conservative assumption, as part of this long-term deflection is likely to have occurred prior to the fixing of the partitions.

The equation for the limiting effective span to depth ratio (Equation 8.5.4) involves no approximations other than those implicit in the values selected for k_1 and k_2, and of course, the assumptions associated with the use of the deflection multiplier k_{cs}. However, the assumed value for I_{ef} (= $k_1 b_{ef} d^3$) is close to I_{cr} and this is generally conservative. With the member assumed to be heavily cracked, the use of k_{cs} = 2 will usually overestimate long-term deflection. With these assumptions, satisfaction of the limiting span-to-depth ratio will ensure in-service deflections less than the maximum desirable value, often significantly less than the maximum value.

C8.6 Crack control of beams

C8.6.1 General requirements

Reinforced concrete elements crack wherever the tensile stress in the concrete reaches the tensile strength of the concrete. Concrete tensile stress at any location in a concrete structure may be caused by a number of factors, including the applied loads, restrained shrinkage, temperature changes (including early-age cooling), support settlement and so on. Cracks formed by axial tensile forces and restrained shrinkage (direct tension cracks) often penetrate completely through a member. Cracks caused by bending (flexural cracks) occur at the tensile face when the extreme fibre tensile stress reaches the tensile strength of the concrete.

Flexural cracks propagate from the extreme tensile fibre through the tensile zone and are arrested at or near the neutral axis. Flexural cracks increase in width as the distance from the tensile reinforcement increases and tapers to zero width near the neutral axis. A linear relationship is generally assumed to exist between the crack width at the side or soffit of a member and the distance from the bar. In general, the spacing between flexural cracks is in the range 0.5 to 1.5 times the depth of the member.

Many variables influence the width and spacing of cracks in reinforced concrete members, including the magnitude and duration of loading, the quantity, orientation and distribution of the reinforcing steel, the cover to the reinforcement, the slip between the tensile reinforcement and the concrete in the vicinity of the crack (which depends on the bond characteristics of the reinforcement), the deformational properties of the concrete (including its creep and shrinkage characteristics) and the size of the member. Considerable variations exist in the crack width from crack to crack and the spacing between adjacent cracks because of random variations in the properties of concrete.

Restraint to shrinkage is provided by the bonded reinforcement in a reinforced concrete member, with the concrete compressing the reinforcement as it shrinks and the reinforcement imposing an equal and opposite tensile force on the concrete at the level of the steel. This internal tensile restraining force is often significant enough to cause time-dependent cracking. In addition, the connections of a concrete member to other parts of the structure or to the foundations also provide restraint to shrinkage. The tensile restraining force that develops rapidly with time at the restrained ends of the member usually leads to cracking, often within days of the commencement of drying. In a restrained flexural member, restraint to shrinkage causes a gradual widening of flexural cracks and a gradual build-up of tension in the uncracked regions, which may lead to additional cracking. The influence of shrinkage on flexural and direct tension crack widths should be considered in the design for crack control.

The requirements specified in Clause 8.6 have been adapted from the approach outlined in Eurocode 2 (Ref. 47). Irrespective of the importance of the structure, the maximum crack width requirements and the exposure condition, the following minimum reinforcement requirement and the maximum cover and bar spacing requirements specified in Clause 8.6.1 need to be satisfied:

(a) *Minimum reinforcement requirement* — The quantity of tensile reinforcement in a beam needs to be greater than that required to provide the minimum ultimate bending strength $(M_{uo})_{min}$ specified in Equation 8.1.6.1(1). The magnitude of $(M_{uo})_{min}$ is 20 % higher than the cracking moment specified in Clause 8.5.3.1, with σ_{cs} set to zero and $f'_{ct.f}$ taken as the characteristic tensile strength of the concrete (see Clause C8.1.6.1).

This requirement ensures that a lightly loaded member has an adequate reserve of strength if unexpected cracking occurs and applies to all tension members and flexural members.

(b) *Maximum cover and bar spacing requirements* — Crack widths increase as the distance from the reinforcing bar increases and crack widths on the concrete surface become aesthetically unacceptable when the concrete cover become too large. It is noted that the 100 mm limit applies to the axis distance and not to the clear concrete cover. Crack widths also depend on the proximity to the nearest bar and when bars are spaced further apart than 300 mm, crack control may be compromised.

To satisfy the maximum cover and spacing requirements, bars with a diameter less than half the diameter of the largest bar in the tensile zone are to be ignored, as they may not be effective in controlling cracking. In addition, for T-beams and L-beams, where the tensile zone is located in the flange (such as over an internal support in a beam and slab floor), the reinforcement required for strength and for deflection control is to be distributed across the full width of the effective flange. It is important for crack control, that the tensile reinforcement is well distributed across the full width of the tensile zone, so additional steel may be required in that part of the flange outside the outside the effective flange width.

In addition, the Standard places a maximum limit of $0.8f_{sy}$ on the stress in the tensile reinforcement at a cracked section ($\sigma_{scr.1}$) under the short-term serviceability load combination (calculated with $\psi_s = 1.0$). This limit applies irrespective of the diameter of the tensile reinforcing bars or the bar spacing. The applied moments at the serviceability limit state are normally estimated using elastic analysis. However, the Standard cautions that substantial errors may result where the actual in-service moments are likely to have redistributed significantly from the elastic distribution.

For crack control in beams that are fully enclosed within a building and sheltered from the environment (except for a brief period during construction) or in other situations where cracking will not impair the functioning of the structure, only the minimum reinforcement and maximum cover and bar spacing requirements of Clause 8.6.1 need to be satisfied. In other situations, flexural cracks may be controlled by calculation of the maximum crack width in accordance with Clause 8.6.2.3 (and Clause 16.4.7.4 for fibre reinforced concrete beams) and limiting the calculated value to the characteristic maximum crack width w'_{max}. Alternatively, flexural cracks may be controlled by limiting the tensile reinforcement stress to the maximum values specified in Clause 8.6.2.2 (for reinforced concrete sections and sections containing unbonded tendons) and Clause 8.6.3 for slabs containing bonded tendons.

Crack control therefore involves the selection of a maximum crack width w'_{max} appropriate for the concrete surface under consideration. For structures subjected to the long-term service loads, recommended values for w_{max} are as follows:

(i) For exposure classes A1 and A2, where crack width has no influence on durability, w'_{max} = 0.4 mm. This limit may be relaxed in situations where acceptable appearance is not required.

(ii) For exposure classes A1 and A2, where crack width may influence durability, w'_{max} = 0.3 mm.

(iii) For exposure classes B1 and B2, w'_{max} = 0.3 mm.

(iv) In aggressive soils and in coastal splash zones, w'_{max} = 0.2 mm.

It is noted that cracking is a variable process and that some cracks in the beam may exceed the value of w'_{max} selected in design.

The Concrete Institute of Australia's Recommended Practice Z7-06 (Ref. 60) provides design guidance for crack control in concrete structures.

C8.6.2 Crack control for tension and flexure in reinforced beams

C8.6.2.1 General

For members subjected to axial tension and bending, where the axial tension dominates and the whole of a particular cross-section is in tension, the Standard defines the resultant action primarily as tension. Where flexure predominates and the tensile stress distribution is triangular with some part of the cross-section in compression, the resultant action is defined primarily as flexure.

C8.6.2.2 Crack control without direct calculation of crack widths

The deemed-to-satisfy approach to crack control does not involve any direct calculation of maximum crack width. It involves limiting the stress in the bonded reinforcement crossing a crack to an appropriately low value. The limit on the tensile steel stress imposed in design depends on the maximum acceptable crack width.

For members primarily in tension, the calculated steel stress (σ_{scr}) on the cracked section caused by the serviceability design actions is not to exceed the maximum steel stress given in Tables 8.6.2.2(A). The specified maximum steel stress corresponding to the diameter of the largest bar in the tensile zone should be selected. For primarily tension members, cracking may therefore be controlled by selecting a bar diameter small enough to satisfy the requirements of Table 8.6.2.2(A).

For members primarily in flexure, the calculated steel stress (σ_{scr}) caused by the serviceability design moment is not to exceed the larger of the maximum steel stresses given in Tables 8.6.2.2(A) and 8.6.2.2(B). For flexural members, cracking may be controlled by selecting either a bar diameter small enough to satisfy the requirements of Table 8.6.2.2(A) or a centre-to-centre bar spacing small enough to satisfy the requirements of Table 8.6.2.2(B). When determining the centre-to-centre bar spacing, bars with a diameter less than half the diameter of the largest bar in the tensile zone should be ignored.

The calculated steel stress (σ_{scr}) is the steel stress on the cracked section due to the quasi-permanent service loads. When determining the steel stresses (σ_{scr} and $\sigma_{scr.1}$), the corresponding in-service bending moments should be calculated from an appropriate elastic analysis (linear or nonlinear). They should not be determined by scaling down from moments determined at the strength limit state, where plastic redistribution of moments may have been assumed.

The maximum steel stresses specified in Tables 8.6.2.2(A) and 8.6.2.2(B) have been determined using the crack width calculation procedure specified in Clause 8.6.2.3. The values in both tables have been determined for members primarily in tension assuming f_{ct} = 3.0 MPa, E_c = 28 000 MPa, E_s = 200 000 MPa, c = 40 mm, $(D\text{-}d) = c + d_b/2$, $h_{c,eff} = 2.5(D\text{-}d)$, φ_{cc} = 2.5 and ε_{cs} = 0.0005.

C8.6.2.3 Crack control by calculation of crack widths

The approach outlined here for the calculation of maximum crack width is a modified version of the procedure specified in Eurocode 2 (Ref. 50). It is a deterministic procedure that is intended to control cracking by limiting the calculated maximum crack width w to some appropriately low value (w'_{max}).

The maximum calculated crack width (w) is expressed in Equation 8.6.2.3(1) as the product of the maximum crack spacing $s_{r,max}$ and the difference between the mean strain in the reinforcement and the mean strain in the concrete between the cracks $\varepsilon_{sm} - \varepsilon_{cm}$. The mean strain in the reinforcement ε_{sm} at the design loads, includes the effects of tension stiffening and any imposed deformations. The mean strain in the concrete between the cracks ε_{cm} includes the mean tensile stress-related strain and the shrinkage strain. The mean tensile stress-related strain is caused by the tensile stress that develops between the cracks due to bond between the reinforcement and the tensile concrete and this is made up of elastic strain and tensile creep strain. The difference between the mean strain in the reinforcement and the mean strain in the concrete $\varepsilon_{sm} - \varepsilon_{cm}$ may be approximated by Equation 8.6.2.3(2).

The maximum crack spacing $s_{r,max}$ is affected by the quantity of tensile reinforcement, the bar diameter, the bar spacing, the concrete cover and tensile creep and shrinkage characteristics of the concrete and is given by Equation 8.6.2.3(3). The first term on the right-hand side of Equation 8.6.2.3(3) reflects the effect of cover, while the second term accounts for the quantity of reinforcement, the bar size and spacing and the reduction in the maximum crack spacing with time due to the formation of additional cracks due to on-going shrinkage. Equation 8.6.2.3(3) is taken from Eurocode 2 (Ref. 47), except that the constant 0.3 in the second term replaces the 0.425 in the Eurocode. The reduction accounts for the effect of shrinkage on maximum crack spacing (an effect that has not been considered in Eurocode 2).

Where a mixture of bar diameters is used in a section, an equivalent bar diameter, $d_{b.eq}$, may be used in the determination of $s_{r,max}$. The equivalent bar diameter is calculated using Equation 8.6.2.3(4).

Calculating crack widths using Equation 8.6.2.3(1) is approximate. The method approximates the maximum crack width, and makes an approximate allowance for tension stiffening. Designers should be aware that the maximum crack widths on-site may exceed the calculated value.

C8.6.3 Crack control for flexure in prestressed beams

When flexural cracking occurs in a prestressed concrete beam, the loss of stiffness is less sudden than for a reinforced concrete beam with the same area of tensile reinforcement, and the change in the tensile steel stress is relatively small. At first cracking, the axial prestressing force on the concrete controls the propagation of the crack, and unlike cracking in a reinforced concrete beam, the crack does not suddenly propagate over much of the cross-section and the change in strain at the tensile steel level is much smaller. Cracks become deeper only as the load increases and the loss of stiffness due to cracking is far more gradual than for a reinforced member.

Therefore after cracking, prestressed beams behave better than reinforced concrete beams, with less deformation and with finer, less extensive cracks. Flexural crack control in prestressed concrete beams is not usually a critical design consideration if bonded reinforcement is provided in the tensile zone.

If the maximum tensile stress in the concrete caused by the short-term service loads is less than $0.25\sqrt{f'_c}$, the section is considered uncracked and no further consideration needs to be given to crack control. When calculating the maximum tensile stress in the concrete, in addition to the stresses caused by the short-term service loads and the prestress, the loss of compressive stress in the concrete due to the restraint provided by the bonded reinforcement to creep and shrinkage deformations of the concrete should be considered.

If the maximum tensile concrete stress caused by the short-term service loads is above $0.25\sqrt{f'_c}$, then bonded reinforcement and/or bonded tendons are required to be provided near the tensile face with a centre-to-centre spacing not exceeding 300 mm. In addition, one of the following alternatives has to be satisfied:

(a) The calculated maximum flexural tensile stress caused by the short-term service loads at the extreme concrete tensile fibre is to be less than $0.6\sqrt{f'_c}$.

(b) The increment in the tensile stress in the steel near the tension face, as the applied load increases from its value when the extreme fibre is at zero stress (the decompression load) to the full short-term service load, is to not exceed the maximum value given in Table 8.6.3.

Crack control is deemed to be provided in Item (a), if the maximum tensile stress calculated on the uncracked transformed cross-section does not exceed the lower characteristic flexural tensile strength of the concrete. In this case, cracking may still occur, but the change in tensile concrete and steel strains will not be great and crack control will not be a problem, provided some bonded steel at a spacing less than 300 mm is located in the tensile zone.

The alternative provision for crack control in Item (b) is to limit the change in stress that occurs in the tensile steel due to cracking to a maximum value that depends on the diameter of the bonded reinforcement or tendons. At the decompression moment in a beam, the stress in the non-prestressed reinforcement will be compressive and so the final maximum tensile steel stress in a prestressed beam may be limited to a value that is less than the value that is permissible in a reinforced concrete beam.

Considering that prestressed beams generally perform better than reinforced beams after cracking, the deemed-to-satisfy crack control provisions for prestressed beams are generally conservative.

C8.6.4 Crack control in the side face of beams

The width of a flexural crack at the surface of the tensile reinforcement is usually very small, but as the distance from the tensile steel increases so too does the crack width. Where the depth of a beam exceeds 750 mm, the flexural cracks may become excessively wide on the side faces of beams in the mid-depth regions away from the longitudinal tensile reinforcement, unless some additional crack control reinforcement is provided in the side faces of the beam. The Standard specifies additional longitudinal reinforcement (in the form of 12 mm diameter bars at 200 mm centres or 16 mm diameter bars at 300 mm centres) to be placed in the side faces of such beams over the depth of the beam, and at least from the neutral axis of the cracked section to the level of the tensile reinforcement. The first of these side face bars should be located no further than 200 mm for 12 mm bars (or 300 mm for 16 mm bars) above the main tensile reinforcement.

This additional longitudinal side face reinforcement, together with the minimum transverse shear reinforcement, is not only considered adequate for flexural crack control on the side faces of beams, it will also assist in limiting the width of inclined shear cracks and any shrinkage-induced cracks in regions of low moment.

C8.6.5 Crack control at openings and discontinuities

Openings and discontinuities can be the cause of stress concentrations that may result in diagonal cracks emanating from re-entrant corners. Additional trimming bars are required at holes and discontinuities to control these cracks. A suitable method of estimating the number and size of the trimming bars is to postulate a possible crack and to provide reinforcement to carry a force at least equivalent to the area of the crack multiplied by the mean direct tensile strength of the concrete. For crack control, the maximum stress in the trimming bars should be limited to 250 MPa.

While additional reinforcement is required for serviceability to control cracking at re-entrant corners, it should not be assumed that this same steel is satisfactory for strength. When openings are located in the shear zone of beams, for example, the strength of the beam should be carefully calculated, as any contribution by the concrete to the shear capacity will be lost. Analysis using strut-and-tie modelling is a convenient method to visualize the flow of forces required to establish a viable load path. Appropriate reinforcement patterns should be detailed to achieve this load path and to provide adequate strength. Additional trimming bars may also be required for crack control under service loads.

C8.7 Vibration of beams

As outlined in Clause C2.3.4, vibration can usually be controlled by limiting the frequency of the fundamental mode of vibration of the structure to a value markedly different from the frequency of the source of vibration. Alternatively, and ideally, the structure and the source of vibration should be dynamically isolated from one another. If this is not possible, either the structure or the source (or both) may be suitably damped to reduce the magnitude of the structural vibrations to acceptable levels (Ref. 61).

The susceptibility of a beam to excessive levels of vibration depends on its physical properties, such as mass and frequency, and also upon the nature of the dynamic forces applied. For example, long-span lightweight beams are much more likely to experience excessive vibrations from pedestrian traffic than short-span relatively stocky beams. On the other hand, machinery placed on and supported by short-span beams may have an operational frequency close to the natural frequency of the beam, resulting in excessive vibration, while the same machine supported on a long-span beam may result in minimal vibration.

As a consequence, no simple design rules have been formulated to cover the range of possible situations. The designer is therefore referred to the list of references noted in Clause C2.3.4.

C8.8 T-beams and L-beams

C8.8.1 General

At the interface of the web and flange of both monolithic and isolated T-beams and L-beams, the longitudinal shear stress capacity ($\phi\tau_u$) should be not less than the design shear stress (τ^*). The capacity calculated in accordance with Clause 8.4.3 comprises contributions from the concrete and any fitments that cross the interface.

For isolated T-beams and L-beams where load is applied to the flange, an additional check should be made on the flexural shear capacity of the flange at the critical sections indicated in Figure C8.8.1.

If bars in slabs are anchored using cogs as shown in Figure C8.8.1, it is important to ensure that the slab thickness is sufficient to allow the standard cog for the bar size used (see Table C13.1.2.7) to be placed within the depth of the slab without encroaching on the specified cover.

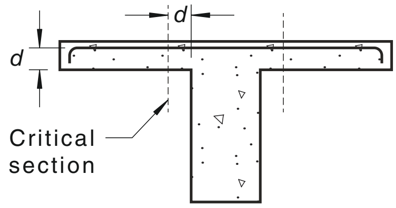

Figure C8.8.1 — Critical sections for flexural shear in the flange of isolated T- and L-beams

C8.8.2 Effective width of flange for strength and serviceability

T-sections and L-sections occur when a flange and web act together structurally, often as a result of being cast together (monolithic). Under positive bending, longitudinal compressive stresses are produced in the flange at the top of the cross-section. Because of shear lag, these stresses reduce in magnitude with the distance away from the web. To facilitate the design process, a width of flange is chosen over which the longitudinal stresses are assumed to be essentially constant. This effective width (b_{ef}), depends on the type of loading on the beam and various geometrical parameters. Transverse bending moments in the flange also affect the effective width.

For simplicity, a conservative effective width, which is constant along the span, is used in all strength and serviceability calculations. It is a function of the web width (b_w) and the distance a between the points of contraflexure along the beam. For simply supported beams, a may be taken as approximately equal to the span L, while for continuous beams, a may be taken as $0.7L$ for interior spans. For an end span of a continuous beam, Eurocode 2 (Ref. 47) gives the distance from the discontinuous support to the point of contraflexure (a) as approximately $0.85L$.

A maximum limit on the overhanging part of the effective flange equal to half the clear distance to the next member is specified.

C8.9 Slenderness limits for beams

A slender beam lacking lateral support is prone to lateral torsional buckling, if the flexural stiffness in the loaded plane of bending is very much greater than its lateral stiffness. Therefore adequate lateral restraint should be provided to ensure that the flexural capacity of the member is not reduced by buckling. Particular attention should be paid to any lateral eccentricity of loading that may cause bending about the weak axis or torsion. Care should be taken to minimize lateral bending in precast beams during handling operations.

In the beam's final *in situ* position, lateral restraint will normally be provided by construction attached to the compression zone of the beam. In the case of beams whose webs are upstanding, lateral restraint is provided by the slabs attached to the tension zone through the moment developed between the slab and beam.

The deemed-to-satisfy provisions in Clauses 8.9.2 and 8.9.3 are based on the provisions of CP110 (Ref. 62). Their development assumes that the beam is not subjected to a significant axial force, the ends of the beam are restrained against rotation, and in other than the case of pure bending, the loads are applied along the centre-line of the beam or at the centroid.

Simply supported and continuous reinforced concrete beams with $L_1 D / b_{ef}^2 \leq 60$ are not normally considered to be slender.

C8.10 References

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