C8.4.6 Minimum thickness of structural components

The Clause is particularly important for toppings. Thin toppings require careful curing and variations in thickness should be minimized.

C8.5 DEFLECTION OF BEAMS

(See AS 5100.2 Supp 1)

C8.5.1 General

(No Commentary)

C8.5.2 Beam deflection by refined calculation

The Clause is intended to provide for top tier methods based on estimated creep and shrinkage properties and the integration of curvatures to obtain the deflection. The design engineer is free to choose suitable procedures.

The following should be taken into consideration:

(a) *The expected shrinkage and creep properties of concrete* The effect of environmental influences on creep and shrinkage is often difficult to predict. Section 6 specifies shrinkage and creep properties of concrete for a range of environmental conditions [Warner 1973 (Ref. 22) and 1978 (Ref. 23); Wyche 1984 (Ref. 24)].

(b) *The expected load history* The loading used in the analysis should receive careful consideration.

    An aspect of the loading that has to be considered is the history or time sequence of loads. For the purpose of calculating the extent of cracking and hence tension stiffening, construction loading and early temperature and shrinkage stresses may be important. In general, the earlier the structure is loaded the greater will be the long term deflection.

    Two other load history factors that influence the deflection are the duration of the load and the age at first loading. Simple assumptions here may lead to very conservative results.

(c) *The effects of cracking and tension stiffening* Cracking of reinforced and partially prestressed concrete reduces the stiffness of the section; however, the onset and extent of cracking is difficult to predict. Construction loads may be applied on flexural members at a time when the concrete strength is below design requirements and cracking may result. In the application of the design methods, it is therefore recommended that unless better information exists, the effective moment of inertia should be based on the assumption that the member has been loaded to its maximum short-term service load or design construction load whichever is greater.

    There is also the possibility that significant cracking may be caused by factors that are not load dependent such as shrinkage and temperature. Severe cracking problems, caused by excessive early shrinkage associated with inadequate curing and rapid drying, have been observed even where the laboratory tests showed that the concrete did not have a high ultimate shrinkage.

    In the design process, it is recommended that due allowance be made for shrinkage, particularly for lightly reinforced sections which would otherwise be uncracked at service loads.

Tension stiffening is the phenomenon whereby the concrete between cracks contributes significantly to the stiffness of the section and any model for reinforced concrete must allow for this effect. [Bridge and Smith 1982 (Ref. 25); Clark and Spiers 1978 (Ref. 26); Gilbert and Warner 1978 (Ref. 27); Wyche 1984 (Ref. 24)].

Other secondary factors influencing deflection have been discussed by Beeby (1970) (Ref. 28). These are related to partial fixity of nominally simply supported members, increase in modulus of elasticity over calculated values, and similar effects.

C8.5.3 Beam deflection by simplified calculation

C8.5.3.1 *Short-term deflection*

The simplified rules for calculating deflections follow the Branson equation for effective second moment of area. (Branson 1968) (Ref. 29).

The effect of this equation on the calculated deflection of beams is shown in Figure C8.5.3.1 where typical moment deflection curves for reinforced and partially prestressed beams are given. Below the cracking moment, the gross transformed section properties govern the deflection, and for simplicity, the Standard permits use of the gross concrete section properties in this range.

For moments greater than the cracking moment, an empirical transition for I_{ef} is given by the Branson equation, where I_{ef} approaches I_{cr} as the service moment increases.

Conveniently, the Branson equation may conservatively be used for partially prestressed concrete (Warner 1978) (Ref. 23). The extra stiffness of this form of construction is reflected in the higher cracking moment.

The value of I_{ef} used in the Clause should relate to the section of the member that most influences the deflections.

A further problem exists with the value of M_s to be used in the calculation of I_{ef}. In the simple laboratory tests on which the equation was based, M_s represented the service load at which the deflection was calculated. In practice, loads higher than the short-term service load may have been encountered during construction. Consequently, the new clauses specify that M_s be calculated using the short-term service load or design construction load, whichever is greater.

It seems prudent to make some allowance for restrained shrinkage on the cracking moment. This allowance obviates the inconsistency of lightly reinforced sections being regarded as uncracked for deflection computations, whereas the combination of flexural and shrinkage stresses could induce cracking, thus significantly reducing the stiffness of such sections.

For heavily reinforced sections, the problem is not so significant, as the service loads are usually well in excess of the cracking load and the cracked stiffness is closer to the gross stiffness. Therefore, for lightly reinforced sections, some allowance should be made for shrinkage on the cracking moment. This approach may be conservative as an allowance for shrinkage is already included in the long-term deflection multiplier. However, experience has indicated initial cracking may be a more serious problem than would have been encountered in laboratory tests. Thus an upper limit on I_{ef} of $0.6I$ is recommended for lightly reinforced sections (Gilbert 1983) (Ref. 30).

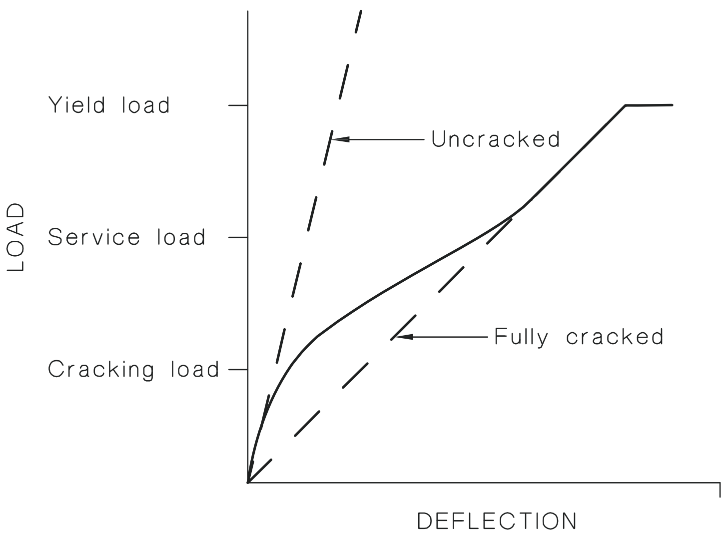

FIGURE C8.5.3.1 LOAD DEFLECTION CURVE OF A REINFORCED CONCRETE BEAM UNDER SHORT TERM LOADING

C8.5.3.2 *Long-term deflection for beams cracked under permanent loads*

The Clause applies primarily to reinforced beams. The long-term deflection multiplier for creep and shrinkage in a reinforced beam (k_{cs}) is derived from laboratory tests that cannot take account of the variable conditions to which the structures are exposed in service. The simple multiplier technique should, therefore, only be seen as an approximate predictor of final deflection and not as a complete guide to actual behaviour.

Long-term deflection for beams uncracked under permanent loads apply primarily to prestressed concrete beams. Long-term deflections are calculated from shrinkage effects and from creep of the concrete under permanent loads. Changes in permanent loads, time of change and duration of loads will affect the long-term deflection and have to be taken into account when determining the creep coefficients.

For partially prestressed beams this multiplier method should be used with caution as shrinkage and creep can have a large effect on the deflection.

C8.6 CRACK CONTROL OF BEAMS

C8.6.1 Crack control for tension and flexure in reinforced beams

The introduction of higher grade reinforcement into the Standard has highlighted the need to include better crack control design provisions.

These provision, based on the method used in Eurocode 2 (2005) (Ref. 13) endeavour to limit crack widths to acceptable levels.

Concrete will crack whenever the tensile strength of the concrete is exceeded. As the width of these cracks affects serviceability in both deflection and durability, they should be considered in design. It is essential that the cracks form in a well-distributed pattern and that they do not become excessively wide under service loads.

While cracking may be due to direct loading, imposed deformations or restrained deformations, flexural cracking from direct loading is primarily considered. In Eurocode 2 (2005) (Ref. 13), it is intended that a crack width limit of 0.3 mm is a characteristic value with only a 5% probability of being exceeded. Where tighter control of cracking is sought, tighter controls than the Standard requirements may be needed.

Item (a) of the Clause gives the minimum area of reinforcement in a tensile zone. In areas where crack control is critical for aesthetic or durability reasons, this minimum reinforcement is required. In other areas, these minimum requirements may be waived.

The requirements for limiting steel stresses to control cracking under direct loads are detailed in the Clause. Limits are based on either the bar diameter or spacing and so there is considerable scope to minimize the amount of additional reinforcement needed for crack control.

Wheeler and Patrick (2001) (Ref. 31) and Wheeler, Patrick and Bridge (Ref. 32) give an outline of the requirements for AS 3600 including its application to a T-beam and a slab.

C8.6.2 Crack control for flexure in prestressed beams

Item (a) *Monolithic beams*

The Clause makes provision for both prestressed and partially prestressed beams and includes simple alternatives.

If the tensile stress in the concrete is less than $0.25\sqrt{f'_c}$, the section is considered uncracked and no further check is needed.

If the stress is greater than $0.25\sqrt{f'_c}$ then bonded reinforcement, which can include tendons, should be provided near the tensile face. Since crack control is proportional to cover and spacing, the smaller the cover and closer the spacing of such reinforcement the better the control, although the Standard provides no specific rule.

Further control of crack widths relies on limiting the concrete or steel stress. It is considered that a concrete tensile stress, based on the uncracked section, is the lower limit for significant cracks.

An alternative provision allows for a stress of 200 MPa resulting from an increment of moment from the decompression moment. This requires that the decompression moment for zero tensile stress be calculated. The steel stress caused by the excess of the service moment over this decompression moment is then limited to 200 MPa. This gives rise to tensile strains at the level of the steel of 1000×10^{-6} which requires a higher level of crack control. This is provided by the requirement that the reinforcement spacing be limited to that for a non-prestressed beam, thus giving 'cover' controlled cracks.

Item (b) *Segmental members at unreinforced joints*

The Clause applies to prestressed segmental members with no unstressed reinforcement across the tensile face of the joint. With no reinforcement to distribute cracks, large crack widths can result at the joints which may affect the integrity of the structure at the joint and affect shear transfer between segments.

For railway bridges, a residual compression should be provided at the unreinforced joints of segmental members at the serviceability limit state. The residual compression is to ensure cracking will not occur under slightly overloaded trains.

Item (c) *Prestressed members in exposure classifications B2, C or U*

Because of the high rate of corrosion that occurs in prestressing tendons, no tensile stress is allowed at the level of the tendon under serviceability loads other those likely to occur for a short time.

High tensile steel is more susceptible to rapid deterioration with little warning than normal reinforcement and it is therefore considered prudent to minimize cracks that may open frequently where prestressed elements are located in an aggressive environment.

Item (d) *Railway bridges*

For reinforced beams of railway bridges, the limits on tensile stress range will have the effect of limiting cracking.

C8.6.3 Crack control in the side face of beams

Clause 2.4.3 of the Standard provides minimum reinforcement for all concrete surfaces to limit cracking due to shrinkage and other causes.

C8.6.4 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. Often, only nominal reinforcement will be needed. A suitable method of estimating the size of the bars is to postulate a possible crack and to provide reinforcement at least equivalent to the area of the crack multiplied by the tensile strength of the concrete (Beeby 1970) (Ref. 28).

Openings in the shear zone of beams should be treated with caution, as any contribution by the concrete to the shear capacity may be considered dubious if openings exist. Some guidance for reinforcement patterns may be found from the force patterns of the truss analogy.

C8.7 VIBRATION OF BEAMS

(See AS 5100.C2.)

C8.8 PROPERTIES OF BEAMS

C8.8.1 General

The equations for the calculation of effective width of flange for strength and serviceability have been adopted from the CEB-FIP Model Code (1978) (Ref. 15). The effective widths calculated by the formulas are smaller than the values given in AS 3600. For the flexural strength of a T-beam or L-beam, the concrete in the flange has no effect when the flange is in tension (negative moment regions) and has little effect when the flange is in compression (positive moment regions). On the other hand, the flange width has a significant influence on the flexural stiffness of the beam and hence on deflections. Test results have shown that the effective width of flange as given in AS 3600 may be too large for use in stiffness calculations. For this reason, smaller values, consistent with Austroads Bridge Code, Section 5, 1992 (Ref. 33) have been retained. It should be noted that, unlike flexural strength, the concrete in a tensile flange will increase the cracking moment and therefore affect the overall bending stiffness of a T-beam or L-beam.

C8.8.2 Effective width of flange for analysis for serviceability

The equations for the calculation of effective width of flange for serviceability have been adopted from the CEB-FIP (1978) (Ref. 15).

C8.8.3 Effective width of flange for analysis for strength

At the ultimate limit state, plastic concrete behaviour will ensure that the entire concrete cross-section contributes.

C8.9 SLENDERNESS LIMITS FOR BEAMS

C8.9.1 General

The limits on the distance between points of lateral restraint are provided to guard against lateral buckling and consequent premature failure. Lateral eccentricity of loading causing torsion in slender laterally unbraced beams may be a problem; however, tests [(Hansell and Winter, 1959 (Ref. 34) and Sant and Bletzacker, 1961 (Ref. 35)] indicate that lateral buckling is unlikely to be a problem in beams loaded with no lateral eccentricity.

C8.9.2 Simply supported and continuous beams

(No Commentary)

C8.9.3 Cantilever beams

(No Commentary)

C8.9.4 Additional reinforcement for prestressed beams

(No Commentary)

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SECTION C9 DESIGN OF SLABS FOR STRENGTH AND SERVICEABILITY

C9.1 STRENGTH OF SLABS IN BENDING

C9.1.1 General

As most slabs in bridge design are considered as one-way slabs, the Clauses are limited to provisions for one-way slab design. Where two-way slab design is required, the provisions of AS 3600 may be used.

C9.1.2 Distribution reinforcement for slabs

Distribution reinforcement is required in all slabs for distribution of concentrated loads. If detailed analysis is not carried out, the amount of reinforcement to be used is stated as a percentage of the main reinforcement area. These values are taken from NAASRA (1976) (Ref. 1).

C9.1.3 Edge stiffening

At an edge or end of a slab, distribution of loads is restricted by the discontinuity of the slab. Hence, the edge or end of the slab has to carry a more concentrated load than the slab section away from the edges, and an edge beam or diaphragm provides the additional strength required.

The specific nature of rail loadings makes the general requirements of slabs for road loadings inappropriate to railway bridges. Possible occasional loadings, for example during construction or during maintenance, nevertheless need to be considered.

C9.1.4 Minimum thickness of deck slabs

A practical limit is imposed.

C9.2 STRENGTH OF SLABS IN SHEAR

C9.2.1 Application

Shear failure can occur in two different modes—

(a) a slab could act as a wide beam and fail in beam-type shear; and

(b) a slab could fail by 'punching' type shear along a truncated cone or pyramid around the support or loaded area. In this mode of failure, the extent of bending moment transferred from the slab to the support has an influence on the design.

C9.2.2 Design shear strength of slabs

(No Commentary)

C9.2.3 Shear strength of slabs without moment transfer

In most bridge designs, moments are not transferred from slabs directly to the supports, and hence equations are given in the Clause for the following cases:

(a) The equation has been adopted from the ACI 318-83 (Ref. 2) and assumes shear stresses are distributed uniformly around a critical perimeter and that failure occurs when these stresses reach a value equal to—

$$\left(f_{cv} + 0.3\sigma_{cp}\right)$$

(b) Where shear reinforcement or a shear head is provided so that shear failure will not occur within the shear head or the reinforced area, the value of f_{cv} is taken as $0.5\sqrt{f'_c}$. The upper limit on V_{uo} in this case avoids crushing failure.

When calculating the value of d_{om}, the geometry of the assumed critical shear surface needs to be taken into account. In many cases, it will be easier to calculate the effective area of the critical shear surface (ud_{om}) as the sum of a number of simple rectangular areas [$\Sigma(u_i d_{omi})$] rather than calculating u and d_{om} separately.

C9.2.4 Shear strength of slabs with moment transfer

For structures where slabs transfer moments to the supports, the relevant clauses of AS 3600 may be used.

C9.3 DEFLECTION OF SLABS

C9.3.1 General

A two-tiered approach is adopted for deflection control of slabs. Deflections may be calculated by refined methods for all slabs or by simplified methods for one-way slabs.

C9.3.2 Slab deflection by refined calculation

Methods for the calculation of slab deflection by refined methods range from complex, non-linear, finite element models [Gilbert and Warner 1978 (Ref. 3); Gilbert 1979 (Ref. 4); Scanlon and Murray 1984 (Ref. 5)] to more approximate methods [Nilson and Walters 1975 (Ref. 6); Vanderbilt et al 1963 (Ref. 7); Rangan 1976 (Ref. 8)].

Account should be taken of two-way action, the time-dependent effects of creep and shrinkage, the expected load history and cracking and tension stiffening (see also Clause C8.5.2).

C9.3.3 Slab deflection by simplified calculation

One-way slabs may be considered as wide beams and deflections calculated by beam deflection methods of Clause 8.5.3.

C9.4 CRACK CONTROL OF SLABS

The Standard gives only specific detailing rules as a means of controlling cracking in slabs; however, the calculation of crack widths may be used as an alternative procedure in controlling cracking (see also Clause C8.6).

C9.4.1 Crack control for flexure in reinforced slabs

As for beams, the provisions for slabs are based on the method used in Eurocode 2 (2005) (Ref. 9).

The minimum area of reinforcement given in Item (a) of the Clause is required in the critical tensile zone. A critical tensile zone is defined as a region of a slab where the design bending moment at the serviceability limit state $\left(M^*_{s1}\right)$ is greater than or equal to the critical moment for flexural cracking (M_{crit}) calculated assuming a flexural tensile strength of concrete equal to 3.0 MPa. Wheeler and Patrick (2001) (Ref. 10) discuss details of critical tensile zones.

C9.4.2 Crack control for flexure in prestressed slabs

See Clause C8.6.2. Note that the limit on increment in steel stress is 150 for slabs compared with 200 MPa for beams, and reflects the different bond resistance of slab ducts and beam ducts.

Where distribution reinforcement in a prestressed skew slab is placed on the skew, the angle of skew in relation to the direction normal to the main reinforcement should be limited to 30° to avoid concentration of cracking, unless other measures are provided.

The requirements of Clause 8.6.2(c) apply equally to beams and slabs.

C9.4.3 Crack control for shrinkage and temperature effects

Clause 2.4.3 of the Standard provides minimum reinforcement for all concrete surfaces to control cracking due to shrinkage and other causes.

C9.4.4 Reinforcement for restrained slabs

Where slabs are restrained from expanding and contracting, a minimum area of reinforcement is required for crack control.

C9.4.5 Crack control at openings and discontinuities

(See Clause C8.6.5.)

C9.4.6 Crack control in the vicinity of restraints

Account has to be taken of the stress distribution in the vicinity of restraints. Consideration should be given to strain compatibility to ensure adequate reinforcement is provided to control cracking.

C9.5 VIBRATION OF SLABS

(See Clause C8.7.)

C9.6 MOMENT RESISTING WIDTH FOR ONE-WAY SLABS SUPPORTING CONCENTRATED LOADS

(No Commentary)

C9.7 LONGITUDINAL SHEAR IN SLABS

(See Clause C8.4.)

C9.8 FATIGUE OF SLABS

The design engineer should be aware that provisions of the Clause may override requirements of Clause 9.4 of the Standard.

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