforcement.

9.2.5 Indirect supports

(1) Where a beam is supported by a beam instead of a wall or column, reinforcement should be provided and designed to resist the mutual reaction. This reinforcement is in addition to that required for other reasons. This rule also applies to a slab not supported at the top of a beam.

(2) The supporting reinforcement between two beams should consist of links surrounding the principal reinforcement of the supporting member. Some of these links may be distributed outside the volume of the concrete, which is common to the two beams, as indicated in Figure 9.8.



A supporting beam with height h_1 B supported beam with height h_2 ($h_1 \geq h_2$)

Figure 9.8: Placing of supporting reinforcement in the intersection zone of two beams (plan view)

9.3 Solid slabs

(1) This section applies to one-way and two-way solid slabs for which b and l_{eff} are not less than $5h$ (see 5.3.1).

9.3.1 Flexural reinforcement

9.3.1.1 General

(1) For the minimum and the maximum steel percentages in the main direction 9.2.1.1 (1) and (3) apply.

Note: In addition to Note 2 of 9.2.1.1 (1), for slabs where the risk of brittle failure is small, $A_{s,min}$ may be taken as 1,2 times the area required in ULS verification.

(2) Secondary transverse reinforcement of not less than 20% of the principal reinforcement should be provided in one way slabs. In areas near supports transverse reinforcement to principal top bars is not necessary where there is no transverse bending moment.

(3) The spacing of bars should not exceed $s_{max,slabs}$.

Note; The value of $s_{max,slabs}$ for use in a Country may be found in its National Annex. The recommended value is:
- for the principal reinforcement, $3h \leq 400$ mm, where h is the total depth of the slab;
- for the secondary reinforcement, $3{,}5h \leq 450$ mm .

In areas with concentrated loads or areas of maximum moment those provisions become respectively:
- for the principal reinforcement, $2h \leq 250$ mm
- for the secondary reinforcement, $3h \leq 400$ mm.

(4) The rules given in 9.2.1.3 (1) to (3), 9.2.1.4 (1) to (3) and 9.2.1.5 (1) to (2) also apply but with $a_l = d$.

9.3.1.2 Reinforcement in slabs near supports

(1) In simply supported slabs, half the calculated span reinforcement should continue up to the support and be anchored therein in accordance with 8.4.4.

Note: Rules for curtailing and anchoring reinforcement may be carried out according to 9.2.1.3, 9.2.1.4 and 9.2.1.5.

(2) Where partial fixity occurs along an edge of a slab, but is not taken into account in the analysis, the top reinforcement should be capable of resisting at least 25% of the maximum moment in the adjacent span. This reinforcement should extend at least 0,2 times the length of the adjacent span, measured from the face of the support. It should be continuous across internal supports and anchored at end supports. At an end support the moment to be resisted may be reduced to 15% of the maximum moment in the adjacent span.

9.3.1.3 Corner reinforcement

(1) If the detailing arrangements at a support are such that lifting of the slab at a corner is restrained, suitable reinforcement should be provided.

9.3.1.4 Reinforcement at the free edges

(1) Along a free (unsupported) edge, a slab should normally contain longitudinal and transverse reinforcement, generally arranged as shown in Figure 9.9.

(2) The normal reinforcement provided for a slab may act as edge reinforcement.

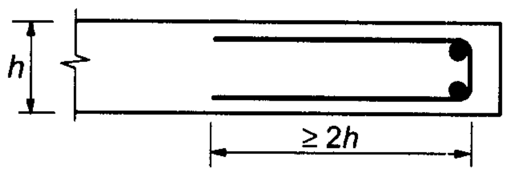


Figure 9.9: Edge reinforcement for a slab

9.3.2 Shear reinforcement

(1) A slab in which shear reinforcement is provided should have a depth of at least 200 mm.

(2) In detailing the shear reinforcement, the minimum value and definition of reinforcement ratio in 9.2.2 apply, unless modified by the following.

(3) In slabs, if $|V_{Ed}| \leq 1/3\ V_{Rd,max}$, (see 6.2), the shear reinforcement may consist entirely of bent-up bars or of shear reinforcement assemblies.

(4) The maximum longitudinal spacing of successive series of links is given by:

$$s_{max} = 0{,}75d(1+\cot\alpha) \qquad (9.9)$$

where α is the inclination of the shear reinforcement.

The maximum longitudinal spacing of bent-up bars is given by:

$$s_{max} = d. \tag{9.10}$$

(5) The maximum transverse spacing of shear reinforcement should not exceed 1,5*d*.

9.4 Flat slabs

9.4.1 Slab at internal columns

(1) At internal columns in flat slab construction, top reinforcement of area 0,67 A_{cs} should be placed symmetrically about the column centreline in a width equal to half that of the column strip. A_{cs} represents the area of reinforcement required to resist the negative moment in the column strip (see Figure 9.10).

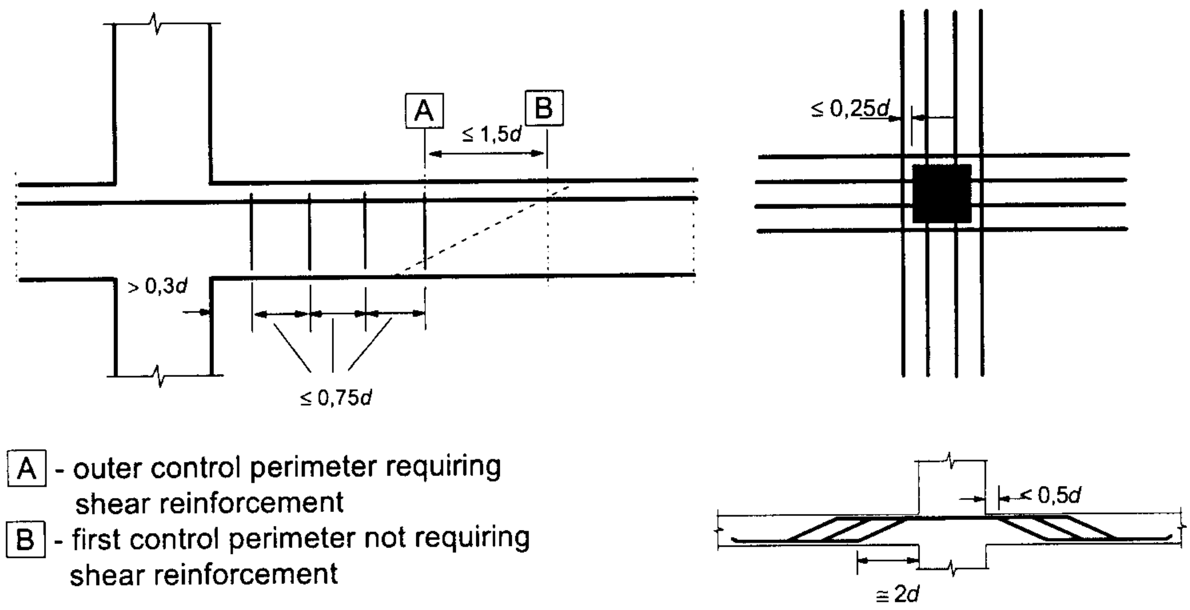
(2) Bottom reinforcement (≥ 2 bars) in each orthogonal direction should be provided at internal columns and this reinforcement should pass through the column.

9.4.2 Slab at edge columns

(1) Reinforcement perpendicular to a free edge required to transmit bending moments from the slab to an edge column should be placed within the effective breadth b_e shown in Figure 9.11.

9.4.3 Punching shear reinforcement

(1) Where punching shear reinforcement is required (see 6.4) it should be placed between the loaded area/column and 1.5*d* inside the control perimeter at which shear reinforcement is no longer required. It should be provided in at least two perimeters of link legs (see Figure 9.12). The spacing of the link leg perimeters should not exceed 0,75*d*.



a) Spacing of links **b) Spacing of bent-up bars**

Figure 9.12: Punching shear reinforcement

The spacing of link legs around a perimeter should not exceed 1,5*d* within the first control perimeter (2*d* from loaded area), and should not exceed 2*d* for perimeters outside the first control perimeter where that part of the perimeter is assumed to contribute to the shear capacity (see Figure 6.22).

For bent down bars as arranged in Figure 9.12(b) one perimeter of link legs may be considered sufficient.

(2) Where shear reinforcement is required the area of a link leg (or equivalent), $A_{sw,min}$, is given by Expression (9.11).

$$A_{sw,min} \cdot (1{,}5 \cdot \sin\alpha + \cos\alpha)/(s_r \cdot s_t) \geq (0{,}08 \cdot \sqrt{(f_{ck})}/f_{yk} \quad (9.11)$$

where :

α is the angle between the shear reinforcement and the main steel (i.e. for vertical links α = 90° and sin α = 1)
s_r is the spacing of shear links in the radial direction
s_t is the spacing of shear links in the tangential direction
f_{ck} is in MPa

(3) Bent-up bars passing through the loaded area or at a distance not exceeding 0,25*d* from this area may be used as shear reinforcement.

(4) The distance between the face of a support, or the circumference of a loaded area, and the nearest shear reinforcement taken into account in the design should not exceed *d*/2. This distance should be taken at the level of the tensile reinforcement; if only a single line of bent-up bars is provided, their slope may be reduced to 30°.

9.5 Columns

9.5.1 General

(1) This clause deals with columns for which the larger dimension *h* is not greater than 4 times the smaller dimension *b*.

9.5.2 Longitudinal reinforcement

(1) Bars should have a diameter of not less than ϕ_{min}.

Note: The value of ϕ_{min} for use in a Country may be found in its National Annex. The recommended value is 8 mm.

(2) The minimum amount of total longitudinal reinforcement should not exceed $A_{s,min}$.

Note: The value of $A_{s,min}$ for use in a Country may be found in its National Annex. The recommended value is given by Expression (9.12N)

$$A_{s,min} = \frac{0{,}10\ N_{Ed}}{f_{yd}} \text{ or } 0{,}002\ A_c \text{ whichever is the greater} \quad (9.12N)$$

where:

f_{yd} is the design yield strength of the reinforcement
N_{Ed} is the design axial compression force

(3) The area of reinforcement should not exceed $A_{s,max}$.

Note: The value of $A_{s,min}$ for use in a Country may be found in its National Annex. The recommended value is 0,04 A_c outside lap locations unless it can be shown that the integrity of concrete is not affected, and that the full strength is achieved at ULS. This limit should be increased to 0,08 A_c at laps.

(4) For columns having a polygonal cross-section, at least one bar should be placed at each corner. The number of longitudinal bars in a circular column should not be less than four.

9.5.3 Transverse reinforcement

(1) The diameter of the transverse reinforcement (links, loops or helical spiral reinforcement) should not be less than 6 mm or one quarter of the maximum diameter of the longitudinal bars, whichever is the greater. The diameter of the wires of welded mesh fabric for transverse reinforcement should not be less than 5 mm.

(2) The transverse reinforcement should be anchored adequately.

(3) The spacing of the transverse reinforcement along the column should not exceed $s_{cl,tmax}$

Note: The value of $s_{cl,tmax}$ for use in a Country may be found in its National Annex. The recommended value is the least of the following three distances:
- 20 times the minimum diameter of the longitudinal bars
- the lesser dimension of the column
- 400 mm

(4) The maximum spacing required in (3) should be reduced by a factor 0,6:

(i) in sections within a distance equal to the larger dimension of the column cross-section above or below a beam or slab;

(ii) near lapped joints, if the maximum diameter of the longitudinal bars is greater than 14 mm. A minimum of 3 bars evenly placed in the lap length is required.

(5) Where the direction of the longitudinal bars changes, (e.g. at changes in column size), the spacing of transverse reinforcement should be calculated, taking account of the lateral forces involved. These effects may be ignored if the change of direction is less than or equal to 1 in 12.

(6) Every longitudinal bar or bundled bars placed in a corner should be held by transverse reinforcement. No bar within a compression zone should be further than 150 mm from a restrained bar.

9.6 Walls

9.6.1 General

(1) This clause refers to reinforced concrete walls with a length to thickness ratio of 4 or more and in which the reinforcement is taken into account in the strength analysis. The amount and proper detailing of reinforcement may be derived from a strut-and-tie model (see 6.5). For walls subjected predominantly to out-of-plane bending the rules for slabs apply (see 9.3).

9.6.2 Vertical reinforcement

(1) The area of the vertical reinforcement should lie between $A_{s,vmin}$ and $A_{s,vmax}$.

Note 1: The value of $A_{s,vmin}$ for use in a Country may be found in its National Annex. The recommended value is 0,002 A_c.

Note 2: The value of $A_{s,vmax}$ for use in a Country may be found in its National Annex. The recommended value is 0,04 A_c outside lap locations unless it can be shown that the concrete integrity is not affected and that the full strength is achieved at ULS. This limit may be doubled at laps.

(2) Where the minimum area of reinforcement controls in design, half of this area should be located at each face.

(3) The distance between two adjacent vertical bars shall not exceed 3 times the wall thickness or 400 mm whichever is the lesser.

9.6.3 Horizontal reinforcement

(1) Horizontal reinforcement running parallel to the faces of the wall (and to the free edges) should be provided at each surface. It should not be less than $A_{s,hmin}$.

Note: The value of $A_{s,hmin}$ for use in a Country may be found in its National Annex. The recommended value is either 25% of the vertical reinforcement or 0,001 A_c.

(2) The spacing between two adjacent horizontal bars should not be greater than 400 mm.

9.6.4 Transverse reinforcement

(1) In any part of a wall where the total area of the vertical reinforcement in the two faces exceeds 0,02 A_c transverse reinforcement in the form of links should be provided in accordance with the requirements for columns (see 9.4.2).

(2) Where main the reinforcement is placed nearest to the wall faces, transverse reinforcement should also be provided in the form of links with at least of 4 per m^2 of wall area.

Note: Transverse reinforcement need not be provided where welded wire mesh and bars of diameter $\phi \leq 16$ mm are used with concrete cover larger than 2ϕ,

9.7 Deep beams

(1) Deep beams (for definition see 5.3.1 (2), (3)) should normally be provided with an orthogonal reinforcement mesh near each face, with a minimum of $A_{s,dbmin}$.

Note: The value of $A_{s,dbmin}$ for use in a Country may be found in its National Annex. The recommended value is 0,1% but not less than 150 mm²/m in each face and each direction.

(2) The distance between two adjacent bars of the mesh should not exceed the lesser of twice the wall thickness or 300 mm.

(3) Reinforcement, corresponding to the ties considered in the design model, should be fully anchored for equilibrium in the node, see 6.5.4, by bending the bars, by using U-hoops or by anchorage devices, unless a sufficient length is available between the node and the end of the beam permitting an anchorage length of l_{bd}.

9.8 Foundations

9.8.1 Pile caps

(1) The distance from the outer edge of the pile to the edge of the pile cap should be such that the tie forces in the pile cap can be properly anchored. The expected deviation of the pile on site should be taken into account.

(2) Reinforcement in a pile cap should be calculated either by using strut-and-tie or flexural methods as appropriate.

(3) The main tensile reinforcement to resist the action effects should be concentrated in the stress zones between the tops of the piles. A minimum bar diameter d_{min} should be provided. If the area of this reinforcement is at least equal to the minimum reinforcement, evenly distributed bars along the bottom surface of the member may be omitted. Also the sides and the top surface of the member may be unreinforced if there is no risk of tension developing in these parts of the member.

Note: The value of d_{min} for use in a Country may be found in its National Annex. The recommended value is 8 mm.

(4) Welded transverse bars may be used for the anchorage of the tension reinforcement. In this case the transverse bar may be considered to be part of the transverse reinforcement in the anchorage zone of the reinforcement bar considered.

(5) The compression caused by the support reaction from the pile may be assumed to spread at 45 degree angles from the edge of the pile (see Figure 9.13). This compression may be taken into account when calculating the anchorage length.

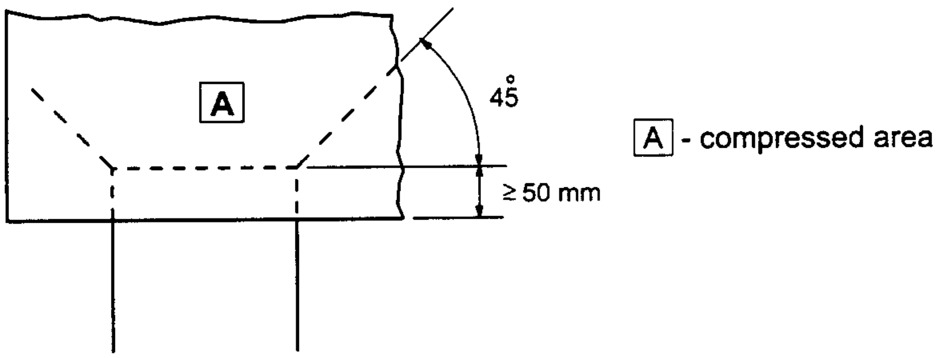


Figure 9.13: Compressed area increasing the anchorage capacity

9.8.2 Column and wall footings

9.8.2.1 General

(1) The main reinforcement should be anchored in accordance with the requirements of 8.4 and 8.5. A minimum bar diameter d_{min} should be provided. In footings the design model shown in 9.8.2.1 may be used.

Note: The value of d_{min} for use in a Country may be found in its National Annex. The recommended value is 8. mm.

(2) The main reinforcement of circular footings may be orthogonal and concentrated in the middle of the footing for a width of 50% ± 10% of the diameter of the footing, see Figure 9.14. In this case the unreinforced parts of the structure should be considered as plain concrete for design purposes.

(3) If the action effects cause tension at the upper surface of the footing, the resulting tensile stresses should be checked and reinforced as necessary.

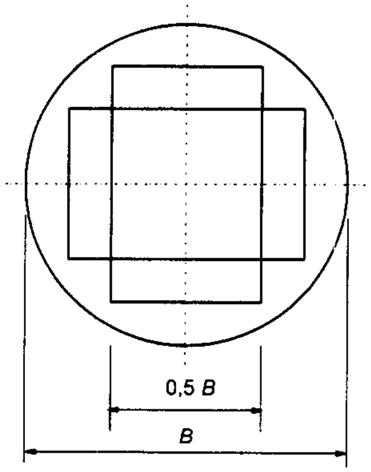


Figure 9.14: Orthogonal reinforcement in circular spread footing on soil

9.8.2.2 Anchorage of bars

(1) The tensile force in the reinforcement is determined from equilibrium conditions, taking into account the effect of inclined cracks, see Figure 9.15. The tensile force F_s at a location x should be anchored in the concrete within the same distance x from the edge of the footing.

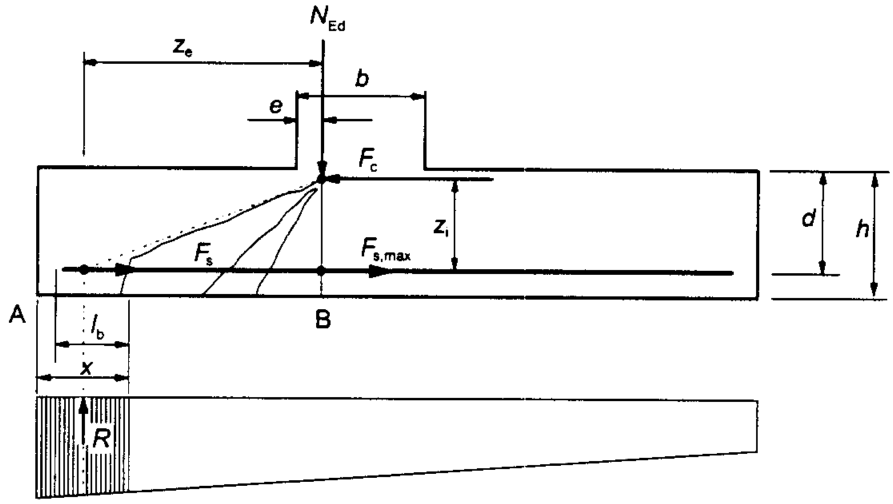


Figure 9.15: Model for tensile force with regard to inclined cracks

(2) The tensile force to be anchored is given by:

$$F_s = R \cdot z_e/z_i \tag{9.13}$$

where:

R is the resultant of ground pressure within distance x

z_e is the external lever arm, i.e. distance between R and the vertical force N_{Ed}

N_{Ed} is the vertical force corresponding to total ground pressure between sections A and B

z_i is the internal lever arm, i.e. distance between the reinforcement and the horizontal force F_c

F_c is the compressive force corresponding to maximum tensile force $F_{s,max}$

(3) Lever arms z_e and z_i may be determined with regard to the necessary compression zones for N_{Ed} and F_c respectively. As simplifications, z_e may be determined assuming $e = 0,15b$, see Figure 9.15 and z_i may be taken as $0,9d$.

(4) The available anchorage length for straight bars is denoted l_b in Figure 9.15. If this length is not sufficient to anchor F_s, bars may either be bent up to increase the available length or be provided with end anchorage devices.

(5) For straight bars without end anchorage the minimum value of x is the most critical. As a simplification $x_{min} = h/2$ may be assumed. For other types of anchorage, higher values of x may be more critical.

9.8.3 Tie beams

(1) Tie beams may be used to eliminate the eccentricity of loading of the foundations. The beams should be designed to resist the resulting bending moments and shear forces. A minimum bar diameter d_{min} for the reinforcement resisting bending moments should be provided.

Note: The value of d_{min} for use in a Country may be found in its National Annex. The recommended value is 8 mm.

(2) Tie beams should also be designed for a minimum downward load of q_1 if the action of compaction machinery can cause effects to the tie beams.

Note: The value of q_1 for use in a Country may be found in its National Annex. The recommended value is 10 kN/m.

9.8.4 Column footing on rock

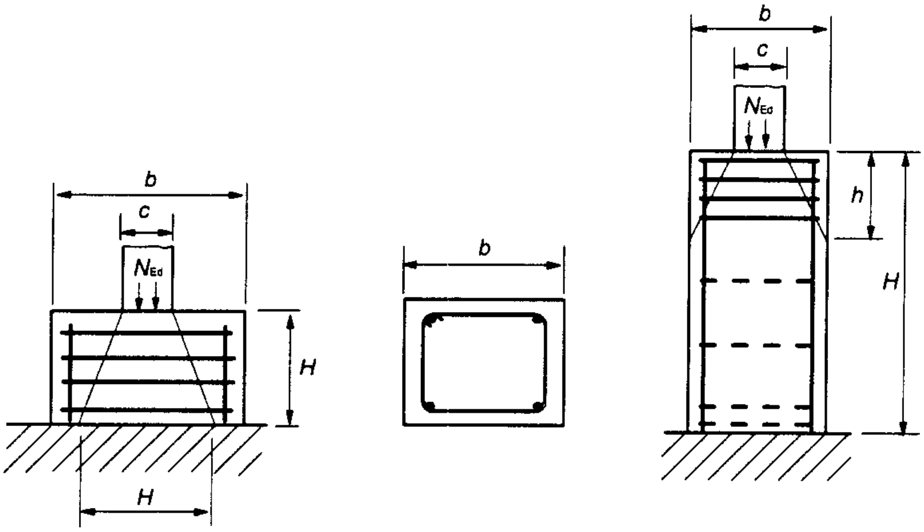
(1) Adequate transverse reinforcement should be provided to resist the splitting forces in the footing, when the ground pressure in the ultimate states exceeds q_2. This reinforcement may be distributed uniformly in the direction of the splitting force over the height h (see Figure 9.16). A minimum bar diameter d_{min} should be provided.

Note: The values of q_2 and of d_{min} for use in a Country may be found in its National Annex. The recommended values of q_2 is 5 MPa and of d_{min} is 8 mm.

(2) The splitting force, F_s, may be calculated as follows (see Figure 9.16) :

$$F_s = 0,25\ (1 - c\ /h)N_{Ed} \tag{9.14}$$

Where h is the lesser of b and H



a) footing with $h \geq H$ b) section c) footing with $h < H$

Figure 9.16: Splitting reinforcement in footing on rock

9.8.5 Bored piles

(1) The following clauses apply for reinforced bored piles. For unreinforced bored piles see Section 12.

(2) In order to allow the free flow of concrete around the reinforcement it is of primary importance that reinforcement, reinforcement cages and any attached inserts are detailed such that the flow of concrete is not adversely affected.

(3) Bored piles with diameters not exceeding h_1 should be provided with a minimum longitudinal reinforcement area $A_{s,bpmin}$ and detailing rules.

Note: The values of h_1 and $A_{s,bpmin}$ for use in a Country may be found in its National Annex. The recommended value of h_1 is 600 mm and of $A_{s,bpmin}$ is given in Table 9.6N. This reinforcement should be distributed along the periphery of the section.

Table 9.6N: Recommended minimum longitudinal reinforcement area in cast-in-place bored piles

Pile cross-section: A_c	**Minimum area of longitudinal reinforcement: $A_{s,bpmin}$**
$A_c \leq 0,5$ m²	$A_s \geq 0,005 \cdot A_c$
0,5 m² $< A_c \leq 1,0$ m²	$A_s \geq 25$ cm²
$A_c > 1,0$ m²	$A_s \geq 0,0025 \cdot A_c$

The minimum diameter for the longitudinal bars should not be less than 16 mm. Piles should have at least 6 longitudinal bars. The clear distance between bars should not exceed 200 mm measured along the periphery of the pile.

(4) The minimum diameter for the longitudinal bars should not be less than d_2. Piles should have at least n_1 longitudinal bars. The clear distance between bars should not exceed s_1 measured along the periphery of the pile.

Note: Values of d_2, n_1 and s_1 for use in a Country may be found in its National Annex. The recommended value of d_2 1s 16 mm, of n_1 is 6 and of n_1 is 200 mm.

(4) For the detailing of longitudinal and transverse reinforcement in bored piles, see EN 1536.

9.9 Regions with discontinuity in geometry or action

P(1) D-Regions shall be designed with strut-and-tie models according to section 6.5 and detailed according to the rules given in Section 8. The following simple rules are deemed to satisfy this requirement.

Note: Further information is given in Annex I.

P(2) The reinforcement, corresponding to the ties, shall be fully anchored by an anchorage of l_{bd} according to 8.4.

9.10 Tying systems

9.10.1 General

(1)P Structures which are not designed to withstand accidental actions shall have a suitable tying system, to prevent progressive collapse by providing alternative load paths after local damage. The following simple rules are deemed to satisfy this requirement.

(2) The following ties should be provided:
a) peripheral ties
b) internal ties
c) horizontal column or wall ties
d) where required, vertical ties, particularly in panel buildings.

(3) Where a building is divided by expansion joints into structurally independent sections, each section should have an independent tying system.

(4) In the design of the ties the reinforcement may be assumed to be acting at its characteristic strength and capable of carrying tensile forces defined in the following clauses.

(5) Reinforcement provided for other purposes in columns, walls, beams and floors may be regarded as providing part of or the whole of these ties depending on the loading case considered.

9.10.2 Proportioning of ties

9.10.2.1 General

(1) Ties are intended as a minimum and not as an additional reinforcement to that required by structural analysis as per section 5.1.3 and other actions.

9.10.2.2 Peripheral ties

(1) At each floor and roof level an effectively continuous peripheral tie within 1,2 m from the edge should be provided. The tie may include reinforcement used as part of the internal tie.

(2) The peripheral tie should be capable of resisting a tensile force:

$$F_{tie,per} = l_i \cdot q_3 \leq q_4 \tag{9.15}$$

where:
$F_{tie,per}$ tie force (here: tension)
l_i length of the end-span

Note: Values of q_3 and q_4 for use in a Country may be found in its National Annex. The recommended value of q_3 iis 10 kN/m and of q_4 is 70 kN.

(3) Structures with internal edges (e.g. atriums, courtyards, etc.) should have peripheral ties in the same way as external edges which shall be fully anchored.

9.10.2.3 Internal ties

(1) These ties should be at each floor and roof level in two directions approximately at right angles. They should be effective continuous throughout their length and should be anchored to the peripheral ties at each end, unless continuing as horizontal ties to columns or walls.

(2) The internal ties may, in whole or in part, be spread evenly in the slabs or may be grouped at or in beams, walls or other appropriate positions. In walls they should be within 0,5 m from the top or bottom of floor slabs, see Figure 9.22.

(3) In each direction, internal ties should be capable of resisting a design value of tensile force $F_{tie,int}$ (in kN per metre width):

Note: Values of $F_{tie,int}$ for use in a Country may be found in its National Annex. The recommended value is 20 kN/m.

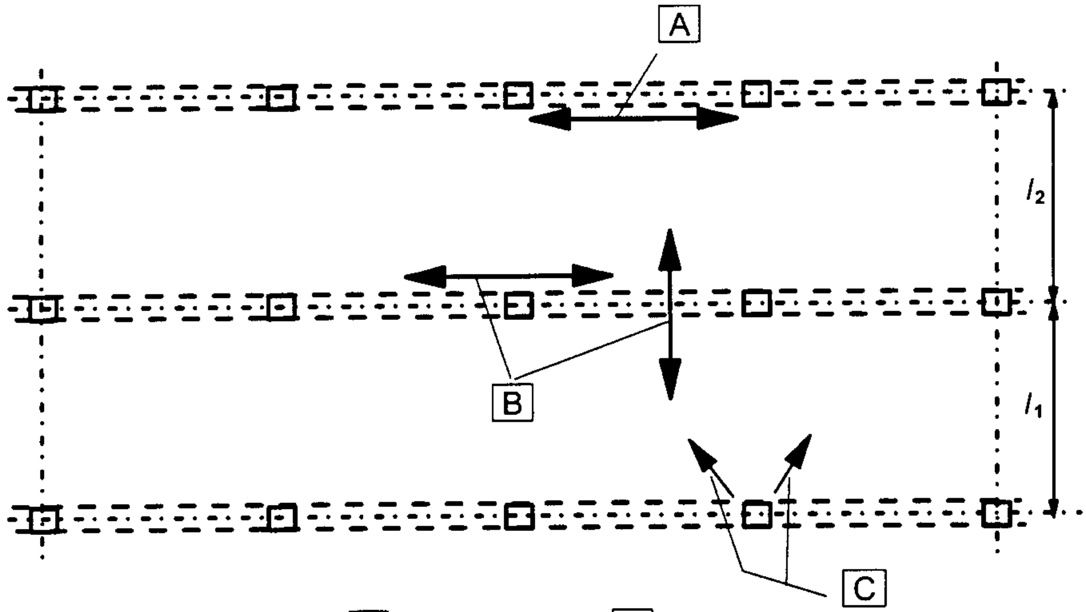
(4) In floors without screeds where ties cannot be distributed across the span direction, the transverse ties may be grouped along the beam lines. In this case the minimum force on an internal beam line is:

$$F_{tie} = (l_1 + l_2)/\,2 \cdot q_4 \leq q_5 \tag{9.16}$$

where:
l_1, l_2 are the span lengths (in m) of the floor slabs on either side of the beam (see Figure 9.22)

Note: Values of q_4 and q_5 for use in a Country may be found in its National Annex. The recommended value of q_4 is 20 kN/m and of q_5 is 70 kN.

(5) Internal ties should be connected to peripheral ties such that the transfer of forces is assured.



A - peripheral tie B - internal tie C - horizontal column or wall tie

Figure 9.22: Ties for Accidental Actions

9.10.2.4 Horizontal ties to columns and/or walls

(1) Edge columns and walls should be tied horizontally to the structure at each floor and roof level.

(2) The ties should be capable of resisting a tensile force $F_{tie,fac}$ per metre of the façade. For columns the force need not exceed $F_{tie,col}$.

Note: Values of $F_{tie,fac}$ and $F_{tie,col}$ for use in a Country may be found in its National Annex. The recommended values of F_{tie} are 20 kN and of $F_{tie,col}$ are 150 kN.

(3) Corner columns should be tied in two directions. Steel provided for the peripheral tie may be used as the horizontal tie in this case.

9.10.2.5 Vertical ties

(1) In panel buildings of 5 storeys or more, vertical ties should be provided in columns and/or walls to limit the damage of collapse of a floor in the case of accidental loss of the column or wall below. These ties should form part of a bridging system to span over the damaged area.

(2) Normally, continuous vertical ties should be provided from the lowest to the highest level, capable of carrying the load in the accidental design situation, acting on the floor above the column/wall accidentally lost. Other solutions e.g. based on the diaphragm action remaining wall elements and/or on membrane action in floors, may be used if equilibrium and sufficient deformation capacity can be verified.

(3) Where a column or wall is supported at its lowest level by an element other than a foundation (e.g. beam or flat slab) accidental loss of this element should be considered in the design and a suitable alternative load path should be provided.

9.10.3 Continuity and anchorage of ties

(1)P Ties in two horizontal directions shall be effectively continuous and anchored at the perimeter of the structure.

(2) Ties may be provided wholly within the insitu concrete topping or at connections. Where ties are not continuous in one plane, the bending effects resulting from the eccentricities should be considered.

(3) Ties should not normally be lapped in narrow joints between precast units. Mechanical anchorage should be used in these cases.

SECTION 10 ADDITIONAL RULES FOR PRECAST CONCRETE ELEMENTS AND STRUCTURES

10.1 General

(1)P The rules in this section apply to buildings made partly or entirely of precast concrete elements, and are supplementary to the rules in other sections. Additional matters related to detailing, production and assembly are covered by specific product standards.

Note: Headings are numbered 10 followed by the number of the corresponding main section. Headings of lower level are numbered consecutively, without connection to sub-headings in previous sections.

10.1.1 Special terms used in this section

Precast element: element manufactured in a factory or a place other than the final position in the structure, protected from adverse weather conditions

Precast product: precast element manufactured in compliance with a specific CEN standard

Composite element: element comprising in-situ and precast concrete with or without reinforcement connectors

Rib and block floors: consist of precast ribs (or beams) with an infill between them, made of blocks, hollow clay pots or other forms of permanent shuttering, with or without an in-situ topping

Diaphragms: plane members which are subjected to in-plane forces; may consist of several precast units connected together

Ties: in the context of precast structures, ties are tensile members, effectively continuous, placed in floors, walls or columns

Isolated precast members: members for which, in case of failure, no secondary means of load transfer is available

Transient situations in precast concrete construction include

- demoulding
- transport to the storage yard
- storage (support and load conditions)
- transport to site
- erection (hoisting)
- construction (assembly)

10.2 Basis of design, fundamental requirements

(1)P In design and detailing of precast concrete elements and structures, the following shall be considered specifically:

- transient situations (see 10.1.1)
- bearings; temporary and permanent
- connections and joints between elements

(2) Where relevant, dynamic effects in transient situations should be taken into account. In the absence of an accurate analysis, static effects may be multiplied by an appropriate factor (see

also product standards for specific types of precast elements).

(3) Where required, mechanical devices should be detailed in order to allow ease of assembly, inspection and replacement.

10.3 Materials

10.3.1 Concrete

10.3.1.1 Strength

(1) For precast elements in continuous production, subjected to an appropriate quality control system according to the product standards, with the concrete tensile strength tested, a statistical analysis of test results may be used as a basis for the evaluation of the tensile strength that is used for serviceability limit states verifications, as an alternative to Table 3.1.

(2) Intermediate strength classes within Table 3.1 may be used.

(3) In the case of heat curing of precast concrete elements, the compressive strength of concrete at an age t before 28 days, $f_{cm}(t)$, may be estimated from Expression (3.3) in which the concrete age t is substituted by the temperature adjusted concrete age obtained by Expression (B.10) of Annex B.

Note: The coefficient $\beta_{cc}(t)$ should be limited to 1.

For the effect of heat curing Expression (10.1) may be used:

$$f_{cm}(t)=f_{cmp}+\frac{f_{cm}-f_{cmp}}{\log(28-t_p+1)}\log(t-t_p+1) \qquad (10.1)$$

Where f_{cmp} is the mean compressive strength after the heat curing (i.e. at the release of the prestress), measured by testing of samples at the time t_p ($t_p < t$), that went through the same heat treatment with the precast elements.

10.3.1.2 Creep and shrinkage

(1) In the case of a heat curing of the precast concrete elements, it is permitted to estimate the values of creep deformations according to the maturity function, Expression (B.10) of Annex B.

(2) In order to calculate the creep deformations, the age of concrete at loading t_0 (in days) in Expression (B.5) should be replaced by the equivalent concrete age obtained by Expressions (B.9) and (B.10) of Annex B.

(3) In precast elements subjected to heat curing it may be assumed that:
a) the shrinkage strain is not significant during heat curing and
b) autogenous shrinkage strain is neglible.

10.3.2 Prestressing steel

10.3.2.2 Technological properties of prestressing steel

(1)P For pre-tensioned members, the effect on the relaxation losses of increasing the temperature while curing the concrete, shall be considered.

Note: The relaxation is accelerated during the application of a thermal curing when a thermal strain is introduced at the same time. Finally, the relaxation rate is reduced at the end of the treatment.

(2) An equivalent time t_{eq} should be added to the time after tensioning t in the relaxation time functions, given in 3.3.2(7), to cater for the effects of the heat treatment on the prestress loss due to the relaxation of the prestressing steel. The equivalent time can be estimated from Expression (10.3):

$$t_{eq} = \frac{1{,}14^{T_{max}-20}}{T_{max}-20} \sum_{i=1}^{n} \left(T_{(\Delta t_i)} - 20\right) \Delta t_i \qquad (10.3)$$

where

t_{eq} is the equivalent time (in hours)

$T_{(\Delta ti)}$ is the temperature (in °C) during the time interval Δt_i

T_{max} is the maximum temperature (in °C) during the heat treatment

10.5 Structural analysis, general provisions

10.5.1 General

(1)P The analysis shall account for:

- the behaviour of the structural units at all stages of construction using the appropriate geometry and properties for each stage, and their interaction with other elements (e.g. composite action with in-situ concrete, other precast units);
- the behaviour of the structural system influenced by the behaviour of the connections between elements, with particular regard to actual deformations and strength of connections;
- the uncertainties influencing restraints and force transmission between elements arising from deviations in geometry and in the positioning of units and bearings.

(2) Beneficial effects of horizontal restraint caused by friction due to the weight of any supported element may only be used in non seismic zones (using $\gamma_{G,inf}$) and where:

- the friction is not solely relied upon for overall stability of the structure;
- the bearing arrangements preclude the possibility of accumulation of irreversible sliding of the elements, such as caused by uneven behaviour under alternate actions (e.g. cyclic thermal effects on the contact edges of simply supported elements);
- the possibility of significant impact loading is eliminated

(3) The effects of horizontal movements should be considered in design with respect to the resistance of the structure and the integrity of the connections.

10.5.2 Losses of prestress

(1) In the case of heat curing of precast concrete elements, the lessening of the tension in the tendons and the restrained dilatation of the concrete due to the temperature, induce a specific

thermal loss ΔP_θ. This loss may be estimated by the Expression (10.4):

$$\Delta P_\theta = 0{,}5\, A_p\, E_p\, \alpha_c\, (T_{max} - T_o) \tag{10.4}$$

Where

- A_p is the cross-section of tendons
- E_p is the elasticity modulus of tendons
- α_c is the linear coefficient of thermal expansion for concrete (see 3.1.2)
- $T_{max} - T_0$ is the difference between the maximum and initial temperature in the concrete near the tendons, in °C

Note: Any loss of prestress, ΔP_θ, caused by elongation due to heat curing may be ignored if preheating of the tendons is applied.

10.9 Particular rules for design and detailing

10.9.1 Restraining moments in slabs

(1) Restraining moments may be resisted by top reinforcement placed in the topping or in plugs in open cores of hollow core units. In the former case the horizontal shear in the connection should be checked according to 6.2.5. In the latter case the transfer of force between the in situ concrete plug and the hollow core unit should be verified according to 6.2.5. The length of the top reinforcement should be in accordance with 9.2.1.3.

(2) Unintended restraining effects at the supports of simply supported slabs should be considered by special reinforcement and/or detailing.

10.9.2 Wall to floor connections

(1) In wall elements installed over floor slabs, reinforcement should normally be provided for possible eccentricities and concentrations of the vertical load at the end of the wall. For floor elements see 10.9.1 (2).

(2) No specific reinforcement is required provided the vertical load is $\leq 0{,}5h.f_{cd}$, where h is the wall thickness, see Figure 10.1. The load may be increased to $0{,}6h.f_{cd}$ with reinforcement according to Figure 10.1, having diameter $\phi \geq 6$ mm and spacing s not greater than the lesser of h and 200 mm. For higher loads, reinforcement should be designed according to (1).

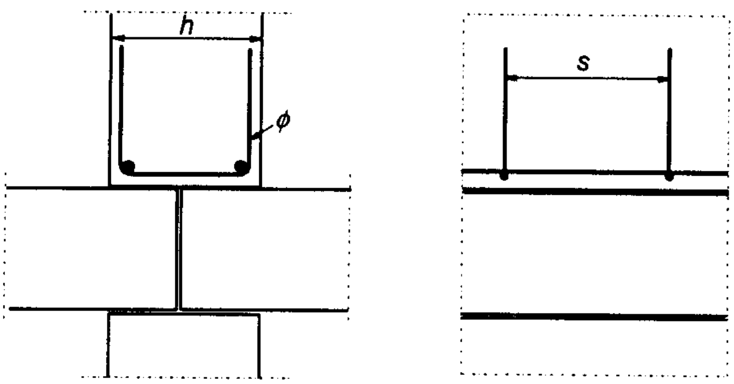


Figure 10.1: Example of reinforcement in a wall over a connection between two floor slabs.

10.9.3 Floor systems

(1)P The detailing of floor systems shall be consistent with assumptions in analysis and design. Relevant product standards shall be considered.

(2)P Where transverse load distribution between adjacent units has been taken into account, appropriate shear connection shall be provided.

(3)P The effects of possible restraints of precast units shall be considered, even if simple supports have been assumed in design.

(4) Shear transfer in connections may be achieved in different ways. Three main types shown in Figure 10.2.

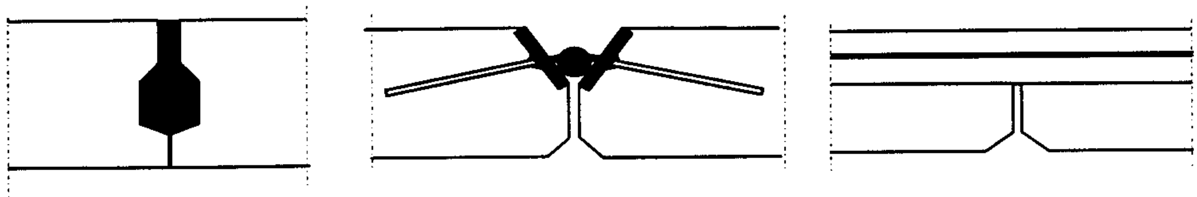
(5) Transverse distribution of loads should be based on analysis or tests, taking into account possible load variations between precast elements. The resulting shear force between floor units should be considered in the design of connections and adjacent parts of elements (e.g. outside ribs or webs).

For floors with uniformly distributed load, and in the absence of a more accurate analysis, this shear force per unit length may be taken as:

$$v_{Ed} = q_{Ed} \cdot b/3 \qquad (10.5)$$

where:

q_{Ed} is the design value of imposed load (kN/m^2)
b_e is the width of the element



a) concreted or grouted connections **b) welded or bolted connections (this shows *one* type of *welded* connection as an example)** **c) reinforced topping. (vertical reinforcement connectors to topping may be required to ensure shear transfer at ULS)**

Figure 10.2: Examples of connections for shear transfer

(6) Where precast floors are assumed to act as diaphragms to transfer horizontal loads to bracing units, the following should be considered:
- the diaphragm should form part of a realistic structural model, taking into account the deformation compatibility with bracing units,
- the effects of horizontal deformations should be taken into account for all parts of the structure involved in the transfer of horizontal loads,
- the diaphragm should be reinforced for the tensile forces assumed in the structural model,
- stress concentrations at openings and connections should be taken into account in the detailing of reinforcement.

(7) Transverse reinforcement for shear transfer across connections in the diaphragm may be concentrated along supports, forming ties consistent with the structural model. This reinforcement may be placed in the topping, if it exists.

(8) Precast units with a topping of at least 40 mm may be designed as composite members, if shear in the interface is verified according to 6.2.5. The precast unit should be checked at all stages of construction, before and after composite action has become effective.

(9) Transverse reinforcement for bending and other action effects may lie entirely within the topping. The detailing should be consistent with the structural model, e.g. if two-way spanning is assumed.

(10) Webs or ribs in isolated slab units (i.e. units which are not connected for shear transfer) should be provided with shear reinforcement as for beams.

(11) Floors with precast ribs and blocks without topping may be analysed as solid slabs, if the insitu transverse ribs are provided with continuous reinforcement through the precast longitudinal ribs and at a spacing s_T according to Table 10.1.

(12) In diaphragm action between precast slab elements with concreted or grouted connections, the average longitudinal shear stress v_{Rdi} should be limited to 0,1 MPa for very smooth surfaces, and to 0,15 MPa for smooth and rough surfaces. Seed 6.2.5 for definition of surfaces.

Table 10.1: Maximum spacing of transverse ribs, s_T for the analysis of floors with ribs and block as solid slabs. s_L = spacing of longitudinal ribs, l_L = length (span) of longitudinal ribs, h = thickness of ribbed floor

Type of imposed loading	$s_L \leq l_L/8$	$s_L > l_L/8$
Residential load, snow load	not required	$s_T \leq 12\ h$
Other loads	$s_T \leq 10\ h$	$s_T \leq\ 8\ h$

10.9.4 Connections and supports for precast elements

10.9.4.1 Materials

(1)P Materials used for connections shall be:
- stable and durable for the lifetime of the structure
- chemically and physically compatible
- protected against adverse chemical and physical influences
- fire resistant to match the fire resistance of the structure.

(2)P Supporting pads shall have strength and deformation properties in accordance with the design assumptions.

(3)P Metal fastenings for claddings, other than in environmental classes X0 and XC1 (Table 4.1) and not protected against the environment, shall be of corrosion resistant material. If inspection is possible, coated material may also be used.

(4)P Before undertaking welding, annealing or cold forming the suitability of the material shall be verified.

10.9.4.2 General rules for design and detailing of connections

(1)P Connections shall be able to resist action effects consistent with design assumptions, to accommodate the necessary deformations and ensure robust behaviour of the structure.

(2)P Premature splitting or spalling of concrete at the ends of elements shall be prevented, taking into account

- relative movements between elements
- tolerances
- assembly requirements
- ease of execution
- ease of inspection

(3) Verification of resistance and stiffness of connections may be based on analysis, possibly assisted by testing (for design assisted testing, see EN 1990, Annex D). Imperfections should be taken into account. Design values based on tests should allow for unfavourable deviations from testing conditions.

10.9.4.3 Connections transmitting compressive forces

(1) Shear forces may be ignored in compression connections if they are less than 10% of the compressive force.

(2) For connections with bedding materials like mortar, concrete or polymers, relative movement between the connected surfaces should be prevented during hardening of the material.

(3) Connections without bedding material (dry connections) should only be used where an appropriate quality of workmanship can be achieved. The average bearing stress between plane surfaces should not exceed 0,3 f_{cd}. Dry connections including curved (convex) surfaces should be designed with due consideration of the geometry.

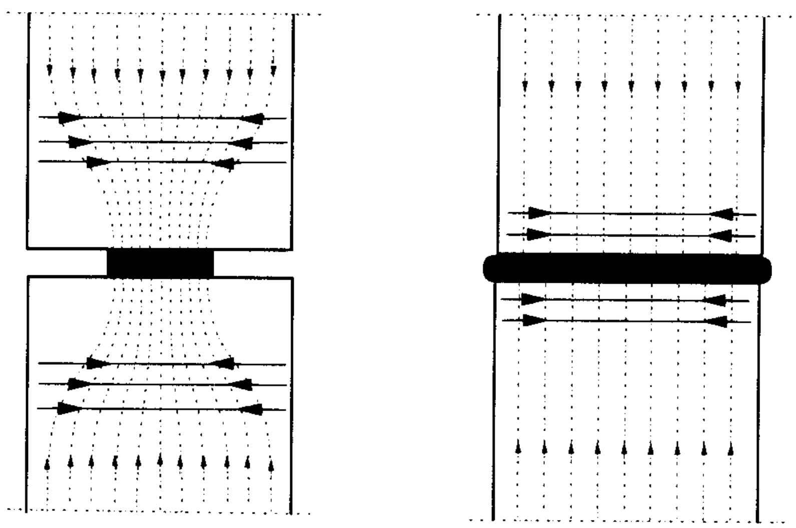
(4) Transverse tensile stresses in adjacent elements should be considered. They may be due to concentrated compression according to Figure 10.3a, or to the expansion of soft padding according to Figure 10.3b. Reinforcement in case a) may be designed and located according to 6.5. Reinforcement in case b) should be placed close to the surfaces of the adjacent elements.

(5) In the absence of more accurate models, reinforcement in case b) may be calculated in accordance with Expression (10.6):

$$A_s = 0,25\ (t\ /\ h)\ F_{Ed}\ /\ f_{yd} \qquad (10.6)$$

where:
A_s is the reinforcement area in each surface
t is the thickness of padding
h is the dimension of padding in direction of reinforcement
F_{Ed} is the compressive force in connection.

(6) The maximum capacity of compression connections can be determined according to 6.7, or can be based on analysis, possibly assisted by testing (for design assisted testing, see EN 1990).



a) Concentrated bearing **b) Expansion of soft padding**

Figure 10.3: Transverse tensile stresses at compression connections.

10.9.4.4 Connections transmitting shear forces

(1) For shear transfer in interfaces between two concretes, e.g. a precast element and in situ concrete, see 6.2.5.

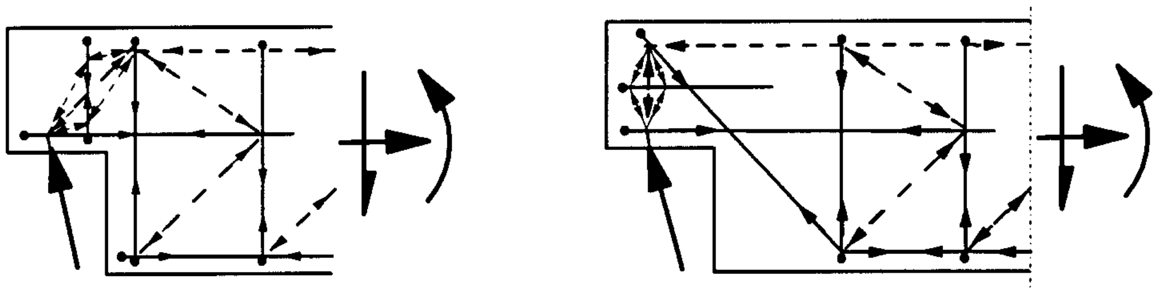
10.9.4.5 Connections transmitting bending moments or tensile forces

(1)P Reinforcement shall be continuous across the connection and anchored in the adjacent elements.

(2) Continuity may be obtained by, for example
- lapping of bars
- grouting of reinforcement into holes
- overlapping reinforcement loops
- welding of bars or steel plates
- prestressing
- mechanical devices (threaded or filled sleeves)
- swaged connectors (compressed sleeves)

10.9.4.6 Half joints

(1) Half joints may be designed using strut-and-tie models according to 6.5. Two alternative models and reinforcements are indicated in Figure 10.4. The two models may be combined.



Note: The figure shows only the main features of strut-and-tie models.

Figure 10.4: Indicative models for reinforcement in half joints.

10.9.4.7 Anchorage of reinforcement at supports

(1) Reinforcement in supporting and supported members should be detailed to ensure anchorage in the respective node, allowing for tolerances. An example is shown in Figure 10.5.

The effective bearing length a_1 is controlled by a distance d (see Figure 10.5) from the edge of the respective elements where, for the suporting element:

$d = c + \Delta a_2$ with horizontal loops or otherwise end anchored bars
$d = c + \Delta a_2 + r$ with vertically bent bars

Here c is concrete cover and Δa is tolerance Δa_2 or Δa_3 according to Figure 10.5 (see also definitions in 10.9.5.2 (1)).

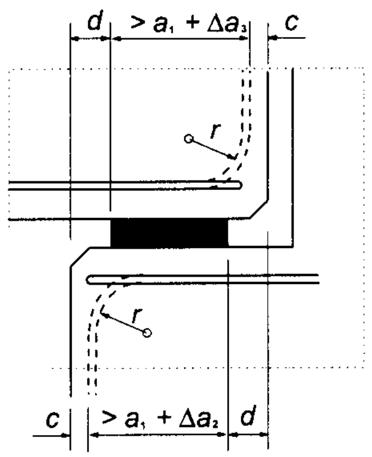


Figure 10.5: Example of detailing of reinforcement in support

10.9.5 Bearings

10.9.5.1 General

(1)P The proper functioning of bearings shall be ensured by reinforcement in adjacent members, limitation of bearing stress and measures to account for movement or restraint.

(2)P For bearings which do not permit sliding or rotation without significant restraint, actions due to creep, shrinkage, temperature, misalignment, lack of plumb etc. shall be taken into account in the design of adjacent members.

(3) The effects of (2)P may require transverse reinforcement in supporting and supported members, and/or continuity reinforcement for tying elements together. They may also influence the design of main reinforcement in such members.

(4)P Bearings shall be designed and detailed to ensure correct positioning, taking into account production and assembling tolerances.

(5)P Possible effects of prestressing anchorages and their recesses shall be taken into account.

10.9.5.2 Bearings for connected members

(1) The nominal length a of a simple bearing as shown in Figure 10.6 may be calculated as:

$$a = a_1 + a_2 + a_3 + \sqrt{\Delta a_2^2 + \Delta a_3^2} \qquad (10.7)$$

where:

a_1 is the net bearing length with regard to bearing stress, $a_1 = F_{Ed} / (b_1 f_{Rd})$, but not less than minimum values in Table 10.2
F_{Ed} is the design value of support reaction
b_1 is the net bearing width, see (3)
f_{Rd} is the design value of bearing strength, see (2)
a_2 is the distance assumed ineffective beyond outer end of supporting member, see Figure 10.6 and Table 10.3
a_3 is the similar distance for supported member, see Figure 10.6 and Table 10.4
Δa_2 is an allowance for tolerances for the distance between supporting members, see Table 10.5
Δa_3 is an allowance for tolerances for the length of the supported member, $\Delta a_3 = l_n/2500$, l_n is length of member.

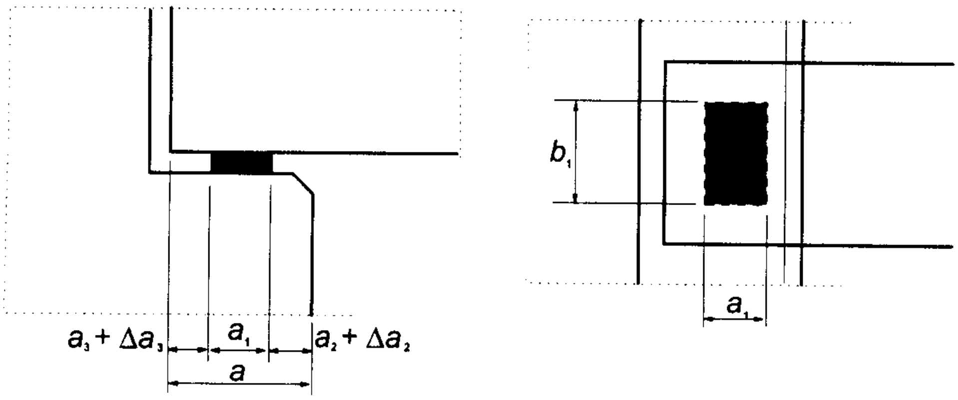


Figure 10.6: Example of bearing with definitions.

Table 10.2: Minimum value of a_1 in mm

Relative bearing stress, σ_{Ed} / f_{cd}	≤ 0,15	0,15 - 0,4	> 0,4
Line supports (floors, roofs)	25	30	40
Ribbed floors and purlins	55	70	80
Concentrated supports (beams)	90	110	140

Table 10.3: Distance a_2 (mm) assumed ineffective from outer end of supporting member. Concrete padstone should be used in cases (-)

Support material and type	σ_{Ed} / f_{cd}	≤ 0,15	0,15 - 0,4	> 0,4
Steel	line	0	0	10
	concentrated	5	10	15
Reinforced concrete ≥ C30	line	5	10	15
	concentrated	10	15	25
Plain concrete and rein. concrete < C30	line	10	15	25
	concentrated	20	25	35
Brickwork	line	10	15	(-)
	concentrated	20	25	(-)

Table 10.4: Distance a_3 (mm) assumed ineffective beyond outer end of supported member

Detailing of reinforcement	Support	
	Line	Concentrated
Continuous bars over support (restrained or not)	0	0
Straight bars, horizontal loops, close to end of member	5	15, but not less than end cover
Tendons or straight bars exposed at end of member	5	15
Vertical loop reinforcement	15	end cover + inner radius of bending

Table 10.5: Allowance Δa_2 for tolerances for the clear distance between the faces of the supports. l = span length

Support material	Δa_2
Steel or precast concrete	$10 \leq l/1200 \leq 30$ mm
Brickwork or cast in-situ concrete	$15 \leq l/1200 + 5 \leq 40$ mm

(2) In the absence of other specifications, the following values can be used for the bearing strength:

$f_{Rd} = 0,4\ f_{cd}$ for dry connections (see 10.9.4.3 (3) for definition)

$f_{Rd} = f_{bed} \leq 0,85\ f_{cd}$ for all other cases

where
f_{cd} is the the lowest design strength of supported and supporting member
f_{bed} is the design strength of bedding material

(3) If measures are taken to obtain a uniform distribution of the bearing pressure, e.g. with mortar, neoprene or similar pads, the design bearing width b_1 may be taken as the actual width of the bearing. Otherwise, and in the absence of a more accurate analysis, b_1 should not be greater than to 600 mm.

10.9.5.3 Bearings for isolated members

(1)P The nominal length shall be 20 mm greater than for non-isolated members.

(2)P If the bearing allows movements in the support, the net bearing length shall be increased to cover possible movements.

(3)P If a member is tied other than at the level of its bearing, the net bearing length a_1 shall be increased to cover the effect of possible rotation around the tie.

10.9.6 Pocket foundations

10.9.6.1 General

(1)P Concrete pockets shall be capable of transferring vertical actions, bending moments and horizontal shears from columns to the soil. The pocket shall be large enough to enable a good concrete filling below and around the column.

10.9.6.2 Pockets with keyed surfaces

(1) Pockets expressly wrought with indentations or keys may be considered to act monolithically with the column.

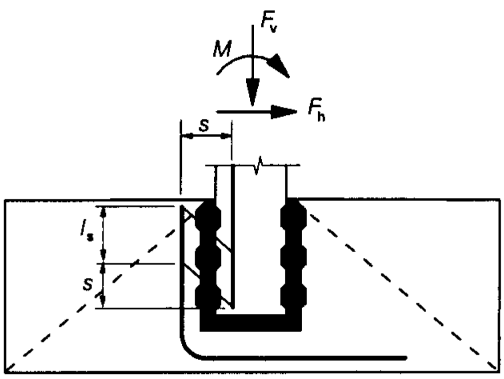
(2) Where vertical tension due to moment transfer occurs careful detailing of the overlap reinforcement of the column and the foundation is needed, allowing for the separation of the lapped bars. The lap length according to 8.6 should be increased by at least the horizontal distance between bars in the column and in the foundation (see Figure 10.7 (a)) Adequate horizontal reinforcement for the lapped splice should be provided.

(3) The punching shear design should be as for monolithic column/foundation connections according to 6.4, as shown in Figure 10.7 (a), provided the shear transfer between the column and footing is verified. Otherwise the punching shear design should be as for pockets with smooth surfaces.

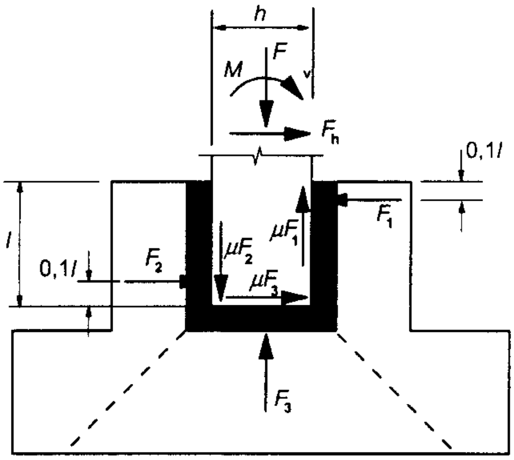
10.9.6.3 Pockets with smooth surfaces

(1) The forces and the moment may be assumed to be transferred from column to foundation by compressive forces F_1, F_2 and F_3 through the concrete filling and corresponding friction

forces, as shown in Figure 10.7 (b). This model requires
$l \geq 1{,}2\ h$.



(a) with keyed joint surface



(b) with smooth joint surface

Figure 10.7: Pocket Foundations

(2) The coefficient of friction should not be taken greater than $\mu = 0{,}3$.

(3) Special attention should be paid to:
- detailing of reinforcement for F_1 in top of pocket walls
- transfer of F_1 along the lateral walls to the footing
- anchorage of main reinforcement in the column and pocket walls
- shear resistance of column within the pocket
- punching resistance of the footing slab under the column force, the calculation for which may take into account the insitu structural concrete placed under the precast element.

10.9.7 Tying systems

(1) For plate elements loaded in their own plane, e.g. in walls and floor diaphragms, the necessary interaction may be obtained by tying the structure together with peripheral and/or internal ties.

The same ties may also act to prevent progressive collapse according to 9.9.

SECTION 11 LIGHTWEIGHT AGGREGATE CONCRETE STRUCTURES

11.1 General

(1)P This section provides additional requirements for lightweight aggregate concrete. Reference is made to the other Sections (1 to 10 and 12) of this document and the Annexes.

Note. Headings are numbered 11 followed by the number of the corresponding main section. Headings of lower level are numbered consecutively, without connection to sub-headings in previous sections. If alternatives are given for Expressions, Figures or Tables in the other sections, the original reference numbers are also prefixed by 11.

11.1.1 Scope

(1)P All clauses of the Sections 1 to 10 and 12 are generally applicable, unless they are substituted by special clauses given in this section. In general, where strength values originating from Table 3.1 are used in Expressions, those values have to be replaced by the corresponding values for lightweight concrete, given in this section in Table 11.3.1

(2)P Section 10 applies to all concretes with closed structure made with natural or artificial mineral lightweight aggregates, unless reliable experience indicates that provisions different from those given can be adopted safely.

(3) This section does not apply to aerated concrete either autoclaved or normally cured nor lightweight aggregate concrete with an open structure.

(4)P Lightweight aggregate concrete is concrete having a closed structure and an oven-dry density of not more than 2200 kg/m^3 consisting of or containing a proportion of artificial or natural lightweight aggregates having a particle density of less than 2000 kg/m^3

11.1.2 Special symbols

1(P) The following symbols are used specially for lightweight concrete:

LC the strength classes of lightweight aggregate concrete are preceded by the symbol LC
η_E is a conversion factor for calculating the modulus of elasticity
η_1 is a coefficient for determining tensile strength
η_2 is a coefficient for determining creep coefficient
η_3 is a coefficient for determining drying shrinkage
ρ is the oven-dry density of lightweight aggregate concrete in kg/m^3

For the mechanical properties an additional subscript *l* (lightweight) is used.

11.2 Basis of design

1(P) Section 2 is valid for lightweight concrete without modifications.

11.3 Materials

11.3.1 Concrete

(1)P In EN 206-1 lightweight aggregate is classified according to its density as shown in Table 11.1. In addition this table gives corresponding densities for plain and reinforced concrete with normal percentages of reinforcement which may be used for design purposes in calculating self-weight or imposed permanent loading. Alternatively, the density may be specified as a target value.

(2) Alternatively the contribution of the reinforcement to the density may be determined by calculation.

Table 11.1: Density classes and corresponding design densities of LWAC according to EN 206-1

Density class		**1,0**	**1,2**	**1,4**	**1,6**	**1,8**	**2,0**
Oven-dry density ρ (kg/m^3)		801-1000	1001-1200	1201-1400	1401-1600	1601-1800	1801-2000
Density (kg/m^3)	Plain concrete	1050	1250	1450	1650	1850	2050
	Reinforced concrete	1150	1350	1550	1750	1950	2150

(3) The tensile strength of lightweight aggregate concrete may be obtained by multiplying the f_{ct} values given in Table 3.1 by a coefficient:

$$\eta_1 = 0{,}40 + 0{,}60\rho/2200 \qquad (11.1)$$

where
ρ is the upper limit of the oven-dry density in accordance with Table 11.1

11.3.2 Elastic deformation

(1) An estimate of the mean values of the secant modulus E_{lcm} for LWAC may be obtained by multiplying the values in Table 3.1, for normal density concrete, by the following coefficient:

$$\eta_E = (\rho/2200)^2 \qquad (11.2)$$

where ρ denotes the oven-dry density in EN 206 Section 4 (see Table 11.1).

Where accurate data are needed, e.g. where deflections are of great importance, tests should be carried out in order to determine the E_{lcm} values in accordance with ISO 6784.

(2)P The coefficient of thermal expansion of LWAC depends mainly on the type of aggregate used and varies over a wide range between about $4 \cdot 10^{-6}$ and $14 \cdot 10^{-6}$/K

For design purposes where thermal expansion is of no great importance, the coefficient of thermal expansion may be taken as $8 \cdot 10^{-6}$/K.

The differences between the coefficients of thermal expansion of steel and lightweight aggregate concrete need not be considered in design.

Table 11.3.1: Stress and deformation characteristics for lightweight concrete

Strength classes for light weight concrete														Analytical relation/Explanation
f_{lck} (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	
$f_{lck,cube}$ (MPa)	13	18	22	28	33	38	44	50	55	60	66	77	88	
f_{lcm} (MPa)	17	22	28	33	38	43	48	53	58	63	68	78	88	$f_{lcm} = f_{lck} + 8$(MPa)
f_{lctm} (MPa)	$f_{lctm} = f_{ctm} \cdot \eta_1$													η_1=0,40+0,60ρ/2200
$f_{lctk,0,05}$ (MPa)	$f_{lctk,0,05} = f_{ctk,0,05} \cdot \eta_1$													5% - fractile
$f_{lctk,0,95}$ (MPa)	$f_{lctk,0,95} = f_{ctk,0,95} \cdot \eta_1$													95% - fractile
E_{lcm} (GPa)	$E_{lcm} = E_{cm} \cdot \eta_E$													$\eta_E = (\rho/2200)^2$
ε_{lc1} (‰)	$kf_{lcm}/(E_{lci} \cdot \eta_E)$ $\begin{cases} k = 1{,}1 \text{ for sanded lightweight aggregate concrete} \\ k = 1.0 \text{ for all lightweight aggregate concrete} \end{cases}$													see Figure 3.2
ε_{lcu1}(‰)	ε_{lc1}													see Figure 3.2
ε_{lc2} (‰)	2,0									2,2	2,3	2,4	2,5	see Figure 3.3
ε_{lcu2} (‰)	3,5 η_1									3,1η_1	2,9η_1	2,7η_1	2,6η_1	see Figure 3.3 $\lvert\varepsilon_{lcu2u}\rvert \geq \lvert\varepsilon_{lc2}\rvert$
n	2,0									1,75	1,6	1,45	1,4	
ε_{lc3}(‰)	1,75									1,8	1,9	2,0	2,2	see Figure 3.4
ε_{lcu3}(‰)	3,5 η_1									3,1η_1	2.9η_1	2.7η_1	2,6η_1	see Figure 3.4 $\lvert\varepsilon_{lcu3}\rvert \geq \lvert\varepsilon_{lc3}\rvert$

11.3.3 Creep and shrinkage

(1) For lightweight aggregate concrete the creep coefficient φ may be assumed equal to the value of normal density concrete multiplied by a factor $(\rho/2200)^2$.

The creep strains so derived should be multiplied by a factor η_2:

η_2 = 1,3 for $f_{lck} \leq$ LC16/20
= 1,0 for $f_{lck} \geq$ LC20/25

(2) The final drying shrinkage values for lightweight concrete can be obtained by multiplying the values for normal density concrete in Table 3.2 by a factor η_3:

η_3 = 1,5 for $f_{lck} \leq$ LC16/20
= 1,2 for $f_{lck} \geq$ LC20/25

(3) The Expressions (3.11), (3.12) and (3.13), which provide information for autogenous shrinkage, give maximum values for lightweight aggregate concretes, where no supply of water from the aggregate to the drying microstructure is possible. If water-saturated, or even partially saturated lightweight aggregate is used, the autogenous shrinkage values will be considerably reduced.

11.3.4 Stress-strain relations for non-linear structural analysis

(1) For lightweight aggregate concrete the values ε_{c1} and ε_{cu1} given in Figure 3.2 should be substituted by ε_{lc1} and ε_{lcu1} given in Table 11.3.1.

11.3.5 Design compressive and tensile strengths

(1)P The value of the design compressive strength is defined as:

$$f_{lcd} = \alpha_{lcc}\, f_{lck} / \gamma_c \qquad (11.3.15)$$

where γ_c is the partial safety factor for concrete, see 2.4.1.4, and α_{lcc} is a coefficient according to 3.1.6 (1)P.

Note: The value of α_{lcc} for use in a Country may be found in its National Annex. The recommended value is 0,85.

(2)P The value of the design tensile strength is defined as

$$f_{lctd} = \alpha_{lct}\, f_{lctk} / \gamma_c \qquad (11.3.16)$$

where γ_c is the partial safety factor for concrete, see 2.4.1.4 and α_{lct} is a coefficient according to 3.1.6 (2)P.

Note: The value of α_{lct} for use in a Country may be found in its National Annex. The recommended value is 0,85.

11.3.6 Stress-strain relations for the design of sections

(1) For lightweight aggregate concrete the values ε_{c2} and ε_{cu2} given in Figure 3.3 should be replaced with the values of ε_{lc2} and ε_{lcu2} given in Table 11.3.1.

(2) For lightweight aggregate concrete the values ε_{c3} and ε_{cu3} given in Figure 3.4 should be replaced with the values of ε_{lc3} and ε_{lcu3} given in Table 11.3.1.

11.3.7 Confined concrete

(1) If more precise data are not available, the stress-strain relation shown in Figure 3.6 may be used, with increased characteristic strength and strains according to:

$$f_{lck,c} = f_{lck}\,(1{,}0 + k\sigma_2/f_{lck}) \qquad (11.3.24)$$

where k = 1,1 for lightweight aggregate concrete with sand as the fine aggregate
k = 1,0 for lightweight aggregate (both fine and coarse aggregate) concrete

$$\varepsilon_{lc2,c} = \varepsilon_{lc2}\,(f_{lck,c}/f_{lck})^2 \qquad (11.3.26)$$
$$\varepsilon_{lcu2,c} = \varepsilon_{lcu2} + 0{,}2\sigma_2/f_{lck} \qquad (11.3.27)$$

where ε_{lc2} and ε_{lcu2} follow from Table 11.3.1.

11.4 Durability and cover to reinforcement

11.4.1 Environmental conditions

(1) For lightweight aggregate concrete in Table 4.1 the same indicative exposure classes can be used as for normal density concrete.

11.4.2 Concrete cover and properties of concrete

(1)P For lightweight aggregate concrete the values of minimum concrete cover given in Table 4.2 shall be increased by 5 mm.

11.5 Structural analysis

11.5.1 Rotational capacity

Note: For light weight concrete the value of θ_{plast}, as shown in Figure 5.6N, should be multiplied by a factor $\varepsilon_{lc2u}/\varepsilon_{c2u}$.

11.6 Ultimate limit states

11.6.1 Members not requiring design shear reinforcement

(1) The design value of the shear resistance of a lightweight concrete member without shear reinforcement $V_{lRd,ct}$ follows from:

$$V_{lRd,ct} = [C_{lRd,ct}\eta_1 k(100\rho_l\, f_{lck})^{1/3} + 0{,}15\,\sigma_{cp}]\,b_w d \;\geq (0{,}15 v_{l,min}\eta_1\sigma_{cp})/b_w d \qquad (11.6.2)$$

where η_1 is defined in Expression (11.1), f_{lck} is taken from Table 11.3.1 and σ_{cp} is the mean compressive stress in the section due to axial force and prestress.

Note: the values of $C_{lRd,c}$ and v_{lmin} for use in a Country may be found in its National Annex. The recommended value for $C_{lRd,c}$ is $0{,}15/\gamma_c$ and that for $v_{l,min}$ is $0{,}02\,k^{3/2} f_{lck}^{\,1/2}$.

Table 11.6.1N: Values of $v_{l,min}$ for given values of d and f_{ck}

d (mm)	$v_{l,min}$ (MPa)						
	f_{ck} (MPa)						
	20	30	40	50	60	70	80
200	0.36	0.44	0.50	0.56	0.61	0.65	0.70
400	0.29	0.35	0.39	0.44	0.48	0.52	0.55
600	0.25	0.31	0.35	0.39	0.42	0.46	0.49
800	0.40	0.28	0.32	0.36	0.39	0.42	0.45
≥ 1000	0.22	0.27	0.31	0.34	0.37	0.40	0.43

(2) At a distance $0{,}5d \leq x < 2{,}0d$ from the edge of a support the shear capacity may be increased to:

$$V_{lRd,ct} = [C_{lRd,ct}\ \eta_1 k\ (100\rho_l\ f_{lck})^{1/3}\ (\frac{2d}{x}) + 0{,}15\ \sigma_{cp}]b_w d \leq 0{,}5\nu\, f_{lcd}\ b_w d \qquad (11.6.5)$$

where
η_1 is defined in Expression (11.1), f_{lck} is taken from Table 11.3.1 and ν follows from Expression (11.6.5)
$C_{lRd,c}$ see 11.6.1 (1)

11.6.2 Members requiring design shear reinforcement

(1) The reduction factor ν for the crushing resistance of the concrete struts is

$$\nu = 0{,}5\eta_1\ (1 - f_{lck}/250) \qquad (11.6.6)$$

11.6.3 Torsion

11.6.3.1 Design procedure

(1) In Expression (6.30) for lightweight concrete ν is defined according to Expression (11.6.5)

11.6.4 Punching

11.6.4.1 Slabs or column bases without punching shear reinforcement

(1) The punching shear resistance per unit area of a lightweight concrete slab follows from

$$V_{Rd,c} = C_{lRd,c}\ k\ \eta_1(100\rho_l\, f_{lck}\,)^{1/3} + 0{,}08\sigma_{cp} \ \geq (\eta_1 v_{lmin} + 0{,}08\sigma_{cp}) \qquad (11.6.49a)$$

where
η_1 is defined in Expression (11.1)
$C_{lRd,c}$ see 11.6.1 (1)
v_{lmin} see 11.6.1 (1)

(2) The punching shear resistance of lightweight concrete column bases follows from

$$V_{lRd} = C_{lRd,c}\ \eta_1 k\ (100\rho_l\ f_{lck})^{1/3}\ 2d/a \quad \geq \eta_1\ v_{lmin}\cdot 2d/a \qquad (11.6.49b)$$

where

η_1 is defined in Expression (11.1)

$\rho_1 \geq 0{,}005$

$C_{lRd,c}$ see 11.6.1 (1)

v_{lmin} see 11.6.1 (1)

11.6.4.2 Slabs or column bases with punching shear reinforcement

(1) Where shear reinforcement is required the punching shear resistance is given by

$$v_{lRd,cs} = 0{,}75 v_{lRd,c} + 1{,}5\left(\frac{d}{s_r}\right)\left(\frac{1}{u_1 d}\right) A_{sw} f_{ywd,eff} \sin\alpha \tag{11.6.54}$$

where $v_{lRd,c}$ is defined in Expression (11.6.48)

(2) Adjacent to the column the punching shear capacity is limited to a maximum of

$$v_{Ed} = \frac{V_{Ed}}{u_0 d} \leq v_{lRd,max} = 0{,}5\,\nu f_{lcd} \tag{11.6.55}$$

where ν is defined in Expression (11.6.5)

11.6.5 Partially loaded areas

(1) For a uniform distribution of load on an area A_{c0} (see Figure 6.29) the concentrated resistance force may be determined as follows:

$$F_{Rdu} = A_{c0} \cdot f_{lcd} \cdot [A_{c1} / A_{c0}]^{\frac{\rho}{4400}} \leq 3{,}0 \cdot f_{lcd} \cdot A_{c0}\left(\frac{\rho}{2200}\right) \tag{11.6.65}$$

11.7 Serviceability limit states

(1)P The basic ratios of span/effective depth for reinforced concrete members without axial compression, given in Table 7.4, should be reduced by a factor $\eta_E^{0.15}$ when applied to LWAC.

11.8 Detailing of reinforcement - General

11.8.1 Permissible mandrel diameters for bent bars

(1) For normal density concrete the mandrel size should be restricted to the values given in Table 8.2, in order to avoid splitting of the concrete behind bends, hooks and loops. For lightweight aggregate concrete those values should be increased by 50%.

11.8.2 Ultimate bond stress

(1) The design value of the ultimate bond stress for bars in lightweight concrete may be calculated using Expression 8.2, by substituting the value f_{lctd} for f_{ctd}, with $f_{lctd} = f_{lctk,0,05}/\gamma_c$. The values for $f_{lctk,0,05}$ are found in Table 11.3.1.

11.9 Detailing of members and particular rules

(1) The diameter of bars embedded in LWAC should not normally exceed 32 mm. For LWAC bundles of bars should not consist of more than two bars and the equivalent diameter should not be exceed 45 mm.

11.12 Plain and lightly reinforced concrete structures

(1)P This section shall be applied to lightweight aggregate concrete without modifications.

SECTION 12 PLAIN AND LIGHTLY REINFORCED CONCRETE STRUCTURES

12.1 General

(1)P This section provides additional rules for plain concrete structures or where the reinforcement provided is less than the minimum required for reinforced concrete.

Note: Headings are numbered 12 followed by the number of the corresponding main section. Headings of lower level are numbered consecutively, without reference to subheadings in previous sections.

(2) This section applies to members, for which the effect of dynamic actions may be ignored. It does not apply to the effects such as those from rotating machines and traffic loads. Examples of such members include:

- members mainly subjected to compression other than that due to prestressing, e.g. walls, columns, arches, vaults, and tunnels;
- strip and pad footings for foundations;
- retaining walls;
- piles whose diameter is ≥ 600 mm and where $N_{Ed}/A_c \leq 0{,}3f_{ck}$.

(3) Where members are made with lightweight aggregate concrete with closed structure according to Section 11 or for precast concrete elements and structures covered by this Eurocode, the design rules should be modified accordingly.

(4) Members using plain concrete do not preclude the provision of steel reinforcement needed to satisfy serviceability and/or durability requirements, nor reinforcement in certain parts of the members. This reinforcement may be taken into account for the verification of local ultimate limit states as well as for the checks of the serviceability limit states.

12.2 Basis of design

12.2.1 Strength

(1) Due to the less ductile properties of plain concrete the values for $\alpha_{cc,pl}$ and $\alpha_{ct,pl}$ should be taken to be less than α_{cc} and α_{ct} for reinforced concrete.

Note: The values of $\alpha_{cc,pl}$ and $\alpha_{ct,pl}$ for use in a Country may be found in its National Annex. The recommended value for both is 0,8.

12.3 Materials

12.3.1 Concrete: additional design assumptions

(1) Due to the less ductile properties of plain concrete the values for $\alpha_{cc,pl}$ and $\alpha_{ct,pl}$ should be taken to be less than α_{cc} and α_{ct} for reinforced concrete.

Note: The values of $\alpha_{cc,pl}$ and $\alpha_{ct,pl}$ for use in a Country may be found in its National Annex. The recommended value for both is 0,8.

(2) When tensile stresses are considered for the design resistance of plain concrete members, the stress strain diagram (see 3.1.7) may be extended up to the tensile design strength using Expression (3.16) or a linear relationship.

$$f_{ctd} = \alpha_{ct}\, f_{ctk,0,05}/\gamma_c \qquad (12.1)$$

(3) Fracture mechanic methods may be used provided it can be shown that they lead to the required level of safety.

12.5 Structural analysis: ultimate limit states

(1) Since plain concrete members have limited ductility, linear analysis with redistribution or a plastic approach to analysis, e.g. methods without an explicit check of the deformation capacity, should not be used unless their application can be justified.

(2) Structural analysis may be based on the non-linear or the linear elastic theory. In the case of a non-linear analysis (e.g. fracture mechanics) a check of the deformation capacity should be carried out.

12.6 Ultimate limit states

12.6.1 Design resistance to bending and axial force

(1) In the case of walls, subject to the provision of adequate construction details and curing, the imposed deformations due to temperature or shrinkage may be ignored.

(2) The stress-strain relations for plain concrete should be taken from 3.1.7.

(3) The axial resistance, N_{Rd}, of a rectangular cross-section with a uniaxial eccentricity, e, in the direction of h_w, may be taken as:

$$N_{Rd} = \eta f_{cd} \times b \times h_w \times (1-2e/h_w) \qquad (12.2)$$

where:

ηf_{cd} is the design effective compressive strength (see 3.1.7 (3)

b is the overall width of the cross-section (see Figure 12.1)

h_w is the overall depth of the cross-section

e is the eccentricity of N_{Ed} in the direction h_w.

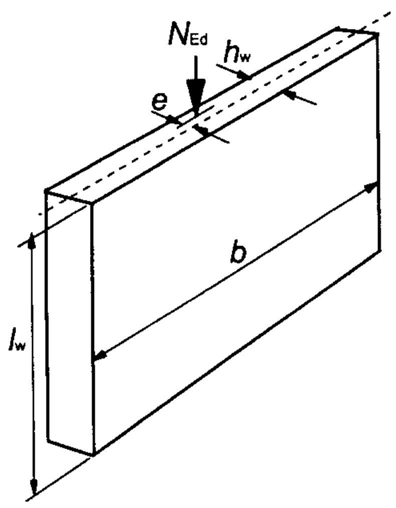


Figure 12.1: Notation for plain walls

Other simplified methods may be used provided that they are not less conservative than a rigorous method using a stress-strain relationship given in 3.1.7.

12.6.2 Local failure

(1)P Unless measures to avoid local tensile failure of the cross-section have been taken, the maximum eccentricity of the axial force N_{Ed} in a cross-section shall be limited to avoid large cracks.

12.6.3 Shear

(1) In plain concrete members account may be taken of the concrete tensile strength in the ultimate limit state for shear, provided that either by calculations or by experience brittle failure can be excluded and adequate resistance can be ensured.

(2) For a section subject to a shear force V_{Ed} and a normal force N_{Ed} acting over a compressive area A_{cc} the the absolute value of the components of design stress should be taken as:

$$\sigma_{cp} = N_{Ed} / A_{cc} \quad (12.3)$$

$$\tau_{cp} = kV_{Ed} / A_{cc} \quad (12.4)$$

Note: the value of k for use in a Country may be found in its National Annex. The recommended value is 1,5.

and the following should be checked:

$$\tau_{cp} \leq f_{cvd}$$

where:

if $\sigma_{cp} \leq \sigma_{c,lim}$

$$f_{cvd} = \sqrt{f_{ctd}^2 + \sigma_{cp} f_{ctd}} \quad (12.5)$$

or

if $\sigma_{cp} > \sigma_{c,lim}$

$$f_{cvd} = \sqrt{f_{ctd}^2 + \sigma_{cp} f_{ctd} - \left(\frac{\sigma_{cp} - \sigma_{c,lim}}{2}\right)^2} \quad (12.6)$$

$$\sigma_{c,lim} = f_{cd} - 2\sqrt{f_{ctd}\left(f_{ctd} + f_{cd}\right)} \quad (12.7)$$

where:

f_{cvd} is the concrete design strength in shear and compression
f_{cd} is the concrete design strength in compression
f_{ctd} is concrete design strength in tension

(3) A concrete member may be considered to be uncracked in the ultimate limit state if either it remains completely under compression or if the absolute value of the principal concrete tensile stress σ_{ct1} does not exceed f_{ctd}.

12.6.4 Torsion

(1) Cracked members should not normally be designed to resist torsional moments unless it can be justified otherwise.

12.6.5 Ultimate limit states induced by structural deformation (buckling)

12.6.5.1 Slenderness of columns and walls

(1) The slenderness of a column or wall is given by

$$\lambda = l_0/i \tag{12.8}$$

where:

i is the minimum radius of gyration

l_0 is the effective length of the member which can be assumed to be:

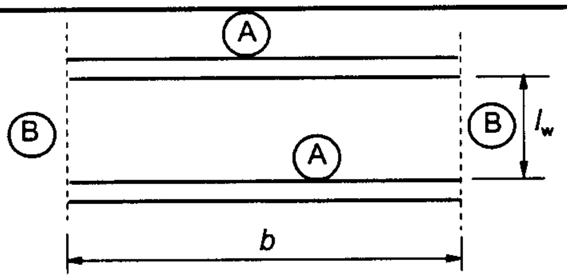
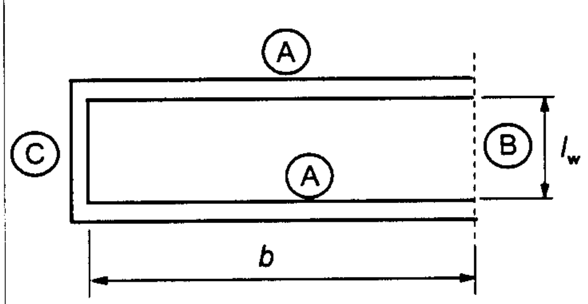
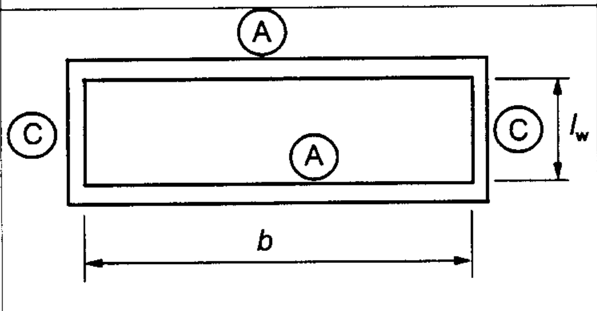
$$l_0 = \beta \cdot l_w \tag{12.9}$$

where:

l_w clear height of the member

β coefficient which depends on the support conditions:
for columns $\beta = 1$ should in general be assumed;
for cantilever columns or walls $\beta = 2$;
for other walls β-values are given in Table 12.1.

Table 12.1: Values of β for different edge conditions

Lateral restraint	Sketch	Expression	Factor β	
along two edges			β = 1,0 for any ratio of l_w/b	
Along three edges		$\beta = \dfrac{1}{1+\left(\dfrac{l_w}{3b}\right)^2}$	b/l_w 0,2 0,4 0,6 0,8 1,0 1,5 2,0 5,0	β 0,26 0,59 0,76 0,85 0,90 0,95 0,97 1,00
Along four edges		If $b \geq l_w$ $\beta = \dfrac{1}{1+\left(\dfrac{l_w}{b}\right)^2}$ If $b < l_w$ $\beta = \dfrac{b}{2l_w}$	b/l_w 0,2 0,4 0,6 0,8 1,0 1,5 2,0 5,0	β 0,10 0,20 0,30 0,40 0,50 0,69 0,80 0,96

Ⓐ - Floor slab Ⓑ - Free edge Ⓒ - Transverse wall

Note: The information in Table 12.1 assumes that the wall has no openings with a height exceeding 1/3 of the wall height l_w or with an area exceeding 1/10 of the wall area. In walls laterally restrained along 3 or 4 sides with openings exceeding these limits, the parts between the openings should be considered as laterally restrained along 2 sides only and be designed accordingly.

(2) The β-values should be increased appropriately if the transverse bearing capacity is affected by chases or recesses.

(3) A transverse wall may be considered as a bracing wall if:
- its total depth is not less than 0,5 h_w, where h_w is the overall depth of the braced wall;
- it has the same height l_w as the braced wall under consideration;
- its length l_{ht} is at least equal to l_w / 5, where l_w denotes the clear height of the braced wall;
- within the length l_{ht} the transverse wall has no openings.

(4) In the case of a wall connected along the top and bottom in flexurally rigid manner by insitu concrete and reinforcement, so that the edge moments can be fully resisted, the values for β given in Table 12.1 may be factored by 0,85.

(5) The slenderness of walls in plain concrete cast insitu should generally not exceed λ = 86 (i.e. l_0/h_w = 25). However, for compression members with l_0/h_w < 2,5, second order analysis is not necessary.

12.6.5.2 Simplified design method for walls and columns

(1) In absence of a more rigorous approach, the design resistance in terms of axial force for a slender wall or column in plain concrete may be calculated as follows:

$$N_{Rd} = b \times h_w \times f_{cd} \times \Phi \qquad (12.10)$$

where
N_{Rd} is the axial resistance
b is the overall width of the cross-section
h_w is the overall depth of the cross-section
Φ Factor taking into account eccentricity, including second order effects and normal effects of creep; see below

For braced members, the factor Φ may be taken as:

$$\Phi = (1.14 \times (1-2e_{tot}/h_w) - 0.02 \times l_0/h_w \leq (1-2\, e_{tot}/h_w) \qquad (12.11)$$

where:

$$e_{tot} = e_o + e_i \qquad (12.12)$$

e_o is the first order eccentricity including, where relevant, the effects of floors (e.g. possible clamping moments transmitted to the wall from a slab) and horizontal actions
e_i is the additional eccentricity covering the effects of geometrical imperfections, see 5.2

(2) Other simplified methods may be used provided that they are not less conservative than a rigorous method in accordance with 5.8.

12.7 Serviceability limit states

(1) Stresses should be checked where structural restraint is expected to occur.

(2) The following measures to ensure adequate serviceability should be considered:

a) with regard to crack formation:
- limitation of concrete tensile stresses to acceptable values;
- provision of subsidiary structural reinforcement (surface reinforcement, tying system where necessary);
- provision of joints;
- methods of concrete technology (e.g. appropriate concrete composition, curing);
- choice of appropriate method of construction.

b) with regard to limitation of deformations:
- a minimum section size (see 12.9 below);
- limitation of slenderness in the case of compression members.

(3) Any reinforcement provided in plain concrete members, although not taken into account for load bearing purposes, should comply with 4.4.1.

12.9 Detailing provisions

12.9.1 Structural members

(1) The overall depth h_w of a wall should not be smaller than 120 mm for cast in-situ concrete walls.

(2) Where chases and recesses are included checks should be carried out to assure the adequate strength and stability of the member.

12.9.2 Construction joints

(1) Where tensile stresses in the concrete occur in construction joints are expected to occur, reinforcement should be detailed to control cracking.

12.9.3 Strip and pad footings

(1) In the absence of more detailed data, axially loaded strip and pad footings may be designed and constructed as plain concrete provided that:

$$0{,}85 \cdot h_F / a \geq \sqrt{(3\sigma_{gd}/f_{ctd})} \qquad (12.13)$$

where:
- h_F is the foundation depth
- a is the projection from the column face (see Figure 12.2)
- σ_{gd} is the design value of the ground pressure
- f_{ctd} is the design value of the concrete tensile strength (in the same unit as σ_{gd})

As a simplification the relation $h_F/a \geq 2$ may be used.

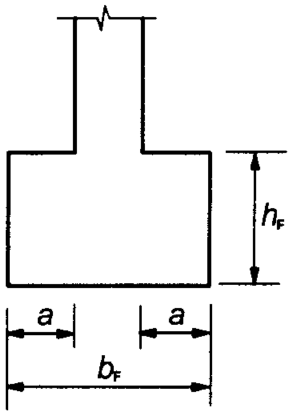


Figure: 12.2: Unreinforced pad footings; notations

Annex A (informative)

Modification of partial factors for materials

A.1 General

(1) The partial factors for materials given in Table 2.2 of 2.2.5 correspond to geometrical tolerances of Class 1 in ENV 13670-1 and normal level of workmanship and inspection (e.g. Inspection Class 2 in ENV 13670-1).

Note: If recommended values for partial factors given in Section 2 are altered in a National Annex, but are still based on the assumptions of A.1 (1), the recommended values for the reduced partial factors should be altered in the same proportion as given in this Annex.

(2) Recommendations for reduced partial factors for materials are given in this Informative Annex. More detailed rules on control procedures may be given in product standards for precast elements.

Note: For more information see Annex B of EN 1990.

A.2 In situ concrete structures

A.2.1 Reduction based on quality control and reduced tolerances

(1) If execution is subjected to a quality control system, which ensures that unfavourable deviations of cross-section dimensions are within the reduced tolerances given in Table A.1, the partial safety factor for reinforcement may be reduced to $\gamma_{S,red1}$.

Table A.1: Reduced tolerances

h or b (mm)	Reduced tolerances (mm)	
	Cross-section dimension $\pm\Delta h$, Δb (mm)	Position of reinforcement $+\Delta c$ (mm)
≤ 150	5	5
400	10	10
≥ 2500	30	20

Note 1: Linear interpolation may be used for intermediate values.
Note 2: $+\Delta c$ refers to the mean value of reinforcing bars or prestressing tendons in the cross-section or over a width of one metre (e.g. slabs and walls).

Note: The value of $\gamma_{S,red1}$ for use in a Country may be found in its National Annex. The recommended value is 1,1.

(2) Under the condition given in A.2.1 (1), and if the coefficient of variation of the concrete strength is shown not to exceed 10 %, the partial safety factor for concrete may be reduced to $\gamma_{C,red1}$.

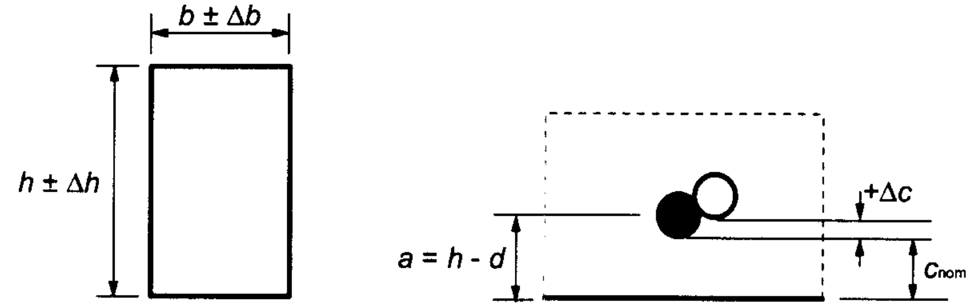
Note: The value of $\gamma_{C,red1}$ for use in a Country may be found in its National Annex. The recommended value is 1,4.

A.2.2 Reduction based on using reduced or measured geometrical parameters in design

(1) If the calculation of design resistance is based on critical dimensions, including effective depth (see Figure A.1), which are either:
- reduced by tolerances, or
- measured in the finished structure,

the partial safety factors may be reduced to $\gamma_{s,red2}$ and $\gamma_{c,red2}$.

Note: The values of $\gamma_{s,red2}$ and $\gamma_{c,red2}$ for use in a Country may be found in its National Annex. The recommended value of $\gamma_{s,red2}$ is 1,05 and of $\gamma_{c,red2}$ is 1,45.



a) Cross section **b) Position of reinforcement** (unfavourable direction for effective depth)

Figure A.1: Cross-section tolerances

(2) Under the conditions given in A.2.2 (1) and provided that the coefficient of variation of the concrete strength is shown not to exceed 10%, the partial factor for concrete may be reduced to $\gamma_{c,red3}$ = 1,35.

Note: The value of $\gamma_{c,red3}$ for use in a Country may be found in its National Annex. The recommended value is 1,35.

A.2.3 Reduction based on assessment of concrete strength in finished structure

(1) For concrete strength values based on testing in a finished structure or element, see prEN 13791[1], EN 206-1 and relevant product standards, γ_c may be reduced by the conversion factor η.

Note: The value of η for use in a Country may be found in its National Annex. The recommended value is 0,85.

The value of γ_c to which this reduction is applied may already be reduced according to A.2.1 or A.2.2. However, the resulting value of the partial factor should not be taken less than $\gamma_{c,red4}$.

Note: The value of $\gamma_{c,red4}$ for use in a Country may be found in its National Annex. The recommended value is 1,3.

A.3 Precast products

A.3.1 General

[1] pr*EN 13791. Assessment of concrete compressive strength in structures or in structural elements*

(1) These provisions apply to precast products as described in Section 10, linked to quality assurance systems and given attestation of conformity.

Note: Factory production control of CE-marked precast products is certified by notified body (Attestation level 2+).

A.3.2 Partial factors for materials

(1) Reduced values of $\gamma_{C,pcred}$ and $\gamma_{S,pcred}$ may be used, if justified by adequate control procedures, endorsed by relevant documents, such as specific product standards.

(2) Recommendations for reduced partial factors for materials, on the same technical basis as in A.2 of this Annex A, and adapted to factory production control of precast products are given in product standards. General recommendations are given in EN 13369 .

A.4 Precast elements

(1) The rules given in A.2 for insitu concrete structures also apply to precast concrete elements as defined in 10.1.1.

ANNEX B (Informative)

Creep and shrinkage

B.1 Basic equations for determining the creep coefficient

(1) The creep coefficient $\varphi(t,t_0)$ may be calculated from:

$$\varphi(t,t_0) = \varphi_0 \cdot \beta_c(t-t_0) \tag{B.1}$$

where:

φ_0 is the notional creep coefficient and may be estimated from:

$$\varphi_0 = \varphi_{RH} \cdot \beta(f_{cm}) \cdot \beta(t_0) \tag{B.2}$$

φ_{RH} is a factor to allow for the effect of relative humidity on the notional creep coefficient:

$$\varphi_{RH} = 1 + \frac{1-RH/100}{0{,}1 \cdot \sqrt[3]{h_0}} \quad \text{for } f_{cm} \leq 35 \text{ MPa} \tag{B.3a}$$

$$\varphi_{RH} = \left[1 + \frac{1-RH/100}{0{,}1 \cdot \sqrt[3]{h_0}} \cdot \alpha_1\right] \cdot \alpha_2 \quad \text{for } f_{cm} > 35 \text{ MPa} \tag{B.3b}$$

RH is the relative humidity of the ambient environment in %

$\beta(f_{cm})$ is a factor to allow for the effect of concrete strength on the notional creep coefficient:

$$\beta(f_{cm}) = \frac{16{,}8}{\sqrt{f_{cm}}} \tag{B.4}$$

f_{cm} is the mean compressive strength of concrete in N/mm^2 at the age of 28 days

$\beta(t_0)$ is a factor to allow for the effect of concrete age at loading on the notional creep coefficient:

$$\beta(t_0) = \frac{1}{(0{,}1 + t_0^{0{,}20})} \tag{B.5}$$

h_0 is the notional size of the member in mm where:

$$h_0 = \frac{2A_c}{u} \tag{B.6}$$

A_c is the cross-sectional area

u is the perimeter of the member in contact with the atmosphere

β_c is a coefficient to describe the development of creep with time after loading, and may be estimated using the following Expression:

$$\beta_c(t-t_0)=\left[\frac{(t-t_0)}{(\beta_H+t-t_0)}\right]^{0,3} \quad \text{(B.7)}$$

t is the age of concrete in days at the moment considered
t_0 is the age of concrete at loading in days
$t-t_0$ is the non-adjusted duration of loading in days
β_H is a coefficient depending on the relative humidity (RH in %) and the notional member size (h_0 in mm). It may be estimated from:

$\beta_H = 1,5\ [1 + (0,012\ RH)^{18}]\ h_0 + 250 \leq 1500$ for $f_{cm} \leq 35$ (B.8a)

$\beta_H = 1,5\ [1 + (0,012\ RH)^{18}]\ h_0 + 250\ \alpha_3 \leq 1500\ \alpha_3$ for $f_{cm} \geq 35$ (B.8b)

$\alpha_{1/2/3}$ are coefficients to consider the influence of the concrete strength:

$$\alpha_1=\left[\frac{35}{f_{cm}}\right]^{0,7} \quad \alpha_2=\left[\frac{35}{f_{cm}}\right]^{0,2} \quad \alpha_3=\left[\frac{35}{f_{cm}}\right]^{0,5} \quad \text{(B.8c)}$$

(2) The effect of type of cement on the creep coefficient of concrete may be taken into account by modifying the age of loading t_0 in Expression (B.5) according to the following Expression:

$$t_0=t_{0,T}\cdot\left(\frac{9}{2+t_{0,T}^{\ 1,2}}+1\right)^{\alpha}\geq 0,5 \quad \text{(B.9)}$$

where:
$t_{0,T}$ is the temperature adjusted age of concrete at loading in days adjusted according to Expression (B.10)
α is a power which depends on type of cement:
= -1 for slowly hardening cements, S
= 0 for normal or rapid hardening cements, N
= 1 for rapid hardening high strength cements, R

(3) The effect of elevated or reduced temperatures within the range 0 – 80°C on the maturity of concrete may be taken into account by adjusting the concrete age according to the following Expression:

$$t_T=\sum_{i=1}^{n} e^{-(4000/[273+T(\Delta t_i)]-13,65)}\cdot\Delta t_i \quad \text{(B.10)}$$

where:
t_T is the temperature adjusted concrete age which replaces t in the corresponding equations
$T(\Delta t_i)$ is the temperature in °C during the time period Δt_i
Δt_i is the number of days where a temperature T prevails.

The mean coefficient of variation of the above predicted creep data, deduced from a computerised data bank of laboratory test results, is of the order of 20%.

The values of $\varphi(t,t_0)$ given above should be associated with the tangent modulus $E_{c(28)} = 1,05\ E_{cm}$

When a less accurate estimate is considered satisfactory, the values given in Figures 3.1 and 3.2 of 3.1.3 may be adopted for creep of concrete at 70 years.

B.2 Basic equations for determining the drying shrinkage

(1) The basic drying shrinkage strain $\varepsilon_{cd,\infty}$ is calculated from

$$\varepsilon_{cd,\infty} = 0{,}85\left[(220+110\cdot\alpha_{ds1})\cdot\exp\left(-\alpha_{ds2}\cdot\frac{f_{cm}}{f_{cmo}}\right)\right]\cdot 10^{-6}\cdot\beta_{RH} \qquad \text{(B.11)}$$

$$\beta_{RH} = -1{,}55\left[1-\left(\frac{RH}{RH_0}\right)^3\right] \qquad \text{(B.12)}$$

where:

f_{cm} is the mean compressive strength (MPa)
f_{cmo} = 10 Mpa
α_{ds1} is a coefficient which depends on the type of cement
= 3 for slowly hardening cements (*S*)
= 4 for normal or rapidly hardening cements (*N*)
= 6 for rapidly hardening high-strength cements (*R*)
α_{ds2} is a coefficient which depends on the type of cement
= 0,13 for slowly hardening cements (*S*)
= 0,12for normal or rapidly hardening cements (*N*)
= 0,11 for rapidly hardening high-strength cements (*R*)
RH is the ambient relative humidity (%)
RH_0 = 100%.

ANNEX C (Normative)

Properties of reinforcement suitable for use with this Eurocode

C.1 General

(1) Table C.1 gives the properties of reinforcement suitable for use with this Eurocode. The properties are valid for temperatures between -40°C and 100°C for the reinforcement in the finished structure. Any bending and welding of reinforcement carried out on site shall be further restricted to the temperature range as permitted by EN 13670.

Table C.1: Properties of reinforcement

Product form	**Bars and de-coiled rods**			**Wire Fabrics**			**Requirement or quantile value (%)**
Class	A	B	C	A	B	C	-
Characteristic yield strength f_{yk} or $f_{0,2k}$ (MPa)	400 to 600						5,0
Minimum value of $k = (f_t/f_y)_k$	≥1,05	≥1,08	≥1,15 <1,35	≥1,05	≥1,08	≥1,15 <1,35	10,0
Characteristic strain at maximum force, ε_{uk} (%)	≥2,5	≥5,0	≥7,5	≥2,5	≥5,0	≥7,5	10,0
Fatigue stress range (MPa) (for N ≥ 2 x 10^6 cycles) with an upper limit of βf_{yk}	≥150			≥100			10,0
Bendability	Bend/Rebend test			-			
Shear strength	-			0,3 $A f_{yk}$ (A is area of wire)			Minimum
Bond: Minimum relative rib area, $f_{R,min}$ — Nominal bar size (mm) 5 - 6 6,5 to 12 > 12	 0,035 0,040 0,056						5,0
Maximum deviation from nominal mass (individual bar or wire) (%) — Nominal bar size (mm) ≤ 8 > 8	 ± 6,0 ± 4,5						5,0

Note: The value of β for use in a Country may be found in its National Annex. The recommended value is 0,6

(2) The values of f_{yk}, k and ε_{uk} in Table C.1 are characteristic values. The maximum % of test results falling below the characteristic value is given for each of the characteristic values in the right hand column of Table C.1.

(3) PrEN10080 does not specify the quantile value for characteristic values, nor the evaluation of test results for individual test units.

In order to be deemed to comply with the long term quality levels in Table C.1, the following limits on test results should be applied:

- where all individual test results of a test unit exceed the characteristic value, (or are below the characteristic value in the case the maximum value of f_{yk} or k) the test unit may be assumed to comply.
- the individual values of yield strength $f_{yk,}$ k and ε_{uk} shall be greater than the minimum values and less than the maximum values. In addition, the mean value, M, of a test unit shall satisfy the equation

$$M \geq C_v + a \tag{C.1}$$

where

C_v is the long term characteristic value

a is a coefficient which depends on the parameter considered

Note 1: The value of a for use in a Country may be found in its National Annex. The recommended value for f_{yk} is 10 MPa and for both k and ε_{uk} is 0.

Note 2: The minimum and maximum values of $f_{yk,}$ k and ε_{uk} for use in a Country may be found in its National Annex. The recommend values are given in Table C.2.

Table C.2. Absolute limits on test results

Performance characteristic	Minimum value	Maximum value
Yield strength f_{yk}	0,97 x minimum C_v	1,03 x maximum C_v
k	0,98 x minimum C_v	1,02 x maximum C_v
ε_{uk}	0,80 x minimum C_v	Not applicable

C.2 Strength

(1)P The maximum actual yield stress $f_{y,max}$ shall not exceed $1,3f_{yk}$.

C.3 Bendability

(1)P Bendability shall be verified by the bend and rebend tests in accordance with EN 10080 and EN ISO 15630-1. In situations where verification is carried out just using a rebend test the mandrel size shall be no greater than that specified for bending in Table 8.1 of this Eurocode. In order to ensure bendability no cracking shall be visible after the first bend.

C.4 Fatigue

(1) The fatigue requirements given in Table C.1 are not required for reinforcement for predominantly static loading. If higher values of the fatigue stress range and/or the number of cycles are shown to apply by testing to apply then the values in Table 6.3 may be modified accordingly. Such testing should be in accordance with EN 10080.

C.5 Bond

(1) Where it can be shown that sufficient bond strength is achievable with f_R values less than specified above, the values may be relaxed. In order to ensure that sufficient bond strength is achieved, the bond stresses shall satisfy Expressions (C.2) and (C.3) when tested using the CEB/RILEM beam test:

$$\tau_m \geq 0{,}098\ (80 - 1{,}2\phi) \tag{C.2}$$

$$\tau_r \geq 0{,}098\ (130 - 1{,}9\phi) \tag{C.3}$$

where:
- ϕ is the nominal bar size (mm)
- τ_m is the mean value of bond stress (MPa) at 0,01, 0,1 and 1 mm slip
- τ_r is the bond stress at failure by slipping

ANNEX D (Informative)

Detailed calculation method for prestressing steel relaxation losses

D.1 General

(1) In the case that the relaxation losses are calculated for different time intervals (stages) where the stress in the prestressing tendon is not constant, for example due to the elastic shortening of the concrete, an equivalent time method should be adopted.

(2) The concept of the equivalent time method is presented in the Figure D.1, where at time t_i there is an instantaneous deformation of the prestressing tendon, with:

σ_{pi}^- is the tensile stress in the tendon just before t_i
σ_{pi}^+ is the tensile stress in the tendon just after t_i
σ_{pi-1}^+ is he tensile stress in the tendon at the preceding stage
$\Delta\sigma_{pr,\,i-1}$ is the absolute value of the relaxation loss during the preceding stage
$\Delta\sigma_{pr,i}$ is the absolute value of the relaxation loss of the stage considered

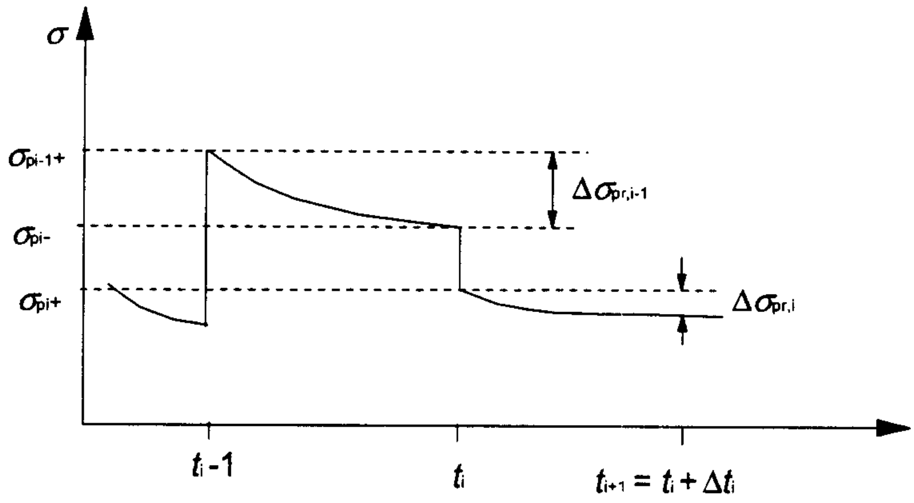


Figure D.1: Equivalent time method

(3) Let $\sum_{1}^{i-1}\Delta\sigma_{pr,j}$ be the sum of all the relaxation losses of the preceding stages and t_e is defined as the equivalent time (in hours) necessary to obtain this sum of relaxation losses that verifies the relaxation time functions in 3.3.2 (7) with an initial stress equal to $\sigma_{pi^+} + \sum_{1}^{i-1}\Delta\sigma_{pr,j}$

and with $\mu = \dfrac{\sigma_{pi^+} + \sum_{1}^{i-1}\Delta\sigma_{pr,j}}{f_{pk}}$.

(4) For example, for a Class 2 prestressing tendon t_e is given by Expression (3.31) becomes:

$$\Delta\sigma_{pr,j} = 0{,}66\,\rho_{1000}\,e^{9{,}09\,\mu}\left(\frac{t_e}{1000}\right)^{0{,}75\,(1-\mu)}\left\{\sigma_{pi^+} + \sum_{1}^{i-1}\Delta\sigma_{pr,j}\right\}10^{-5} \tag{D.1}$$

(5) After resolving the above equation for t_e, the same formula can be applied in order to estimate the relaxation loss of the stage considered, $\Delta\sigma_{pr,i}$ (where the equivalent time t_e is added to the interval of time considered):

$$\Delta\sigma_{pr,i} = 0{,}66\,\rho_{1000}\,e^{9{,}09\,\mu}\left(\frac{t_e + \Delta t_i}{1000}\right)^{0{,}75\,(1-\mu)}\left\{\sigma_{pi^+} + \sum_{1}^{i-1}\Delta\sigma_{pr,j}\right\}10^{-5} - \sum_{1}^{i-1}\Delta\sigma_{pr,j} \tag{D.2}$$

(6) The same principle applies for all three classes of prestressing tendons.

Annex E (Informative)

Indicative strength classes for durability

E.1 General

(1) The choice of adequately durable concrete for corrosion protection of reinforcement and protection of concrete attack, requires consideration of the composition of concrete. This may result in a higher compressive strength of the concrete than is required for structural design. The relationship between concrete strength classes and environmental classes (see Table 4.1) may be described by indicative strength classes.

(2) When the chosen strength is higher than that required for structural design the value of $f_{ct,m}$ should be associated with the higher strength in the calculation of minimum reinforcement according to 7.3.2 and 9.1.1.1 and crack width control according to 7.3.3 and 7.3.4.

Note: Values of indicative strength classes for use in a Country may be found in its National Annex. The recommended values are given in Table E.1N.

Table E.1N: Indicative strength classes

	Exposure Classes according to Table 4.1									
Corrosion										
	Carbonation-induced corrosion				Chloride-induced corrosion			Chloride-induced corrosion from sea-water		
	XC1	XC2	XC3	XC4	XD1	XD2	XD3	XS1	XS2	XS3
Indicative Strength Class	C20/25	C25/30	C30/37		C30/37		C35/45	C30/37	C35/45	

Damage to Concrete							
	No risk	Freeze/Thaw Attack			Chemical Attack		
	X0	XF1	XF2	XF3	XA1	XA2	XA3
Indicative Strength Class	C12/15	C30/37	C25/30	C30/37	C30/37		C35/45

Annex F (Informative)

Reinforcement expressions for in-plane stress conditions

(1) The reinforcement in an element subject to in-plane orthogonal stresses σ_{Edx}, σ_{Edy} and τ_{Edxy} may be calculated using the procedure set out below. Compressive stresses should be taken as positive, with $\sigma_{Edx} > \sigma_{Edy}$, and the direction of reinforcement should coincide with the x and y axes.

The tensile strengths provided by reinforcement should be determined from:

$$f_{tdx} = \rho_x f_{yd} \text{ and } f_{tdy} = \rho_y f_{yd} \tag{F.1}$$

where ρ_x and ρ_y are the geometric reinforcement ratios, along the x and y axes respectively.

(2) In locations where σ_{Edx} and σ_{Edy} are both compressive and $\sigma_{Edx} \cdot \sigma_{Edy} > \tau^2_{Edxy}$, design reinforcement is not required. However the maximum compressive stress should not exceed f_{cd} (See 3.1.5)

(3) In locations where σ_{Edy} is tensile or $\sigma_{Edx} \cdot \sigma_{Edy} \leq \tau^2_{Edxy}$, reinforcement is required.

The optimum reinforcement, indicated by superscript ′, and related concrete stress are determined by:

For $\sigma_{Edx} \leq |\tau_{Edxy}|$

$$f'_{tdx} = |\tau_{Edxy}| - \sigma_{Edx} \tag{F.2}$$

$$f'_{tdy} = |\tau_{Edxy}| - \sigma_{Edy} \tag{F.3}$$

$$\sigma_{cd} = 2|\tau_{Edy}| \tag{F.4}$$

For $\sigma_{Edx} > |\tau_{Edxy}|$

$$f'_{tdx} = 0 \tag{F.5}$$

$$f'_{tdy} = \frac{\tau^2_{Edxy}}{\sigma_{Edx}} - \sigma_{Edy} \tag{F.6}$$

$$\sigma_{cd} = \sigma_{Edx}\left(1 + \left(\frac{\tau_{Edxy}}{\sigma_{Edx}}\right)^2\right) \tag{F.7}$$

The concrete stress, σ_{cd}, should not exceed νf_{cd} (ν may be obtained from Expression (6.8)).

Note: The minimum reinforcement is obtained if the directions of reinforcement are identical to the directions of the principal stresses.

Alternatively, for the general case the necessary reinforcement and the concrete stress can be determined by:

$$f_{tdx} = |\tau_{Edxy}|\cot\theta - \sigma_{Edx} \tag{F.8}$$

$$f_{tdy} = |\tau_{Edxy}|/\cot\theta - \sigma_{Edy} \tag{F.9}$$

$$\sigma_{cd} = |\tau_{Edxy}| \left(\cot\theta + \frac{1}{\cot\theta}\right) \tag{F.10}$$

where θ is the angle of the concrete compressive stress to the x-axis.

Note: If the previous calculations result in negative values of f_{td} they should be taken as zero.

In order to avoid unacceptable cracks for the serviceability state, and to ensure the required deformation capacity for the ultimate limit state, the reinforcement derived from Expressions (F.8) and (F.9) for each direction should not be more than twice and not less than half the reinforcement determined by expressions (F2) and (F3) or (F5) and (F6). These limitations are expressed by $\frac{1}{2} f'_{tdx} \leq f_{tdx} \leq 2 f'_{tdx}$ and $\frac{1}{2} f'_{tdy} \leq f_{tdy} \leq 2 f'_{tdy}$.

(4) The reinforcement should be fully anchored at all free edges, e.g. by U-bars or similar.

Annex G (Informative)

Soil structure interaction

G.1 Shallow foundations

G.1.1 General

(1) The interaction between the ground, the foundation and the superstructure should be considered. The contact pressure distribution on the foundations and the column forces are both dependent on the relative settlements.

(2) In general the problem may be solved by ensuring that the displacements and associated reactions of the soil and the structure are compatible.

(3) Although the above general procedure is adequate, many uncertainties still exist, due to the load sequence and creep effects. For this reason different levels of analysis, depending on the degree of idealisation of the mechanical models, are usually defined.

(4) If the superstructure is considered as flexible, then the transmitted loads do not depend on the relative settlements, because the structure has no rigidity. In this case the loads are no longer unknown, and the problem is reduced to the analysis of a foundation on a deforming ground.

(5) If the superstructure is considered as rigid, then the unknown foundation loads can be obtained by the condition that settlements must lie on a plane. It must be checked that thise rigidity exists until the ultimate limit state is reached.

(6) A further simplifying scheme arises if the foundation system can be assumed to be rigid or the supporting ground is very stiff. In either case the relative settlements may be ignored and no modification of the loads transmitted from the superstructure is required.

(7) To determine the approximate rigidity of the structural system, an analysis may be made comparing the combined stiffness of the foundation, superstructure framing members and shear walls, with the stiffness of the ground. This relative stiffness K_R will determine whether the foundation or the structural system should be considered rigid or flexible. The following expression may be used for building structures:

$$K_R = (EJ)_S / (El^3) \tag{G.1}$$

where:

$(EJ)_S$ is the approximate value of the flexural rigidity per unit width of the building structure under consideration, obtained by summing the flexural rigidity of the foundation, of each framed member and any shear wall
E is the deformation modulus of the ground
l is the length of the foundation

Relative stiffnesses higher than 0,5 indicate rigid structural systems.

G.1.2 Levels of analysis

(1) For design purposes, the following levels of analysis are permitted:

Level 0: In this level, linear distribution of the contact pressure may be assumed.

The following preconditions should be fulfilled:
- the contact pressure does not exceed the design values for both the serviceability and the ultimate limit states;
- at the serviceability limit state, the structural system is not affected by settlements, or the expected differential settlements are not significant;
- at the ultimate limit state, the structural system has sufficient plastic deformation capacity so that differences in settlements do not affect the design.

Level 1: The contact pressure may be determined taking into account the relative stiffness of the foundation and the soil and the resulting deformations evaluated to check that they are within acceptable limits.

The following preconditions should be fulfilled:

- sufficient experience exists to show that the serviceability of the superstructure is not likely to be affected by the resulting soil deformation;
- at the ultimate limit state, the structural system has adequate ductile behaviour.

Level 2: At this level of analysis the influence of ground deformations on the superstructure is considered. The structure is analysed under the imposed deformation of the foundation to determine the adjustments to the loads applied to the foundations. If the resulting adjustments are significant (i.e. > |10| %) then Level 3 analysis should be adopted.

Level 3: This is a complete interactive procedure taking into account the overall structural system.

G.2 Piled foundations

(1) If the pile cap is rigid, a linear variation of the settlements of the individual piles may be assumed which depends on the rotation of the pile cap. If this rotation is zero or may be ignored, equal settlement of all piles may be assumed. From equilibrium equations, the unknown pile loads and the settlement of the group can be calculated.

(2) However, when dealing with a piled raft, interaction occurs not only between individual piles but also between the raft and the piles, and no simple approach to analyse this problem is available.

(3) The response of a pile group to horizontal loads generally involves not only the lateral stiffness of the surrounding soil and of the piles, but also their axial stiffness (e.g. lateral load on a pile group causes tension and compression on edge piles).

Annex H (Informative)

Global second order effects in structures

H.1 Criteria for neglecting global second order effects

H.1.1 General

(1) Clause H.1 gives criteria for structures where the conditions in 5.8.3.3 (1) are not met. The criteria are based on 5.8.2 (6) and take into account global bending and shear deformations, as defined in Figure H.1.

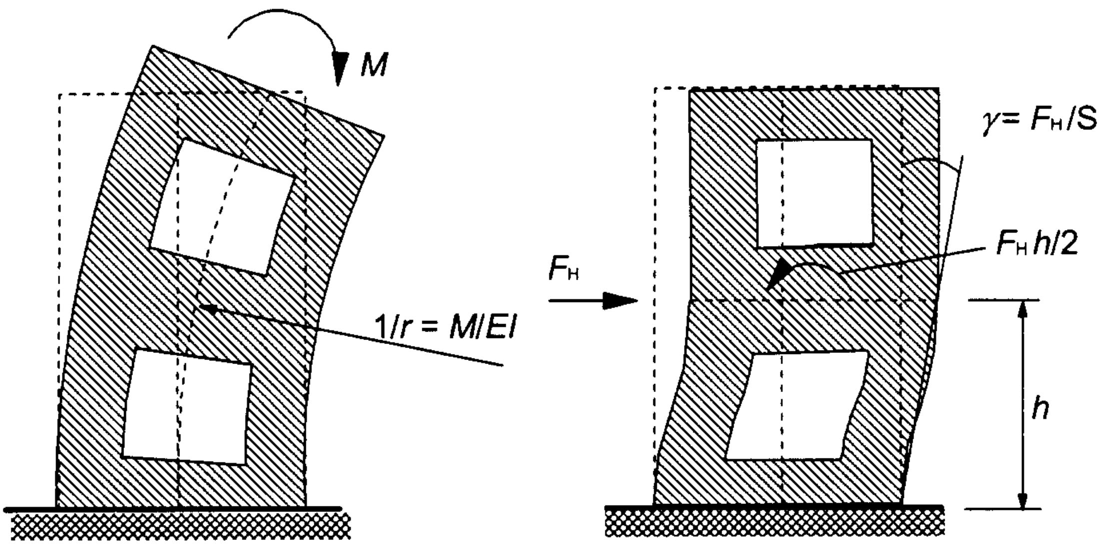


Figure H.1: Definition of global bending and shear deformations (*1/r* and γ respectively) and the corresponding stiffnesses (*EI* and *S* respectively)

H.1.2 Bracing system without significant shear deformations

(1) For a bracing system without significant shear deformations (e.g. shear walls without openings), global second order effects may be ignored if:

$$F_{V,Ed} \leq 0,1 \cdot F_{V,BB} \qquad \text{(H.1)}$$

where:

$F_{V,Ed}$ is the total vertical load (on braced and bracing members)

$F_{V,BB}$ is the nominal global buckling load for global bending, see (2)

(2) The nominal global buckling load for global bending may be taken as

$$F_{V,BB} = \xi \cdot \Sigma EI / L^2 \qquad \text{(H.2)}$$

where:

ξ is a coefficient depending on number of storeys, variation of stiffness, rigidity of base restraint and load distribution; see (4)

ΣEI is the sum of bending stiffnesses of bracing members in direction considered, including possible effects of cracking; see (3)

L is the total height of building above level of moment restraint.

(3) In the absence of a more accurate evaluation of the stiffness, the following may be used for a bracing member with *cracked* section:

$$EI \approx 0{,}4\, E_{cd} I_c \tag{H.3}$$

where:

E_{cd} = E_{cm}/γ_{cE}, design value of concrete modulus, see 5.8.6 (3)
I_c second moment of area of bracing member

If the cross-section is shown to be *uncracked* in the ultimate limit state, constant 0,4 in Expression (H.3) may be replaced by 0,8.

(4) If bracing members have constant stiffness along the height and the total vertical load increases with the same amount per storey, then ξ may be taken as

$$\xi = 7{,}8 \cdot \frac{n_s}{n_s + 1{,}6} \cdot \frac{1}{1 + 0{,}7 \cdot k} \tag{H.4}$$

where:

n_s is the number of storeys
k is the relative flexibility of moment restraint; see (5).

(5) The relative flexibility of moment restraint at the base is defined as:

$$k = (\theta/M)\cdot(EI/L) \tag{H.5}$$

where:

θ is the rotation for bending moment M
EI is the stiffness according to (3)
L is the otal height of bracing unit

Note: For $k = 0$, i.e. rigid restraint, Expressions (H.1)-(H.4) can be combined into Expression (5.18), where the coefficient 0,31 follows from $0{,}1 \cdot 0{,}4 \cdot 7{,}8 \approx 0{,}31$.

H.1.3 Bracing system with significant global shear deformations

(1) Global second order effects may be ignored if the following condition is fulfilled:

$$F_{V,Ed} \leq 0{,}1 \cdot F_{V,B} = 0{,}1 \cdot \frac{F_{V,BB}}{1 + F_{V,BB} / F_{V,BS}} \tag{H.6}$$

where

$F_{V,B}$ is the global buckling load taking into account global bending *and* shear
$F_{V,BB}$ is the global buckling load for pure bending, see H.1.2 (2)
$F_{V,BS}$ is the global buckling load for pure shear, $F_{V,BS} = \Sigma S$
ΣS is the total shear stiffness of bracing units (see Figure H.1)

Note: The global shear deformation of a bracing unit is normally governed mainly by local bending deformations (Figure H.1). Therefore, in the absence of a more refined analysis, cracking may be taken into account for S in the same way as for EI; see H.1.2 (3).

H.2 Methods for calculation of global second order effects

(1) This clause is based on linear second order analysis according to 5.8.7. Global second order effects may then be taken into account by analysing the structure for fictitious, magnified horizontal forces $F_{H,Ed}$:

$$F_{H,Ed} = \frac{F_{H,0Ed}}{1 - F_{V,Ed} / F_{V,B}} \tag{H.7}$$

where:

$F_{H,0Ed}$ is the first order horizontal force due to wind, imperfections etc.
$F_{V,Ed}$ is the total vertical load on bracing *and* braced members
$F_{V,B}$ is the nominal global buckling load, see (2).

(2) The buckling load $F_{V,B}$ may be determined according to H.1.3 (or H.1.2 if global shear deformations are negligible). However, in this case nominal stiffness values according to 5.8.7.2 should be used, including the effect of creep.

(3) In cases where the global buckling load $F_{V,B}$ is not defined, the following expression may be used instead:

$$F_{H,Ed} = \frac{F_{H,0Ed}}{1 - F_{H,1Ed} / F_{H,0Ed}} \tag{H.8}$$

where:

$F_{H,1Ed}$ fictitious horizontal force, giving the same bending moments as vertical load $N_{V,Ed}$ acting on the deformed structure, with deformation caused by $F_{H,0Ed}$ (first order deformation), and calculated with nominal stiffness values according to 5.8.7.2

Note: Expression (H.8) follows from a step-by-step numerical calculation, where the effect of vertical load and deformation increments, expressed as equivalent horizontal forces, are added in consecutive steps. The increments will form a geometric series after a few steps. Assuming that this occurs even at the first step, (which is analogous to assuming β =1 in 5.8.7.3 (3)), the sum can be expressed as in Expression (H.8). This assumption requires that the stiffness values representing the final stage of deformations are used in all steps (note that this is also the basic assumption behind the analysis based on nominal stiffness values).

In other cases, e.g. if uncracked sections are assumed in the first step and cracking is found to occur in later steps, or if the distribution of equivalent horizontal forces changes significantly between the first steps, then more steps have to be included in the analysis, until the assumption of a geometric series is met. Example with two more steps than in Expression (H.8):

$$F_{H,Ed} = F_{H,0Ed} + F_{H,1Ed} + F_{H,2Ed} /(1 - F_{H,3Ed} / F_{H,2Ed})$$

Annex I (Informative)

Examples of regions with discontinuity in geometry or action

I.1 Frame corners

I.1.1 General

(1) The concrete strength $\sigma_{Rd,max}$ should be determined with respect to 6.5.2 (compression zones with or without transverse reinforcement).

I.1.2 Frame corners with closing moments

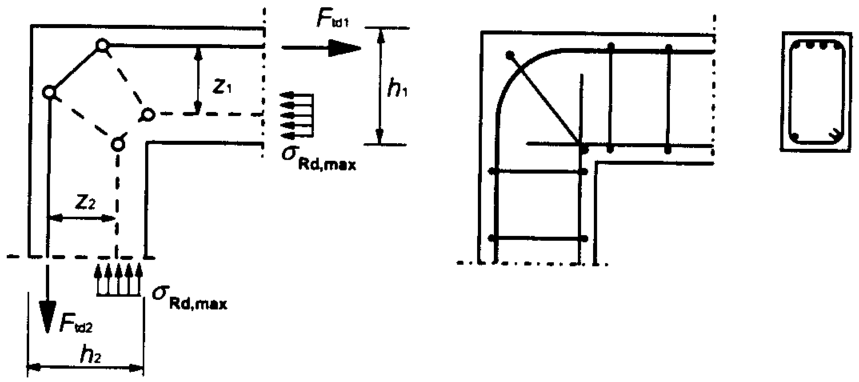
(1) For approximately equal depths of column and beam ($2/3 < h_2/h_1 < 3/2$) (see Figure I.1 (a)) no check of link reinforcement or anchorage lengths within the beam column joint is required, provided that all the tension reinforcement of the beam is bent around the corner.

(2) Figure I.1 (b) shows a strut and tie model for $h_2/h_1 < 3/2$ for a limited range of $\tan\theta$.

Note: The values of the limits of $\tan\theta$ for use in a Country may be found in its National Annex. The recommended value of the lower limit is 0,4 and the recommended value of the upper limit is 1.

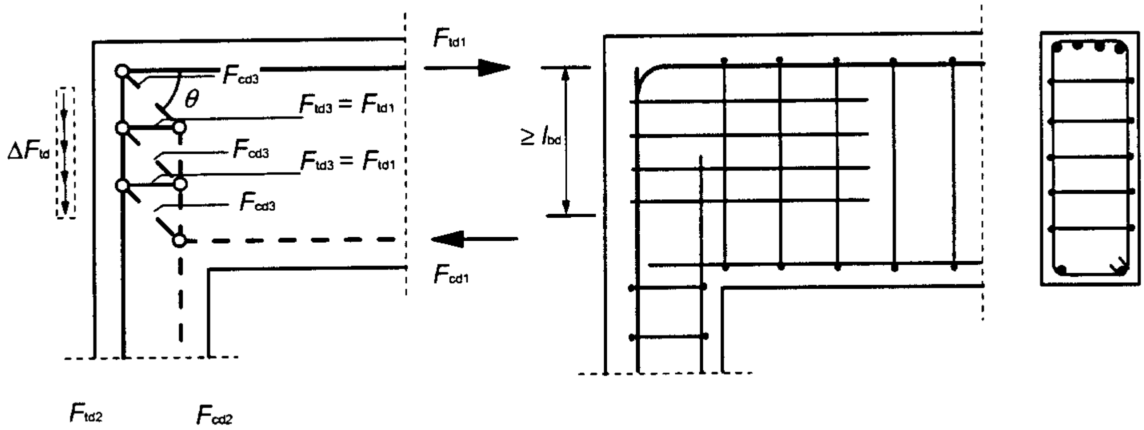
(3) The anchorage length l_{bd} should be determined for the force $\Delta F_{td} = F_{td2} - F_{td1}$.

(4) Reinforcement should be provided for transverse tensile forces perpendicular to an in-plane node.



(a) almost equal depth of beam and column

Figure I.1: Frame Corner with closing moment. Model and reinforcement

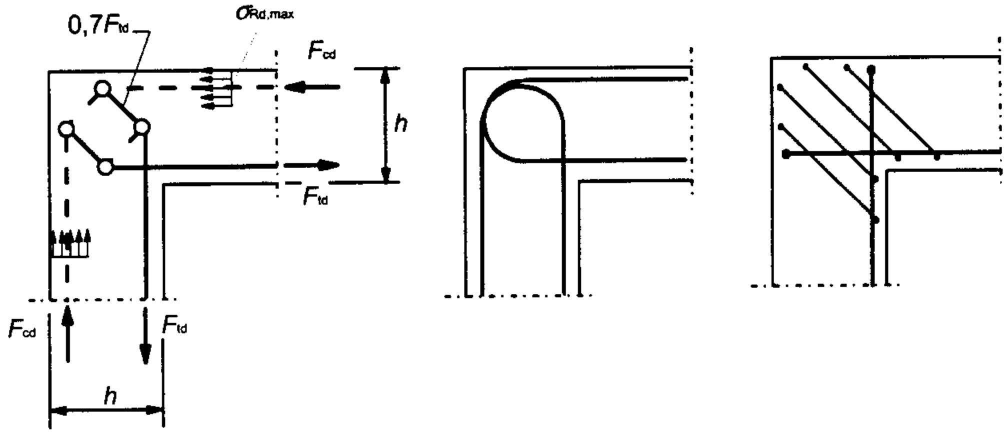


(b) very different depth of beam and column

Figure I.1 (cont.): Frame Corner with closing moment. Model and reinforcement

I.1.3 Frame corners with opening moments

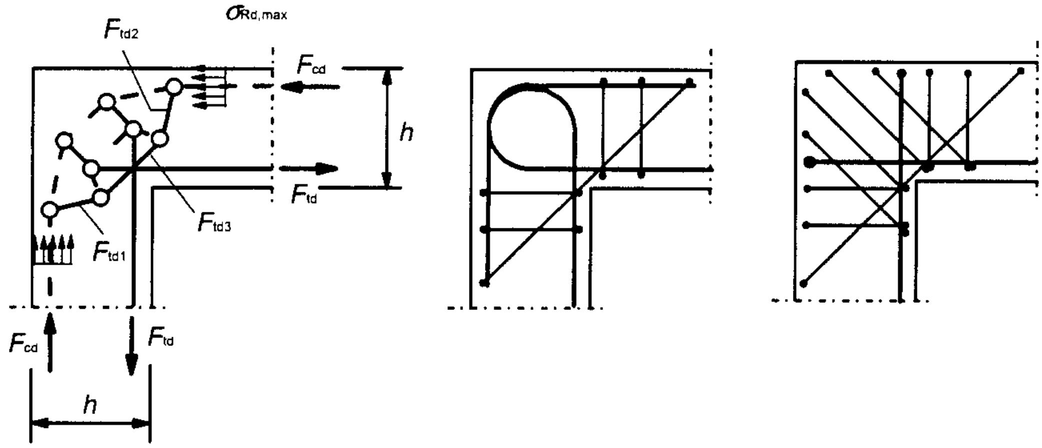
(1) For approximately equal depths of column and beam the strut and tie models given in Figures I.2 (a) and I.3 (a) may be used. Reinforcement should be provided as a loop in the corner region or as two overlapping U bars in combination with inclined links as shown in Figures I.2 (b) and (c) and Figures I.3 (b) and (c).



a) strut and tie model **(b) and (c) detailing of reinforcement**

Figure I.2: Frame corner with moderate opening moment (e.g. $A_s/bh \leq 2\%$)

(2) For large opening moments a diagonal bar and links to prevent splitting should be considered as shown in Figure I.3.



a) strut-and-tie model **(b) and (c) detailing of reinforcement**

Figure I.3: Frame corner with large opening moment (e.g. A_s/bh > 2%)

I.2 Corbels

(1) Corbels ($a_c < z_0$) may be designed using strut-and-tie models as described in 6.5 (see Figure I.4). The inclination of the strut is limited by $1{,}0 \leq \tan\theta \leq 2{,}5$.

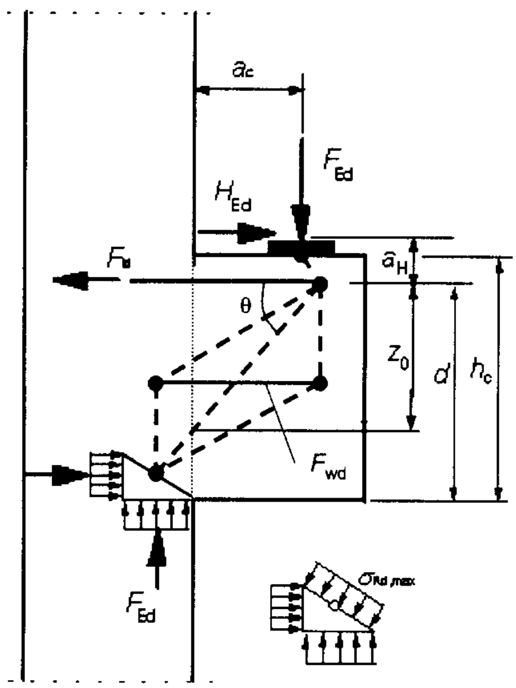


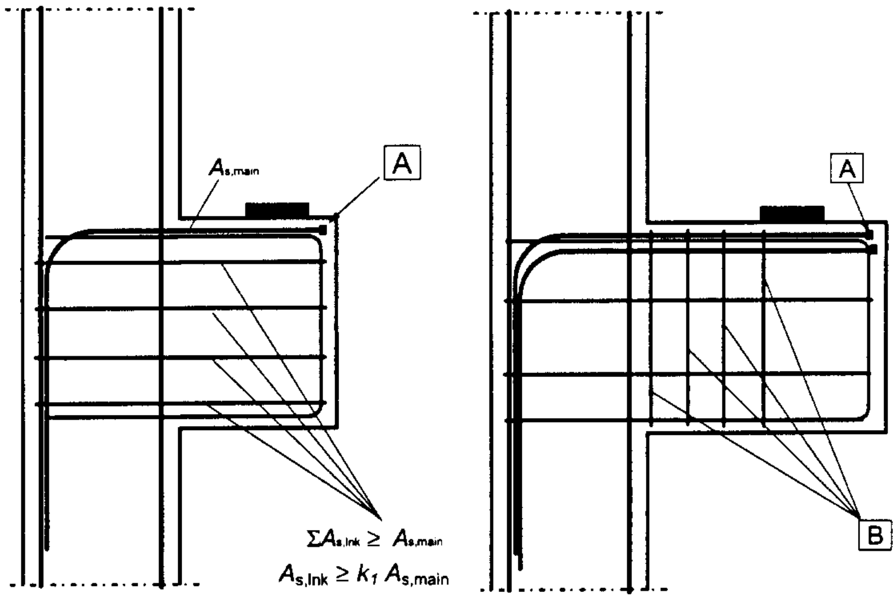
Figure I.4: Corbel strut-and-tie model

(2) If $a_c < 0{,}5\ h_c$ closed horizontal or inclined links with $A_{s,lnk} \geq k_1\ A_{s,main}$ should be provided in addition to the main tension reinforcement (see Figure I.5 (a)).

Note: The value of k_1 for use in a Country may be found in its National Annex. The recommended value is 0,25.

(3) If $a_c > 0{,}5\ h_c$ and $F_{Ed} > V_{Rd,ct}$ (see 6.2.2), closed vertical links $A_{s,lnk} \geq k_2\ F_{wd}/f_{yd}$ should be provided in addition to the main tension reinforcement (see Figure I.5 (b)).

Note: The value of k_2 for use in a Country may be found in its National Annex. The recommended value is 0,5.



A - anchorage devices or loops   B - Links

(a) reinforcement for $a_c \leq 0{,}5\ h_c$ (b) reinforcement for $a_c > 0{,}5\ h_c$

Figure I.5: Corbel detailing

(4) The anchorage of the main tension reinforcement in the supporting element should be verified. For bars bent in the vertical plane the anchorage length begins below the inner edge of the loading plate.

(5) If there are special requirements for crack limitation, inclined stirrups at the re-entrant opening will be effective.