(4) Reinforcement should be provided for the tensile force due to the effect of the action.

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 should be carried out 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, η, given by

$$\eta = \frac{A_S + A_P}{A_S + A_P\sqrt{\xi(\phi_S / \phi_P)}} \tag{6.64}$$

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 where ϕ_{wire} is the wire diameter

$\phi_P=1,20\ \phi_{wire}$ for single 3 wire strands where ϕ_{wire} is the wire diameter

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 6.2 may be used.

Table 6.2: Ratio of bond strength, ξ, between tendons and reinforcing steel

prestressing steel	ξ		
	pre-tensioned	bonded, post-tensioned	
		≤ C50/60	≥ C70/85
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
Note: For intermediate values between C50/60 and C70/85 interpolation may be used.			

(3) In the design of the shear reinforcement the inclination of the compressive struts θ_{fat} may be calculated using a strut and tie model or in accordance with Expression (6.65).

$$\tan\theta_{fat} = \sqrt{\tan\theta} \leq 1{,}0 \tag{6.65}$$

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 (a number of repeated actions of load).

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

$$E_d = E\{G_{k,j};P;\psi_{1,1}Q_{k,1};\psi_{2,i}Q_{k,i}\}\ j \geq 1;\ i > 1 \tag{6.66}$$

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

$$\sum_{j\geq 1} G_{k,j} \text{"+"}\ P \text{"+"}\ \psi_{1,1}Q_{k,1} \text{"+"} \sum_{i>1}\psi_{2,i}Q_{k,i} \tag{6.67}$$

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:

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

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

$$\left(\sum_{j\geq 1} G_{k,j} \text{"+"}\ P \text{"+"}\ \psi_{1,1}Q_{k,1} \text{"+"} \sum_{i>1}\psi_{2,i}Q_{k,i}\right) \text{"+"}\ Q_{fat} \tag{6.69}$$

where:

Q_{fat} is the 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

AC2 (1) The damage of a single stress range $\Delta\sigma$ 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 $\gamma_{F,fat}$. The resisting stress range at N^* cycles $\Delta\sigma_{Rsk}$ obtained should be divided by the safety factor $\gamma_{S,fat}$. AC2

AC2 **Note 1:** The value of $\gamma_{F,fat}$ is given in 2.4.2.3 (1). AC2

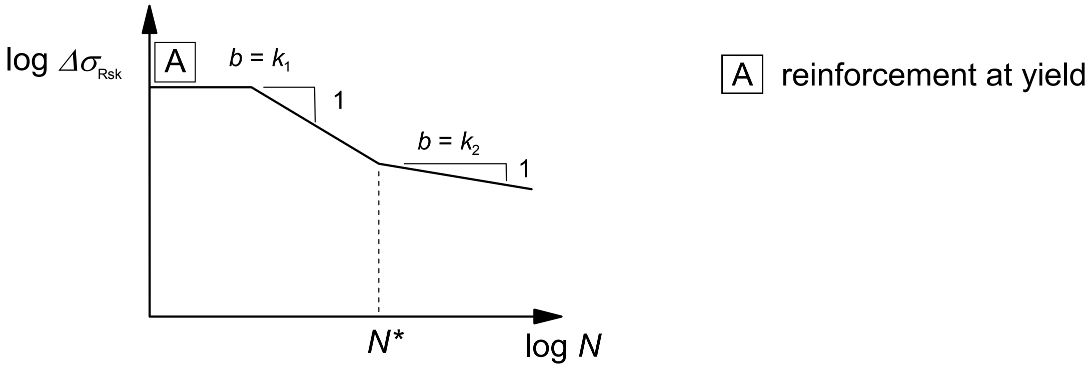

Figure 6.30: Shape of the characteristic fatigue strength curve (S-N-curves for reinforcing and prestressing steel)

Note 2: The values of parameters for reinforcing steels and prestressing steels S-N curves for use in a Country may be found in its National Annex. The recommended values are given in Table 6.3N and 6.4N which apply for reinforcing and prestressing steel respectively.

Table 6.3N: Parameters for S-N curves for reinforcing steel

Type of reinforcement	N^*	stress exponent k_1	stress exponent k_2	$\Delta\sigma_{Rsk}$ (MPa) at N^* cycles
Straight and bent bars[1]	10^6	5	9	162,5
Welded bars and wire fabrics	10^7	3	5	58,5
Splicing devices	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 6.4N: Parameters for S-N curves of prestressing steel

S-N curve of prestressing steel used for	N^*	stress exponent k_1	stress exponent k_2	$\Delta\sigma_{Rsk}$ (MPa) 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	10^6	5	5	80

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

$$D_{Ed} = \sum_i \frac{n(\Delta\sigma_i)}{N(\Delta\sigma_i)} < 1 \tag{6.70}$$

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 yield strength 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 may 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) Instead of an explicit verification of the damage 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) The method of damage equivalent stress range consists of representing the actual operational loading by N^* cycles of a single stress range. 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.

(3) For reinforcing or prestressing steel and splicing devices adequate fatigue resistance should be assumed if the Expression (6.71) is satisfied:

$$\gamma_{F,fat} \cdot \Delta\sigma_{S,equ}(N^*) \le \frac{\Delta\sigma_{Rsk}(N^*)}{\text{AC}_1\rangle\ \gamma_{S,fat}\ \langle\text{AC}_1} \tag{6.71}$$

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 6.3N and 6.4N.

$\Delta\sigma_{S,equ}(N^*)$ is the damage equivalent stress range for different types of reinforcement and considering the number of loading cycles N^*. For building construction $\Delta\sigma_{S,equ}(N^*)$ may be approximated by $\Delta\sigma_{S,max}$.
$\Delta\sigma_{S,max}$ is the maximum steel stress range under the relevant load combinations

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 $\Delta\sigma_S \leq k_1$.

Note: The value of k_1 for use in a Country may be found in its National Annex. The recommended value is 70MPa.

AC1 For welded reinforcing bars under tension adequate fatigue resistance may be assumed if the stress range under frequent cyclic load combined with the basic combination is $\Delta\sigma_S \leq k_2$. AC1

Note: The value of k_2 for use in a Country may be found in its National Annex. The recommended value is 35MPa.

(2) As a simplification to (1) AC1 above verification may be carried out using the frequent load AC1 combination. If this is satisfied then no further checks are necessary.

(3) Where welded joints or splicing devices are used in prestressed concrete, 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 k_3 for the mean value of prestressing force, $P_{m,}$

Note: The value of k_3 for use in a Country may be found in its National Annex. The recommended value is 0,9.

6.8.7 Verification of concrete under compression or shear

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

$$E_{cd,\max,equ} + 0{,}43\sqrt{1 - R_{equ}} \leq 1 \quad (6.72)$$

where:

$$R_{equ} = \frac{E_{cd,min,equ}}{E_{cd,max,equ}} \quad (6.73)$$

$$E_{cd,min,equ} = \frac{\sigma_{cd,min,equ}}{f_{cd,fat}} \quad (6.74)$$

$$E_{cd,max,equ} = \frac{\sigma_{cd,max,equ}}{f_{cd,fat}} \quad (6.75)$$

where :

R_{equ} is the stress ratio
$E_{cd,min,equ}$ is the minimum compressive stress level
$E_{cd,max,equ}$ is the maximum compressive stress level
$f_{cd,fat}$ is the design fatigue strength of concrete according to (6.76)
$\sigma_{cd,max,equ}$ is the upper stress of the ultimate amplitude for N cycles
$\sigma_{cd,min,equ}$ is the lower stress of the ultimate amplitude for N 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.

$$f_{cd,fat} = k_1 \beta_{cc}(t_0) f_{cd} \left(1 - \frac{f_{ck}}{250}\right) \tag{6.76}$$

where:

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

Note: The value of k_1 for use in a Country may be found in its National Annex. The recommended value for N = 10^6 cycles is 0,85.

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

$$\frac{\sigma_{c,max}}{f_{cd,fat}} \le 0{,}5 + 0{,}45 \frac{\sigma_{c,min}}{f_{cd,fat}} \tag{6.77}$$

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

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 $\sigma_{c,min}$ is a tensile stress, then $\sigma_{c,min}$ should be taken as 0.

(3) Expression (6.77) 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 strength reduction factor (see 6.2.2 (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 $\dfrac{V_{Ed,min}}{V_{Ed,max}} \ge 0$:

$$\frac{|V_{Ed,max}|}{|V_{Rd,c}|} \le 0{,}5 + 0{,}45 \frac{|V_{Ed,min}|}{|V_{Rd,c}|} \begin{cases} \le 0{,}9 \text{ up to C50/60} \\ \le 0{,}8 \text{ greater than C55/67} \end{cases} \tag{6.78}$$

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

$$\frac{|V_{Ed,max}|}{|V_{Rd,c}|} \le 0{,}5 - \frac{|V_{Ed,min}|}{|V_{Rd,c}|} \tag{6.79}$$

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

SECTION 7 SERVICEABILITY LIMIT STATES (SLS)

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.

(2) 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.

7.2 Stress limitation

(1)P The compressive stress 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) Longitudinal cracks may occur if the stress level under the characteristic combination of loads exceeds a critical value. Such cracking may lead to a reduction of durability. In the absence of other measures, such as an increase in the cover to reinforcement in the compressive zone or confinement by transverse reinforcement, it may be appropriate to limit the compressive stress to a value $k_1 f_{ck}$ in areas exposed to environments of exposure classes XD, XF and XS (see Table 4.1).

Note: The value of k_1 for use in a Country may be found in its National Annex. The recommended value is 0,6.

(3) If the stress in the concrete under the quasi-permanent loads is less than $k_2 f_{ck}$, linear creep may be assumed. If the stress in concrete exceeds $k_2 f_{ck}$, non-linear creep should be considered (see 3.1.4)

Note: The value of k_2 for use in a Country may be found in its National Annex. The recommended value is 0,45.

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

(5) AC1 For the appearance unacceptable cracking or deformation AC1 may be assumed to be avoided if, under the characteristic combination of loads, the tensile strength in the reinforcement does not exceed $k_3 f_{yk}$. Where the stress is caused by an imposed deformation, the tensile strength should not exceed $k_4 f_{yk}$. The mean value of the stress in prestressing tendons should not exceed $k_5 f_{yk}$.

Note: The values of k_3, k_4 and k_5 for use in a Country may be found in its National Annex. The recommended values are 0,8, 1 and 0,75 respectively.

7.3 Crack control

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) Cracking is normal in reinforced concrete structures subject to bending, shear, torsion or tension resulting from either direct loading or restraint or 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) [AC1⟩ A limiting value, w_{max}, for the calculated crack width, w_k, taking into account ⟨AC1] 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} (mm)

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]
[AC2⟩ XD1, XD2, XD3, XS1, XS2, XS3 ⟨AC2]		Decompression

Note 1: For X0, XC1 exposure classes, crack width has no influence on durability and [AC1⟩ this limit is set to give generally acceptable appearance. In the absence ⟨AC1] 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 bonded 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 with the struts oriented according to the compressive stress trajectories in the uncracked state, it is possible to use the forces in the ties to obtain the corresponding steel stresses to estimate the crack width (see 5.6.4 (2).

(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).

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

where:

$A_{s,min}$ is the minimum area of reinforcing steel within the 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 or spacing (see 7.3.3 (2))

$f_{ct,eff}$ is 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
= 0,65 for 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 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:

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

- For flanges of box sections and T-sections:

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

where

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

$$\sigma_c = \frac{N_{Ed}}{bh} \qquad (7.4)$$

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

$k_1 = \frac{2h^*}{3h}$ if N_{Ed} is a tensile force

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 $A_{c,eff}$.

$A_{c,eff}$ is the effective area of concrete in tension surrounding the reinforcement or prestressing tendons 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).

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 prestressing 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 is compressed or the absolute value of the tensile stress in the concrete is below $\sigma_{ct,p}$.

Note: The value of $\sigma_{ct,p}$ for use in a Country may be found in its National Annex. The recommended value is $f_{ct,eff}$ in accordance with 7.3.2 (2).