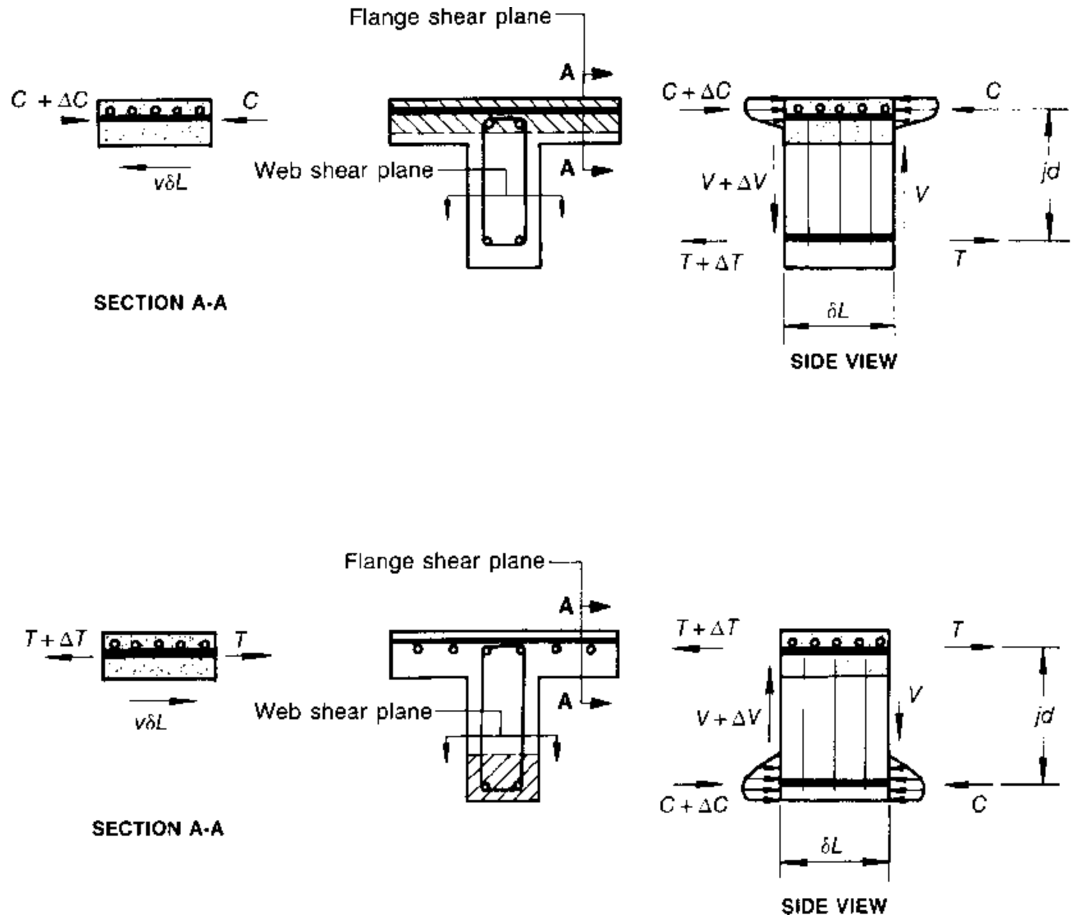


FIGURE C8.4.2 SHEAR FORCE RELATIONSHIPS

## C8.5 DEFLECTION OF BEAMS

**INTRODUCTION** Design and construction for deflection control is far more complex than strength design and there is no simple mathematical solution to the problem. The loading, both in magnitude and time of application and duration, is highly variable. The effects of creep and shrinkage and early age cracking are also difficult to predict. Moreover, the approach of making a conservative assessment of each of these parameters can lead to an overly conservative design (Refs 27 and 28). To design effectively for serviceability, the designer must have an understanding of the non-linear behaviour of concrete structures.

**C8.5.1 General** Serviceability problems, arising from shortcomings in the information given in Section 10 of AS 1480, created a need to revise this part of the Standard. Changes have been made in the span-to-depth ratios and in deflection limitations given in Clause 2.4.2 to reduce the likelihood of excessive deflections of flexural members. However, the use of these procedures without a critical assessment of the variables used may not eliminate serviceability problems.

**C8.5.2 Beam deflection by refined calculation** This Clause provides for refined methods, based on estimated creep and shrinkage properties and the integration of curvatures, to obtain the deflection. The designer is free to choose suitable procedures.

(a) *The expected shrinkage and creep properties of concrete* (Refs 44 and 45). The effect of the environment on creep and shrinkage is often difficult to predict. However, guidance is given in Section 6 as to the expected shrinkage and creep properties of concrete for a range of environmental conditions.

(b) *Loading and loading history* The loading used in the analysis should receive careful consideration (Refs 29, 30, 31, and 33). Certain provisions are made for this aspect of serviceability loads in Clause 3.4.

If a partition is built on top of a member, the long-term deflection may cause the member to creep away from the partition. The partition may be left spanning as a self-supporting deep beam which will apply significant loads to the supporting member only at its ends. Thus, if a partition wall is built over the whole span of a member with no major openings near its centre, some of its weight may be ignored in calculating long-term deflections.

A further aspect of the loading that must 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 which 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.

Cracking resulting from a combination of shrinkage and temperature changes is not an uncommon phenomenon in roof slabs which are exposed to direct sunlight. A sudden drop in temperature can add to the tensile strain caused by shrinkage and produce cracking during the construction at a time when the tensile strength of the concrete is low.

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 (Refs 37, 38 and 46) 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.

Other secondary factors influencing deflection have been discussed by Beeby (Ref. 47). 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** *Immediate deflection* The simplified rules for calculating deflections follow ACI and AS 1480 precedents in recommending the Branson equation (Ref. 36) for effective second moment of area.

The effect of this equation on the calculated deflection of beams is illustrated 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.

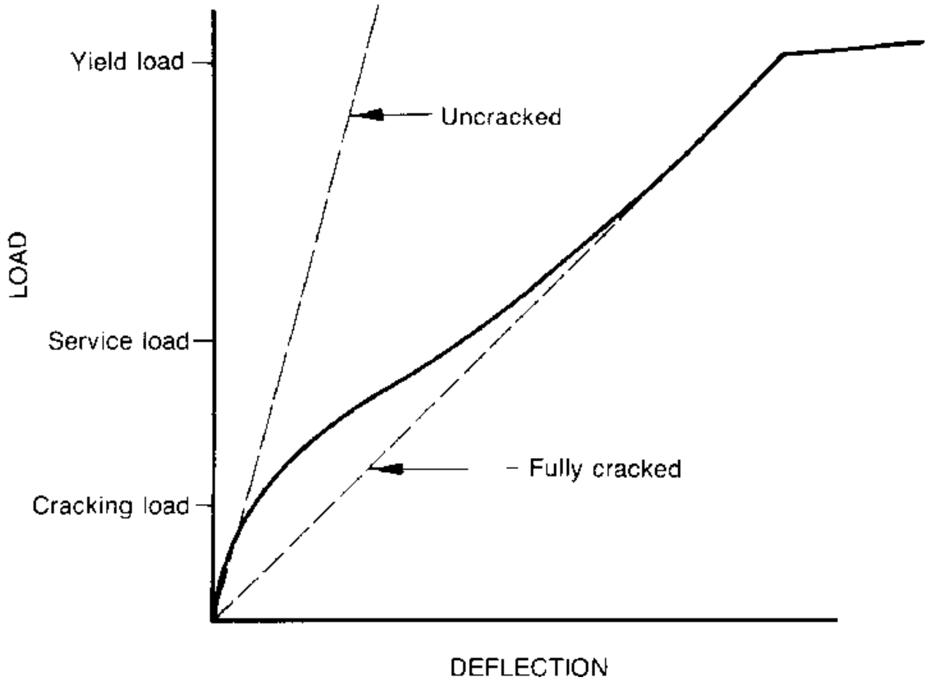


FIGURE C8.5.3.1 LOAD-DEFLECTION CURVE OF A REINFORCED CONCRETE BEAM UNDER SHORT-TERM LOADING

This approach has been found to give reasonable predictions of immediate deflections. The scatter of results is quite high (standard deviation of 40%) (Ref. 51), but considering the complexity of the tension stiffening problem, this may be regarded as satisfactory.

Conveniently, the Branson equation may conservatively be used for partially prestressed concrete (Ref. 45). The extra stiffness of this form of construction is reflected in the higher cracking moment.

The value of $I_{ef}$ used in this Clause should relate to the section of the member that most influences the deflections. The refinement of the approach given in AS 1480 did not seem warranted and a simplified procedure is given.

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 this formula 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 the structure's history. This is quite likely 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.

The basis of calculating the cracking moment ($M_{cr}$) for reinforced sections, using the section modulus and a nominal tensile strength, remains unchanged from AS 1480. For partially prestressed concrete, an allowance is made for the effect of prestress on $M_{cr}$. In addition, 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 the effect of 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 (Ref. 41). As a further simplification, $I_{cr}$ may be approximated by 5 to 6 times $A_{st}d^2$.

For reinforced members an alternative simplified expression for calculating $I_{ef}$ is given. For rectangular sections the value of $I_{ef}$ is approximately equal to $0.5I_g$, which is considered to be very conservative for beams but a good approximation for slabs.

For T and L sections an extra multiplier, $(0.7 + 0.3b_w/b_{ef})^3$, is introduced. This is a crude allowance for the decreased tension stiffening due to the smaller amount of material below the neutral axis when compared with a rectangular beam.

**C8.5.3.2** *Long-term deflection* For reinforced concrete it is convenient to use the simple deflection multiplier given in Clause 8.5.3.3. but this is not appropriate for beams with stressed tendons. For such cases, shrinkage, warping and creep must be calculated separately in accordance with the laws of mechanics and realistic assumed shrinkage and creep behaviour.

**C8.5.3.3** *Multiplier method for long-term deflection of reinforced beams* The long-term deflection multiplier for creep and shrinkage in a reinforced beam, ($k_{cs}$), is essentially the same as in AS 1480 with a simplification in the calculation of the ratio $A_{sc}/A_{st}$.

The long-term deflection multiplier is derived from laboratory tests and although an adequate but crude predictor of the long-term deflection (Ref.51) it has some short-comings.

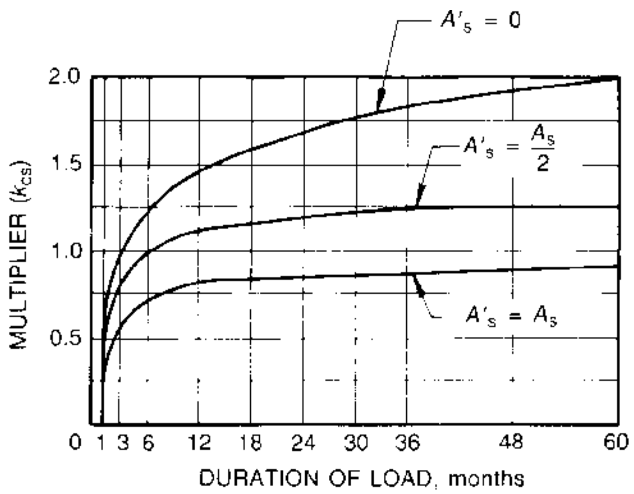


FIGURE C8.5.3.3 MULTIPLIERS FOR LONG-TERM DEFLECTIONS OF REINFORCED BEAMS

Laboratory tests for deflections are often conducted under constant load and environmental conditions. For the long-term deflection, the usual approach is to use a multiplier, based on the experimentally observed ratio of long-term to initial deflection. While this approach may give fair agreement with test data it does not reflect the variable conditions to which 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. Where local conditions indicate that severe creep or shrinkage effects exist, a larger value of $k_{cs}$ than that given above should be used, or the more general method of Clause 8.5.2 should be employed.

**C8.5.4 Deemed-to-comply span-to-depth ratios for reinforced beams** This is a new approach (Refs 41 and 42) based on a model proposed by Rangan (Ref. 43).

The maximum deflection of a beam under the action of a uniformly distributed load is usually expressed in the form:

$$\Delta = \frac{k_2(F_{d.ef})L_{ef}^4}{E_c\, I_{ef}}$$

where $F_{d.ef}$ is in N/mm, $L_{ef}$ is in millimetres and $k_2$ is the appropriate deflection constant derived from elementary principles. For example, for a simply supported beam $k_2$ is 5/384. For a continuous beam $k_2$ depends on the relative stiffness of the spans and on the loading pattern but for more or less uniform spans and where the loading is reasonably uniform, the values are assumed to be:

$k_2$ = 1/185 in an end span (propped cantilever).

$k_2$ = 1/384 in an interior span (fully fixed ends).

The Standard permits these values to be used where the live load does not exceed the dead load and where the ratio of longer to shorter spans does not exceed 1.2. For other situations, an elastic analysis will produce the required coefficient.

In the above equation, if the effective moment of inertia is replaced by—

$$I_{ef} = k_1\, b_{ef}\, d^3$$

then the design form of the equation becomes—

$$L_{ef}/d = [k_1(\Delta/L_{ef})\, b_{ef}E_c/(k_2\, F_{d.ef})]^{1/3}.$$

Thus this equation involves no approximations other than those implicit in the values selected for $k_1$ and $k_2$.

Values for $k_2$ can be obtained from an elastic analysis as noted above and values of $k_1$ can be obtained from Clause 8.5.3.1. Thus the accuracy of the estimate of $L_e$/d given by the equation depends only upon the accuracy adopted in determining $k_1$ and $k_2$. It should be noted that the designer nominates a suitable value of Δ for the member.

The effective design load, $F_{d.ef}$, is given for calculating both the total deflection and the deflection which occurs after the attachment of partitions (incremental deflection) taking into account the short-term and long-term serviceability loads given in Clause 3.4 and the long-term deflection multiplier given in Clause 8.5.3.3. The effective design load for incremental deflection assumes that the total long-term deflection due to creep and shrinkage under dead load occurs after the attachment of the partitions. This is a conservative assumption as part of this long-term deflection is likely to have occurred prior to the fixing of the partitions. Figure C8.5.3.3 and ACI 318 (Ref. 2) give an indication of the time dependency of the multiplier $k_{cs.}$

**C8.6 CRACK CONTROL OF BEAMS** The Standard only gives specific detailing rules as a means of controlling cracking in beams. However, using the provision of Clause 1.3 of the Standard, the calculation of crack widths can be used as an alternative procedure in controlling cracking. Accepted procedures would include the Gergely-Lutz equation adopted by the ACI 318 Code (Ref. 2) and the method given in BS 8110:Part 2 (Ref. 56).

The width of the flexural crack depends primarily on three factors: the proximity to the point considered of reinforcing bars perpendicular to the cracks; the surface strain at the point; and the proximity of the neutral axis to the point. The designer should therefore aim to minimize the cover and distance between bars to control flexural crack widths.

**C8.6.1 Crack control for flexure in reinforced beams** This Clause comes from AS 1480 which was based on BS CP110 and in turn on the work of Beeby (Ref. 47). Some discrepancy occurred in the transfer and the values in AS 1480 were too high. This has now been remedied and the centre-to-centre spacing is restricted to 200 mm while the distance from the bottom or side face to the centre of the bar is limited to 100 mm.

**C8.6.2 Crack control for flexure in prestressed beams** This 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. The tensile stress limit is taken from the working stress provisions of AS 1481.

If the stress is above $0.25\sqrt{f'_c}$ then bonded reinforcement, which can include tendons, must 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 of $0.6\sqrt{f'_c}$, based on the uncracked section, is the lower limit for significant cracks. This is approximately equivalent to a strain of $100 \times 10^{-6}$.

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 $1000 \times 10^{-6}$ and clearly 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.

**C8.6.3 Crack control in the side face of beams** Flexural cracks may become excessively wide on the side faces of beams in the mid-depth regions away from the longitudinal tensile reinforcement. The additional longitudinal reinforcement together with the minimum transverse shear reinforcement is considered adequate for flexural crack control on the side faces of beams. It will also limit the width of any shrinkage induced cracking 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. 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 (Ref. 47).

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** 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, the structure and the source should be dynamically isolated from one another if possible, or one or both suitably damped to reduce the magnitude of the structural vibrations to acceptable levels (Ref. 48).

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 floors are much more likely to experience excessive vibrations from pedestrian traffic than short-span relatively thick floors. On the other hand, machinery placed on short-span floors may have an operational frequency close to the natural frequency of the slab, resulting in excessive vibration, while the same machine on a long-span floor may result in minimal vibration.

As a consequence no simple design rules can be formulated. The designer, is therefore referred to the list of references noted in Clause C2.4.5.

**C8.8 T-BEAMS AND L-BEAMS** The equations for the calculation of effective width of flange for strength and serviceability have been adopted from the CEB Model Code (Ref. 17). The effective widths calculated by the formulas are smaller than the values given in Clause 9.7.2 of AS 1480. 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 available (Refs 49 and 50) have shown that the effective width of flange as given in AS 1480 may be too large for use in stiffness calculations. For this reason, smaller values are given. 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.

**C8.9 SLENDERNESS LIMITS FOR BEAMS** 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 (Refs 52 and 53) indicate that lateral buckling is unlikely to be a problem in beams loaded with no lateral eccentricity.

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