

Reinforced Concrete Buildings Series

Design Booklet RCB-2.1(1)

Crack Control of Slabs

Part 1: AS 3600 Design

**OneSteel Reinforcing
Guide to Reinforced Concrete Design**

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Preface

This design booklet is a part of OneSteel Reinforcing's Guide to Reinforced Concrete Design that has been produced to promote the superiority of OneSteel Reinforcing's reinforcing steels, products and technical support services. The Guide covers important issues concerning the design and detailing of Reinforced Concrete Buildings, Residential Slabs and Footings, and Concrete Pavements. The use of 500PLUS[®] reinforcing steels is featured in the booklets. Special attention is given to showing how to get the most benefit from these new, superior high-strength steels.

The design booklets of the Reinforced Concrete Buildings Series have each been written to form two separate parts: Part 1- AS 3600 Design which provides insight into major new developments in AS 3600; and Part 2 – Advanced Design[™] Using 500PLUS[®] which leads to significant economic advantages for specifiers of OneSteel reinforcing steel. These booklets are supported by 500PLUS computer software that will prove to be indispensable to design engineers who use AS 3600.

To control flexural cracking in slabs, the Concrete Structures Standard AS 3600-1994 required only the maximum spacing of tension reinforcement to be limited, and often this did not guarantee acceptably narrow cracks. The new edition of the Concrete Structures Standard AS 3600-2000¹ will allow 500 MPa reinforcing steels to be used in design. This will inevitably lead to higher steel stresses under serviceability conditions, thereby increasing the importance of designing for crack control under flexural conditions. New design provisions proposed for inclusion in AS 3600-2000, for crack control of slabs in a state of flexure, are reviewed in this design booklet. They have essentially come from Eurocode 2, and their use needs to be well understood by designers in order to design more serviceable reinforced-concrete structures, and also to allow the full benefit of the increase in steel yield strength to be gained, leading to a significant reduction in steel area. This may require judicious detailing of the reinforcing bars or mesh, crack control improving with a reduction in either bar diameter or bar spacing. A computer program 500PLUS-SCC[™] (SCC standing for Slab Crack Control) is being released with this design booklet, and is the second from the 500PLUS software suite. Section 6 of this booklet is effectively the User Guide for the program. Further research is proceeding that will allow these design provisions to be improved upon when using 500PLUS Rebar, OneMesh500[™], BAMTEC[®] and other OneSteel Reinforcing products and systems, and more advanced rules will be found in Part 2² of this design booklet.

¹ AS 3600-2000 is expected to be published this year by Standards Australia.

² In course of preparation.

1. SCOPE AND GENERAL

1.1 Scope

This design booklet is concerned with the control of flexural cracking in reinforced-concrete slabs designed in accordance with AS 3600-2000 (see Appendix A, Referenced Australian Standards). It provides rules essential for being able to efficiently detail the flexural reinforcement. The need for these rules is largely the result of including 500 MPa as a standard strength grade in AS 3600-2000. This is a significant increase over the previous strength grades of 400 MPa for bars and 450 MPa for welded mesh. However, experience is showing that improvements to procedures for controlling flexural cracking in slabs are required in any case.

Deformed reinforcement must be used, either separately as bars, formed into a welded mesh, or a combination of both. The reinforcement must have a rib geometry that satisfies AS 1302 for 400 MPa bar (Ductility Class N, 400Y) or AS 1303 for 450 MPa mesh (Class L, 450F). Alternatively, for 500 MPa reinforcement it must be either 500PLUS[®] Rebar (Class N) or OneMesh500[™] (Class L). It follows that the design yield strength of reinforcing steel that may be used in a slab designed in accordance with AS 3600-2000 is 400, 450 or 500 MPa.

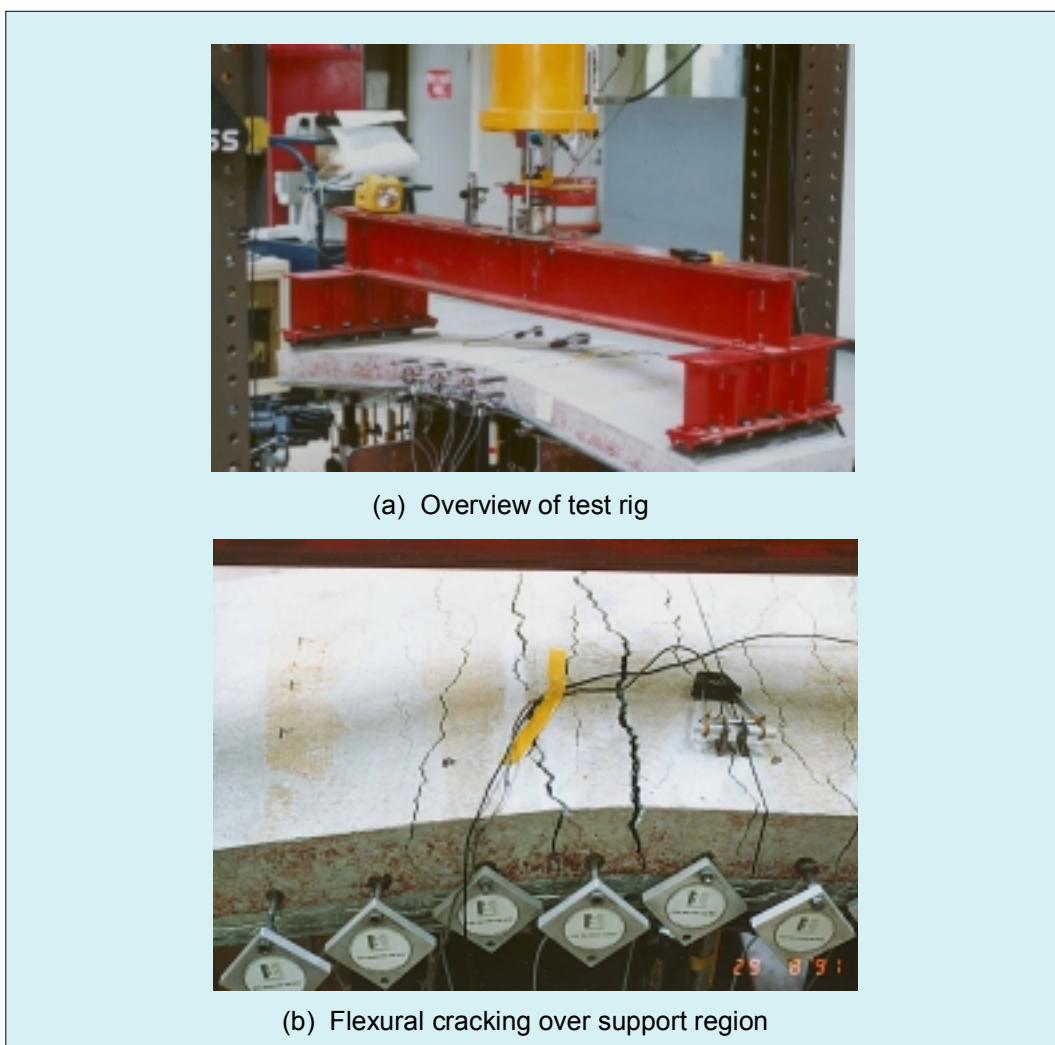


Figure 1.1 Flexural Cracking in Reinforced-Concrete Slabs

Rules for controlling flexural cracking due to direct loading arising from the self-weight of component elements and externally applied loads (see Fig. 1.1), imposed deformation (e.g. settlement of supports) or restrained deformation (e.g. concrete shrinkage) are covered. Controlling cracking

under imposed or restrained deformation may require the use of articulation joints, particularly if the location of stiffening elements (walls and cores) is undesirable in this regard. The use of these joints is beyond the scope of this booklet, but guidance is available in the literature [1,2].

In situations when bending is the main action effect, flexural cracks will form. These cracks appear in the tension face, and under service conditions extend more or less vertically in the regions over interior supports and in mid-span regions of continuous slabs. Flexure-shear cracks form in regions adjacent to the flexural cracks where the shear force is more significant. The flexure-shear cracks form from short vertical flexural cracks, but become inclined. The design rules contained herein for flexural elements are intended to control the width of both of these types of cracks. Cracking usually near the level of the elastic neutral axis, due to diagonal tension caused when shear force is the dominant action effect, is not addressed, although some guidance on this subject is given in Eurocode 2 [3]. Of course, controlling the width of flexure-shear cracks and diagonal tension cracks is only relevant when a slab has an exposed side.

Special issues relating to the calculation of steel stresses in non-flexural members, such as slabs on ground, preclude this booklet being used directly to design these elements for crack control.

Crack control around vertical openings is not covered. Neither are the effects of discontinuities such as at recesses and local changes in depth considered. In such situations, the reinforcement should in general exceed the minimum requirements specified herein.

1.2 General

The design rules presented herein are based on new rules in Eurocode 2 for crack control. The normal strength grade for reinforcement in Eurocode 2 is 500 MPa, which will be permitted in AS 3600-2000, and Eurocode 2 is currently the most appropriate design document to form a basis on which to develop Australian rules [4,5]. However, it will be pointed out herein that important issues concerning use of some of the rules in Eurocode 2 have not been made clear. Therefore, these aspects are still open to interpretation, and it has not been possible to simply transpose the European rules into this design booklet. More development work is still needed.

The rules in Eurocode 2 also address the design of prestressed beams for crack control, but no changes have yet been proposed to the rules for these elements in AS 3600-2000.

Eurocode 2 allows a tiered approach to design: (i) crack width formulae can be used to keep crack widths below the design crack width (normally 0.3 mm – see Section 3.2); or (ii) simplified rules derived directly from the crack width formulae provide acceptable values of bar diameter and bar spacing depending on the maximum stress in the steel under service loads. Both of these design approaches or methods are discussed, although only the simplified rules (partly modified) have been proposed for AS 3600-2000. The objective behind explaining the crack width formulae is to provide engineers with the opportunity to understand the background to the simplified method. This reduces the likelihood of the simplified rules just being followed like a recipe, and helps designers better appreciate the influence of major design parameters such as bar diameter.

Cracking of concrete is a major topic with numerous complex facets. The reader is referred to other documents for a broader insight into the topic, e.g. [6,7,8]. Some documents, which explain some of the development work behind the design rules in Eurocode 2, are also worth citing [9,10,11,12].

Cracking can be caused by any of a variety of actions, which include loads applied during either construction or the in-service condition, foundation movements, temperature changes and gradients, shrinkage and creep of concrete, etc. The calculation of design action effects for crack control design can be a complex exercise in its own right for any of these situations [13].

When designing for flexural cracking in slabs, estimates of the bending moments for the serviceability limit state need to be calculated at critical sections by some means. Otherwise, experience shows that uncontrolled cracking can result, which is particularly undesirable in exposed structures such as car parks where its visual impact is poor and durability can be a concern (see Fig. 1.2). It is beyond the scope of this booklet to address the topic of calculating bending moments in detail, and the reader may need to refer to specialist literature and computer software manuals for guidance. Elastic, finite element analysis is a powerful method for analysing two-way reinforced-concrete slabs, and the importance of its use in serviceability design is illustrated in Section 7 using a

standard, commercially available package. When crack control is important, designers should be wary of using plastic methods of strength design, e.g. yield-line theory. These methods inherently mean that a large amount of moment redistribution must occur at the serviceability limit state, as well as at the strength limit state, which can lead to excessively high steel stresses, particularly in internal support regions, under service conditions. In order to reduce the steel stresses, the cross-sectional area of the reinforcement must be increased above that required for strength alone. Therefore, the reinforcement is under-utilised at the strength limit state, which can become particularly significant when 500 MPa reinforcing steels are being used. This is clearly undesirable for economy. An important objective of writing this booklet has been to reduce the likelihood of this situation occurring, and the reader is referred to the worked examples in Section 7 for guidance on this matter.



Figure 1.2 Avoidable Flexural Cracking in a Carpark

2. TERMINOLOGY

Some important terminology used in this booklet is summarised in this Section.

Action

An agent including direct loading, imposed deformation or restrained deformation, which may act on a structure or its component elements.

Action effects

The forces and moments that are produced in a structure or in its component elements by an action.

Cracked section

A section of a reinforced-concrete slab, cracked over part or all of its cross-sectional area, where the tensile strength of the concrete is ignored in design.

Critical steel content

Minimum amount of steel required in the tensile zone of a tensile or flexural element for multiple cracks to form in a uniform stress zone.

Direct loading

Loading on a structure that includes the self-weight of its component elements and externally applied loads.

Fully cracked section

A section of a reinforced-concrete slab, cracked over all of its cross-sectional area, where the tensile strength of the concrete is ignored in design.

Imposed deformation

Deformation imposed on a slab by its supports.

Main reinforcement

Reinforcement provided by calculation to resist action effects, irrespective of its direction.

Non-flexural elements

Deep beams, footings, pile caps, corbels, etc. as defined in Clause 12.1.1.1 of AS 3600-1994.

One-way slab

A slab characterised by flexural action mainly in one direction.

Restrained deformation

Deformation of a slab resulting from concrete shrinkage or temperature variations restrained by its supports (or by embedded reinforcement).

Stabilised crack pattern

The final crack pattern that forms in a reinforced-concrete element.

State of flexure

The condition when the tensile stress distribution within the section prior to cracking is triangular, with some part of the section in compression.

State of tension

The condition when the whole of the section is subject to tensile stress.

Two-way slab

A slab characterised by significant flexural action in two, normally orthogonal directions.

Uncracked section

A section of a reinforced-concrete slab uncracked over its entire cross-sectional area, where the tensile strength of the concrete is included in design.

3. DESIGN CONCEPTS & MODELS

3.1 General

Cracking of concrete will occur whenever the tensile strength of the concrete is exceeded. This is inevitable in normal reinforced-concrete structures, and once formed the cracks will be present for the remainder of a structure's life. Because cracks affect the serviceability of a building, the limit state of excessive crack width needs to be considered in design.

This booklet is concerned with the design of reinforced-concrete slabs for crack control after the concrete has hardened. This essentially involves ensuring firstly that the cracks form in a well-distributed pattern, normally very early in the life of a structure, and secondly that they do not become excessively wide while a building is in service. Moreover, this first condition requires that the tension reinforcement must have a minimum tensile capacity when the cracks form. The second condition requires that the tensile stresses in the reinforcement do not become too large at any time in the life of a structure, at least under serviceability conditions.

Cracking can occur when the slabs are directly loaded. This may be immediately after the temporary props are removed during construction and the slabs must support their self-weight for the first time. Imposed or restrained deformations can also cause cracking. Support settlement is an example of an imposed deformation that tends to cause flexure in a slab. Shrinkage of concrete or temperature changes can cause the occurrence of restrained deformation. These actions can cause significant flexural or direct tensile stresses to develop in the hardened concrete. Without steel reinforcement, a cracked section cannot provide flexural or tensile restraint to the adjoining concrete segments, and crack control is impossible. Reinforcing steel is required in slabs to control cracking under these circumstances. The way in which tension reinforcement can control cracking in a slab subjected to restrained deformation arising from concrete shrinkage is broadly illustrated in Fig. 3.1. Clause 9.4.3 of AS 3600-2000 contains design provisions specifically for this case, which require designers to consider the degree of restraint to in-plane movement (see Section 3.2).

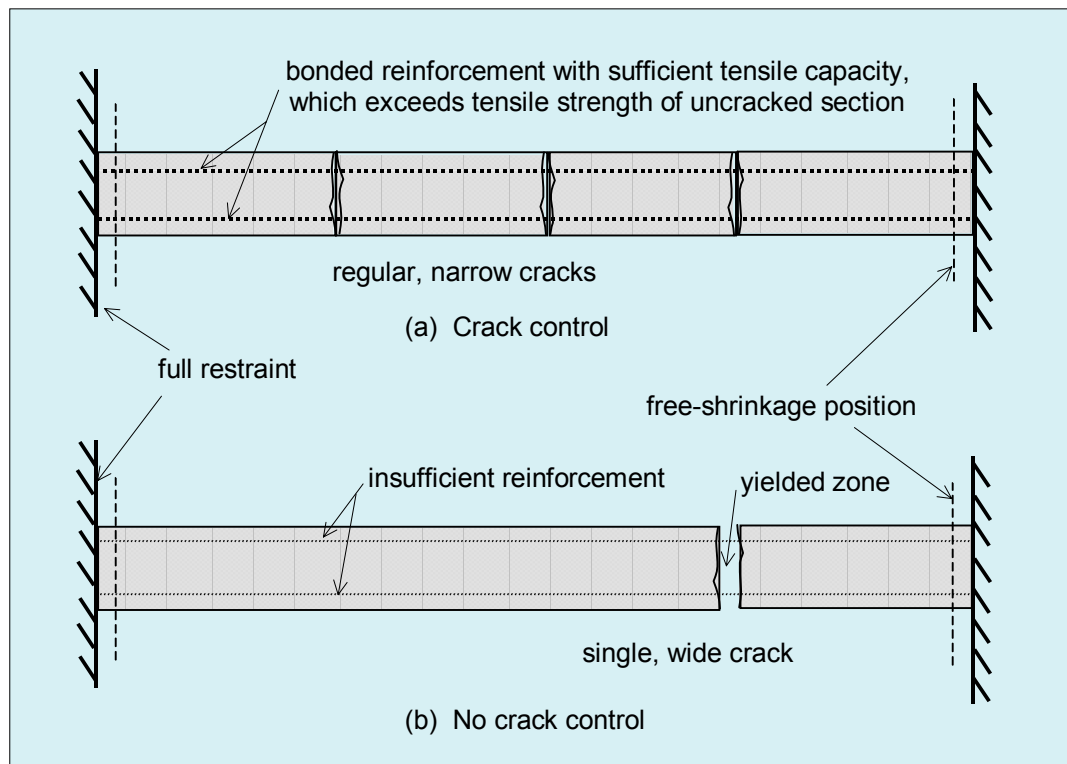


Figure 3.1 Control of Cracking Caused by Restrained Deformation

Prior to the concrete hardening, plastic cracking can arise from either plastic settlement (which

results from differences in density of the mix components), or plastic shrinkage (which results from evaporation of bleed water), or a combination of both.

Plastic settlement cracking involves uneven settlement of the concrete mix, which can be pronounced at changes in thickness of the concrete, and near discrete obstructions such as steel reinforcing bars or larger pieces of aggregate. There are many established practices for reducing plastic settlement. These include using cohesive concrete mixes, using air-entrainer, increasing cover to top reinforcement, and adjusting the placing rate.

Plastic shrinkage cracking is likely to occur when there is rapid loss of bleed water, which might occur by evaporation or possibly by absorption into the adjacent formwork or subgrade. Experienced practitioners will normally take measures to avoid these problems, including sealing the forms, erecting windbreaks and covering the exposed surface of the concrete as early as possible after finishing.

3.2 Crack Width Limits

As a rule, a design engineer should aim to detail slabs such that tensile strains are distributed over a large number of narrow cracks rather than a small number of wide cracks in the surface of the concrete. The control of surface cracking is particularly important in certain situations. The most common of these is where the surface will be visible, as excessive crack widths can give an overall impression of poor quality and can limit the types of floor coverings that can be successfully used. Crack control is also important for durability where the cracks would provide pathways for the ingress of corrosive substances such as water.

Where there are no special requirements such as watertightness, Eurocode 2 recommends that a limit of 0.3 mm under quasi-permanent (long-term) loads should normally be satisfied. This design crack width, w_k , is intended to be a characteristic value with only a 5% probability of being exceeded (see Section 3.5.2). It is recognised that for reinforced-concrete slabs in a dry environment corresponding to the interior of buildings for normal habitation or offices, durability is not normally a concern that affects the choice of design crack width.

In areas where the top surface of the concrete is exposed to foot or vehicular traffic, such as in a carpark or a footbridge, the edges of cracks can become damaged and frayed, and appear considerably wider at the surface. In this type of situation, it is prudent to control crack widths even more tightly.

Crack width is not specifically mentioned as a design parameter in AS 3600. However, in accordance with Clause 9.4.3, it is necessary to consider the degree of control over cracking in the primary and secondary directions in restrained or partially restrained slabs. The degree of control is defined as (AS 3600 Supp1-1994):

- (i) minor control – where cracks are aesthetically inconsequential or hidden from view;
- (ii) moderate control – where cracks will be seen but can be tolerated; and
- (iii) strong control – where the appearance of obvious cracks is unacceptable, or where cracks may reflect through finishes.

In the context of this design booklet, it will be assumed that provided the degree of crack control is at least moderate, then crack widths will be commensurate with those corresponding to the limit of 0.3 mm adopted in Eurocode 2. Thus, in the context of this design booklet minor control is not an acceptable level of crack control in restrained or partially restrained slabs.

3.3 Cracking of Reinforced-Concrete Elements

3.3.1 Introduction

The development of cracking in reinforced-concrete elements by tension or flexure is a complex topic. A brief, general account is given here of this behaviour before specifically discussing cracking of reinforced-concrete slabs in Section 3.4.

Cracking is considered to have occurred under conditions of direct tension, referred to herein more

simply just as “tension”, if tensile stresses existed across the entire cross-section immediately prior to cracking. According to this definition, bending moment can be carried by a section deemed to be in tension, provided there is no compressive stress. Stresses induced by differential temperature or shrinkage that develop through the depth of a reinforced-concrete element can complicate the resultant stress distributions prior to cracking. Their existence increases the difficulty of predicting the onset of cracking, and they will be ignored here.

Many different actions can cause tension or flexure to develop in a section of a reinforced-concrete element, e.g. applied loads, support settlement, concrete shrinkage, or thermal expansion or contraction. The support or restraint that is provided at the boundaries of the element can also significantly affect the stress distribution at each of its cross-sections. However, in order to simplify this discussion, it will generally be assumed that the elements support applied loads of known magnitude. It will also be generally assumed that they are statically determinate, whereby the resultant tensile force and/or bending moment acting at every cross-section is known.

3.3.2 Cracking in Tension Elements

3.3.2.1 Classical Theory

It is instructive to consider the behaviour of a reinforced-concrete tension element with a longitudinal reinforcing bar placed concentrically in its cross-section and loaded at each end by a known force (see Fig. 3.2). The classical theory, which describes the development of cracking in such an element, is well accepted, and has often been used as a basis for deriving equations for predicting crack widths [12,14]. The type of approach taken is described as follows.

When the bar is loaded in tension, some bond breakdown occurs between the bar and the concrete near the ends of the element. Further in, a uniform strain distribution is assumed to develop, and slip between the steel and concrete remains zero.

The first crack forms at the weakest section somewhere in the region of uniform strain when the tensile strength of the concrete is reached. This assumes that the tensile capacity of the bar exceeds that of the concrete, noting that when they are equal this is referred to as the critical steel content. Otherwise, the bar will fail in tension outside the concrete before the concrete can crack.

Just like at the ends of the concrete element, the force in the steel bar at the crack equals the applied load, while the concrete is unstressed at the crack faces. Also, slip occurs and bond stress, τ , develops between the concrete and the steel bar over a transfer length, l_{tr} , each side of the crack. It is by bond that stress is transferred into the concrete. Depending on the overall length of the element in relation to the transfer length, other cracks can form at slightly higher loads.

Theoretically, the spacing between cracks that form adjacent to each other cannot be less than l_{tr} , and nor can it exceed $2l_{tr}$. This is explained as follows. Consider the two cracks that have formed at cross-sections 1 and 2 in Fig. 3.2. A new crack can only form between them if they are at least $2l_{tr}$ apart. If the spacing is just above this limit and another crack forms, then the crack spacing will be close to l_{tr} . Thus, it can be written that:

$$s_{cr.min} = l_{tr} \quad 3.3.2.1(1)$$

$$s_{cr.max} = 2l_{tr} \quad 3.3.2.1(2)$$

It follows from equilibrium of longitudinal forces that if τ_m is the average bond stress over the transfer length l_{tr} , f_t is the tensile strength of concrete and Σ_o is the bar perimeter, then:

$$l_{tr} = \frac{A_c f_t}{\tau_m \Sigma_o} \quad 3.3.2.1(3)$$

Substituting bar perimeter $\Sigma_o=4A_{st}/d_b$ and reinforcement ratio for tension $\rho_s=A_{st}/A_c$, it follows that:

$$l_{tr} = \frac{d_b f_t}{4 \tau_m \rho_s} \quad 3.3.2.1(4)$$

and from Eqs 3.3.2.1(2) and 3.3.2.1(4), the maximum crack spacing becomes:

$$s_{cr,max} = \frac{d_b f_t}{2 \tau_m \rho_s} \quad 3.3.2.1(5)$$

Finally, crack width equals the elongation of the steel between two adjacent cracks less the elongation of the concrete, and one can write:

$$w_{max} = s_{cr,max} (\epsilon_{sm} - \epsilon_{cm}) \quad 3.3.2.1(6)$$

where ϵ_{sm} and ϵ_{cm} are the mean steel and concrete strains over transition length l_{tr} .

At the end of the transition length, the steel bar is fully bonded to the concrete, and the tensile force in the steel at this location, T'_b , is given by:

$$T'_b = T_b \frac{np_s}{1 + np_s} \quad 3.3.2.1(7)$$

where T_b is the tensile force in the bar at cracked sections. Assuming a uniform bond stress over transition length l_{tr} , it follows that the average strain in the steel bar, ϵ_{sm} , is given by:

$$\epsilon_{sm} = \frac{1}{2E_s} \left(\frac{T_b}{A_{st}} + \frac{T'_b}{A_{st}} \right)$$

and from Eq. 3.3.2.1(7), it follows that,

$$\epsilon_{sm} = \frac{f_s}{2E_s} \left(\frac{1 + 2np_s}{1 + np_s} \right) \quad 3.3.2.1(8)$$

where $f_s = T_b/A_{st}$.

From Eq. 3.3.2.1(8) it can be seen that the average steel strain, ϵ_{sm} , is a function of the reinforcement ratio $\rho_s = A_{st}/A_c$. It follows that ϵ_{sm} increases with ρ_s , and in practice, the term in brackets in Eq. 3.3.2.1(8) might reach a maximum value of about 1.3. Therefore, a reasonable upper estimate for ϵ_{sm} is $0.65f_s/E_s$. It follows that, if elongation of the concrete is ignored, i.e. $\epsilon_{cm}=0$, then an approximate formula for maximum crack width, w_{max} , can be written as follows using Eqs 3.3.2.1(6) and 3.3.2.1(8):

$$w_{max} = s_{cr,max} \frac{0.65f_s}{E_s} \quad 3.3.2.1(9)$$

and then substituting Eq. 3.3.2.1(5) into Eq. 3.3.2.1(9) to give:

$$w_{max} = \frac{d_b f_t}{2 \tau_m \rho_s} \frac{0.65f_s}{E_s} \quad 3.3.2.1(10)$$

This important relationship shows that, all other parameters remaining the same, if the bar diameter d_b is increased, then the steel stress f_s at a cracked section must proportionally decrease if the maximum crack width is to remain unchanged. Therefore, the interdependence between crack width, bar diameter and steel stress has been approximately established using classical theory for a reinforced-concrete tensile element with a stabilised crack pattern.

Equation 3.3.2.1(10) for maximum crack width can be refined by including terms to account for the concrete strain. At an uncracked section, it can be written that:

$$f_c = \frac{f_s \rho_s}{(1 + np_s)} \quad 3.3.2.1(11)$$

where f_c is the tensile stress in the concrete, which has a maximum value of f_t , whereby it follows from Eq. 3.3.2.1(11) that:

$$f_s = \frac{f_t(1 + np_s)}{\rho_s} \quad 3.3.2.1(12)$$

It will again be assumed that the bond stress τ is uniform or constant over the transition length. Then it can be written:

$$\epsilon_{cm} = \frac{1}{2} \epsilon_c = \frac{1}{2} \epsilon_s = \frac{1}{2} \frac{f_s}{E_s} \quad 3.3.2.1(13)$$

which leads, from Eq. 3.3.2.1(12), to:

$$\epsilon_{cm} = \frac{1}{2} \frac{f_t(1 + np_s)}{\rho_s E_s} \quad 3.3.2.1(14)$$

As a further refinement, the effect of concrete shrinkage can be included to give:

$$\epsilon_{cm} = \frac{1}{2} \frac{f_t(1 + np_s)}{\rho_s E_s} - \frac{1}{2} \frac{\epsilon_{cs}}{(1 + np_s)} \quad 3.3.2.1(15)$$

where ϵ_{cs} is the free shrinkage strain of the concrete and will be a negative value. It follows from Eqs 3.3.2.1(6) and 3.3.2.1(15) that Eq. 3.3.2.1(10) can be written more generally as:

$$w_{max} = \frac{d_b f_t}{2 \tau_m \rho_s} \left(\frac{0.65 f_s}{E_s} - \frac{1}{2} \frac{f_t(1 + np_s)}{\rho_s E_s} - \frac{1}{2} \frac{\epsilon_{cs}}{(1 + np_s)} \right) \quad 3.3.2.1(16)$$

Returning to the case of a single crack, the equations necessary to calculate its width, w , will be formulated assuming constant bond stress. Firstly, similar to Eq. 3.3.2.1(6) it can be written that:

$$w = 2l_{tr}(\epsilon_{sm} - \epsilon_{cm}) \quad 3.3.2.1(17)$$

It follows from Eqs 3.3.2.1(4) and 3.3.2.1(12) that Eq. 3.3.2.1(17) becomes:

$$w = \frac{d_b f_s}{2 \tau_m (1 + np_s)} (\epsilon_{sm} - \epsilon_{cm}) \quad 3.3.2.1(18)$$

The average steel strain, ϵ_{sm} , is given by Eq. 3.3.2.1(8), while the average concrete strain, ϵ_{cm} , can be shown to equal:

$$\epsilon_{cm} = \frac{f_s}{2E_s} \frac{np_s}{(1 + np_s)} \quad 3.3.2.1(19)$$

Substituting Eqs 3.3.2.1(8) and 3.3.2.1(19) into Eq. 3.3.2.1(18) gives:

$$w = \frac{d_b f_s}{2 \tau_m (1 + np_s)} \left(\frac{f_s}{2E_s} \right) \quad 3.3.2.1(20)$$

Like for Eq. 3.3.2.1(16), the effect of concrete shrinkage can be included to give:

$$w = \frac{d_b f_s}{2 \tau_m (1 + np_s)} \left[\frac{f_s}{2E_s} - \frac{1}{2} \frac{\epsilon_{cs}}{(1 + np_s)} \right] \quad 3.3.2.1(21)$$

Equations 3.3.2.1(16) and 3.3.2.1(21) can be further generalised by incorporating the effects of a non-linear bond stress relationship based on test data. However, it is beyond the scope of this booklet to consider the derivation of more accurate forms of these equations. It can be shown that by simply substituting $w=0.3\text{mm}$, $\tau_m=5.5\text{ MPa}$ (a representative value for deformed bars, although dependent on factors such as the tensile strength of the concrete, confinement, etc. [15]) and $\epsilon_{cs}/(1+np_s)=-250\text{ }\mu\epsilon$ into Eq. 3.3.2.1(21) leads to a very similar relationship between d_b and f_s to that given in Eurocode 2, at least for bar diameters up to 20 mm. The relationship in Eurocode 2 is for simplified crack control design of either tension or flexural elements.

3.3.2.2 Minimum Reinforcement

Ignoring shrinkage restraint of the reinforcing steel, the theoretical static load at first cracking, T_{cr} , is given by [16]:

$$T_{cr} = f_t (A_c + nA_{st}) \quad 3.3.2.2(1)$$

For the situation shown in Fig. 3.2, where the tensile force T_b is shown being applied to both ends of the steel bar protruding from the concrete, a crack will not form unless $T_b \geq T_{cr}$. Thus, it can be written, assuming for simplicity that the reinforcing steel behaves elastic-plastically with a yield strength of f_{sy} :

$$f_{sy} A_{st} > f_t (A_c + nA_{st}) \quad 3.3.2.2(2)$$

and transforming gives:

$$A_{st} > \frac{f_t A_c}{f_{sy} - nf_t} \quad 3.3.2.2(3)$$

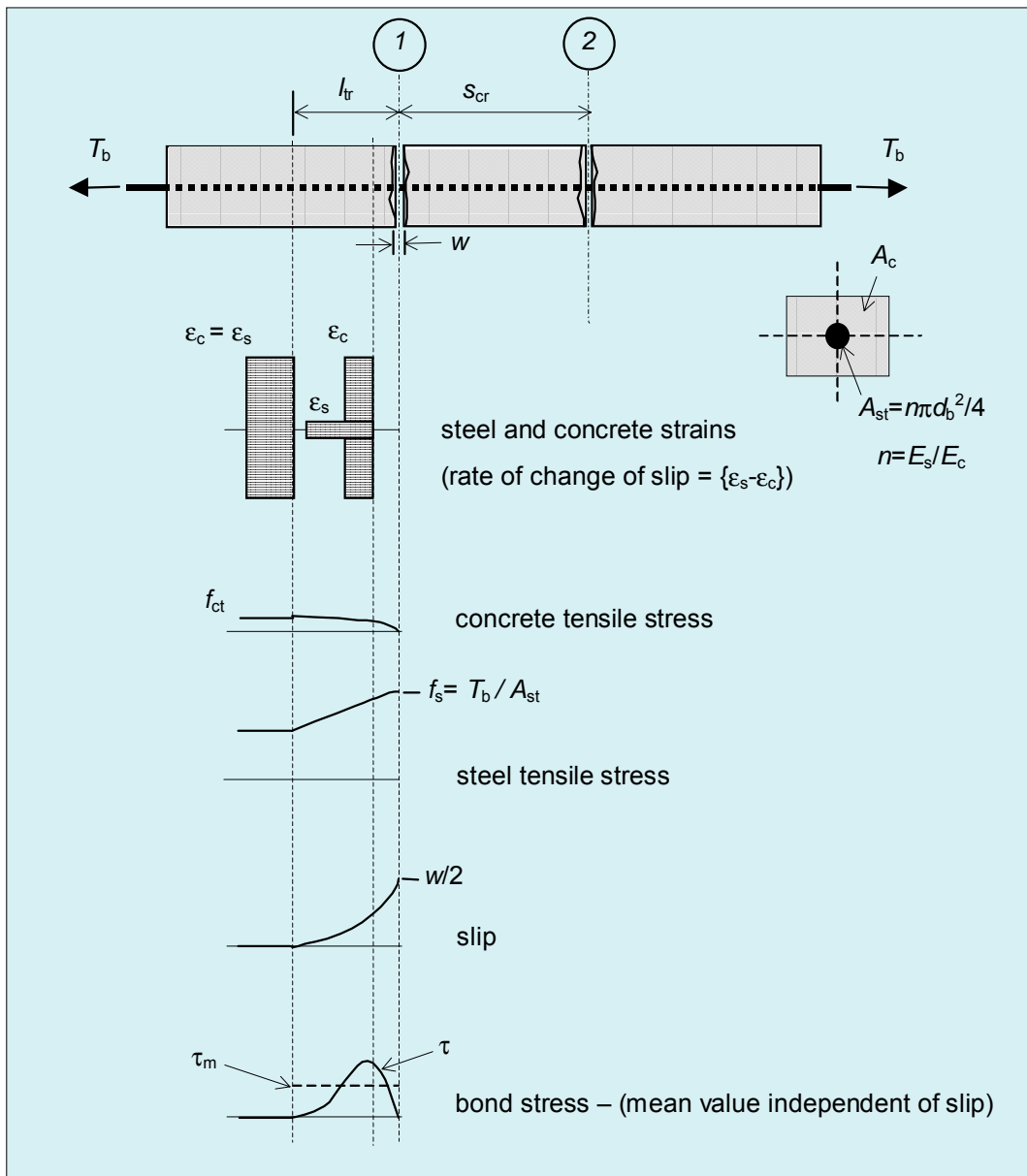


Figure 3.2 Cracking in Tension [16]

With this condition satisfied, at least one crack can form in the concrete. Whilst this equation is only approximate because nominal material properties are implied, the real intention of deriving it is to show when multiple cracks will form. Multiple cracks are desirable since on average they are finer the more cracks that form.

Nevertheless, from a design perspective, it is theoretically not necessary to comply with Eq. 3.3.2.2(3), if at the strength limit state the design tensile force, T^* , is less than T_{cr} . This is because in this case, cracking would theoretically be avoided altogether (ignoring shrinkage, etc.).

The derivation of an equation for calculating the minimum reinforcement of a redundant tensile element like that shown in Fig. 3.1 is quite different to above. Shrinkage restraint of the concrete must be considered, noting that prior to cracking the steel stress is zero. However, if satisfactory crack control is to be achieved Eq. 3.3.2.2(2) must still normally be satisfied, since only then can multiple cracks form. The intention is to avoid yielding of the reinforcement, because crack widths cannot be controlled if this occurs.

As a final comment, if the simple equations given above are used for design, then conservative estimates for the concrete tensile and steel yield strengths need to be used.

3.3.3 Cracking in Flexural Elements

3.3.3.1 General

The classical theory used in Section 3.3.2 to describe the development of cracking and the calculation of crack widths in tension elements is based on a large number of simplifying assumptions. For example, the tensile stress in the concrete would not be uniform, but would vary significantly across the width and depth of the element at cross-sections away from the cracks. This is due to the local nature of the bond transfer mechanism, which involves part of the force in the steel at cracks dispersing into the surrounding concrete creating a sort of stress bulb in the concrete. The area of concrete that can be assumed effective in tension at critical sections would clearly depend on this effect. It is also assumed that cracks have constant width with parallel sides through the width and depth of the element.

For flexural elements, which can be under a combination of bending and tension, the complexity of the internal stress distributions prior to and after cracking is further increased compared with tension elements. This makes the definition of an effective area of concrete in tension an even greater issue. Nor can crack width be assumed to be constant over the depth of the element, naturally equalling zero in zones of compressive stress for example. Moreover, tests have shown that concrete cover can have a significant affect on surface crack widths. Attempts to apply equations of a similar form to Eqs 3.3.2(16) and 3.3.2(21) to predict crack widths in beams and slabs have generally demonstrated a need to reduce the effect of d_b and p_s . This has led to the development of simpler empirical equations. A statistical approach has been used because of the large variability between test results and predicted values.

It is beyond the scope of this document to review the background to the different crack width equations that have been developed. Identifying the main variables is more contentious than for tension elements. However, the crack width equations proposed in Eurocode 2 for flexural elements are based on a modified form of Eq. 3.3.2.1(6). This has made it possible to design either tension or flexural elements using the same basic design equations, with different values for some of the coefficients. These equations are presented in Section 3.5.

3.3.3.2 Minimum Reinforcement

It was stated in Section 3.3.2.2 that one basic principle that must be complied with to control cracks in tensile elements is to avoid yielding the reinforcement. This same principle applies for flexural elements. The most elementary way of expressing this requirement is to write:

$$M_{sy} > M_{cr} \quad 3.3.3.2(1)$$

where M_{sy} is the moment capacity of the flexural element at a cracked section with the reinforcing steel at its yield strain, and M_{cr} is the cracking moment. Here it is conservative to ignore the effects of

shrinkage, because this can reduce the cracking moment.

However, multiple cracks will not form under moment gradient conditions if Eq. 3.3.3.2(1) is only just satisfied at the peak moment cross-section of the critical positive or negative moment region. Therefore, a more stringent requirement may actually be required in practice.

Expanding on Eq. 3.3.3.2(1), one can write:

$$A_{st}f_{sy}\chi d > \frac{\xi f_t b D^2}{6} \quad 3.3.3.2(2)$$

where ξ (≥ 1) is a parameter to account for moment gradient effects, and χd is the lever arm of the internal force couple (noting that for lightly-reinforced elements $\chi \approx 0.9$). Transforming Eq. 3.3.3.2(2) gives:

$$\frac{A_{st}}{bd} > \frac{\xi f_t}{6\chi f_{sy}} \left(\frac{D}{d} \right)^2 \quad 3.3.3.2(3)$$

It is interesting to note that Eq. 3.3.3.2(3) shows that the minimum value of the reinforcement ratio, $p = A_{st}/bd$, required for crack control of a section in bending, is independent of the overall depth, D , depending instead on D/d .

Tests shows that shallow, reinforced-concrete rectangular slabs with a low value of reinforcement ratio, A_{st}/bd , attain significantly greater bending strengths than Eq. 3.3.3.2(3) would indicate [17]. Typically, their overall depth, D , could be up to 200 mm or more for this to still hold. This is because under static loading the tensile strength of the concrete makes a significant contribution towards the moment capacity of the critical section compared with the reinforcement. Therefore, crack growth tends to be stable in shallow, lightly-reinforced flexural slabs.

If one uses $\xi = 1.2$, $f_t = 3.0$ MPa, $\chi = 0.9$ and $(D/d) = 1.1$, then Eq. 3.3.3.2(3) gives $A_{st}/bd = 0.8/f_{sy}$. This is the same as the equation in Clause 9.1.1 of AS 3600-1994, for calculating the minimum tensile reinforcement to provide sufficient strength in bending for slabs supported by beams or walls.

3.4 Actions Causing Cracking of Reinforced-Concrete Slabs

3.4.1 One-Way Action

A one-way slab is characterised by flexural action mainly in one direction. An example is a slab subjected to dominantly uniformly-distributed loads with two free (unsupported) approximately parallel edges. Another is the central part of a rectangular slab supported on all four edges by walls or beams, with the ratio of the lengths of the longer to shorter sides (or aspect ratio) greater than 2.

Some examples are given here to describe the way cracking can develop in one-way reinforced-concrete slabs. Their behaviour is similar to rectangular reinforced-concrete beams. Cracking can be caused by any of a number of actions, which include loads applied during construction or the in-service condition, foundation movements, temperature changes and gradients, shrinkage and creep of concrete, etc. These actions also cause cracking of two-way slabs, which is discussed briefly in Section 3.4.2.

The approach taken in Eurocode 2 when designing a reinforced-concrete beam or slab for crack control is to determine whether it is in a state of tension or flexure prior to the onset of cracking (see Fig. 3.3). This same approach has been adopted in AS 3600-2000 when designing beams, and is described in detail in design booklet RCB-1.1(1). However, for reasons described below, it has not been adopted in AS 3600-2000 when designing slabs for crack control. Nevertheless, it is still instructive to consider what actions can cause a state of tension or flexure in slabs. Recapping from Section 3.3.1, the internal forces in a slab prior to cracking can be a combination of tensile force and bending moment. A state of tension is assumed to exist if there is no compressive stress at any location on the section of concern (see Fig. 3.3(b)).

The types of actions that commonly cause a state of tension or flexure in a one-way slab (or a beam) are shown in Fig. 3.4. These are briefly explained as follows.

Direct loading is a common cause of a state of flexure in a reinforced-concrete slab with a normal span-to-depth ratio (see Fig. 3.4(a)). Many engineers probably regard this as the most important situation for designing for crack control. However, other actions can cause significant cracking in relatively lightly-loaded structures if their effect is overlooked during design.

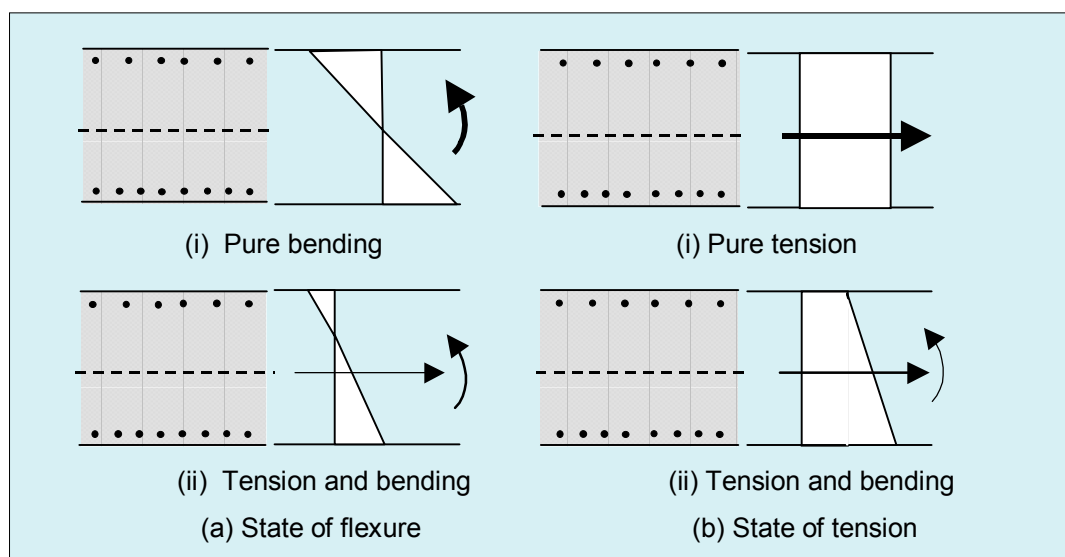


Figure 3.3 State of Flexure or Tension Prior to Cracking

Support or foundation movement is the type of action that is often overlooked by design engineers, even though this can lead to significant serviceability problems if the structure is not appropriately designed. Articulation joints may be required if the movements anticipated are very large. Settlement of a slab support is an example of an *imposed deformation*. It can arise due to differential soil settlement, in which case the deformation is externally imposed on the structure. Differential column shortening is an example of an internally imposed deformation. This can have the same effect on a continuous slab as an externally imposed deformation, and cause significant redistribution of bending moments possibly leading to flexural cracking (see Fig. 3.4(b)).

Differential temperature or shrinkage in a continuous reinforced-concrete slab can also cause a significant change to the distribution of bending moments leading to flexural cracking. Examples of when this might need to be considered are a floor exposed to direct sunlight, or a precast reinforced-concrete slab with a cast-insitu topping slab. Without the supports present, the change in curvature due to the effects of temperature or shrinkage would not cause the bending moment distribution to change. However, under the influence of gravity and applied loads, the slab deformation is restrained by the supports whereby the slab is considered subjected to *restrained deformation* (see Fig. 3.4(c)).

Direct tension is not normally associated with elevated slabs, which are considered to be flexural elements. An example of when a slab might be in a state of tension is if it is lightly loaded, and effectively restrained at its ends or around its edges by beams or walls. This is a classic case of *restrained deformation* arising due to the effects of concrete shrinkage (see Fig. 3.4(d)). Restraining the contraction of a slab subjected to a drop in average temperature also causes a resultant tensile force to develop.

The examples showing states of flexure or tension in Fig. 3.4 are an oversimplification in practice for slabs. This is because slabs, being wide elements, are normally subjected to more than one action. For example, the one-way slabs shown in Fig. 3.4 will all shrink transversely and thus experience the effects of restrained deformation to varying amounts depending on the degree of restraint in the transverse direction. This can give rise to cracks that penetrate completely through the slab. In fact, it is recognised in AS 3600-1994 that shrinkage can cause more severe cracking when it is not accompanied by flexure. In particular, in accordance with AS 3600-1994 it has been normal when detailing slabs for crack control to treat flexure (e.g. due to direct loading) and shrinkage and temperature effects (due to restrained deformation), separately. As far as slabs are concerned, this

approach is unchanged in AS 3600-2000, and will therefore be adopted in this design booklet.

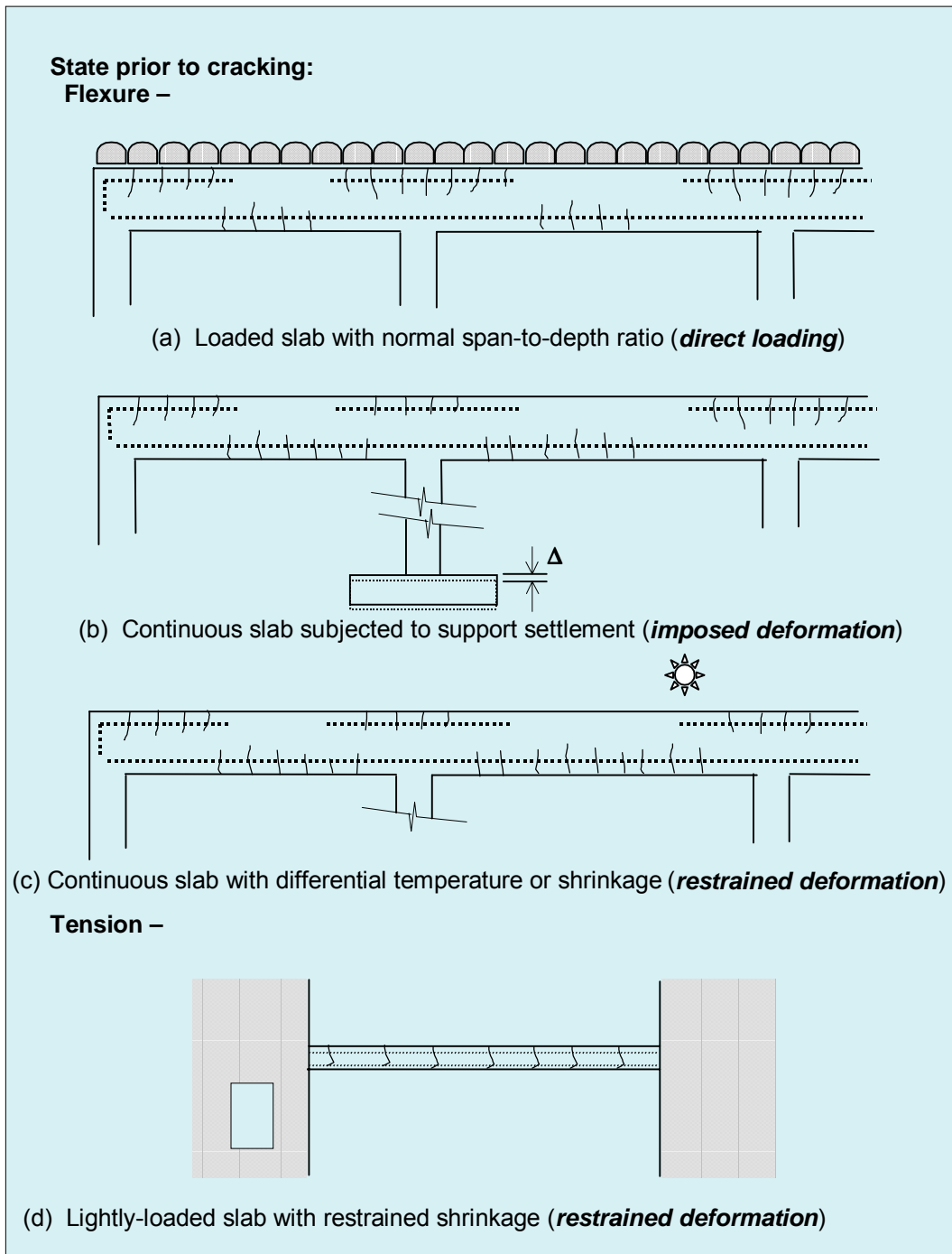


Figure 3.4 Examples of One-Way Slabs Deemed to be in Flexure or Tension under Different Actions

It is beyond the scope of this design booklet to describe the methods of analysis that are required to calculate the change in design action effects that results for each of the actions described above. Standard computer programs that analyse redundant structures can be used to account for the effects of direct loading (whether causing states of tension or flexure), and imposed deformations such as support settlement. They can also be used to account for the effect that restrained deformation at supports has on the flexural action that is caused by differential temperature or differential shrinkage. However, the analysis of slabs subjected to a state of tension due to restrained

deformation is not straightforward, and simplifying assumptions normally need to be made, e.g. [13]. In this latter case, the onset of cracking causes a significant reduction in axial stiffness, and therefore in the tensile restraining force, when concrete shrinkage is involved. Clause 9.4.3 of AS 3600-2000 contains simple rules for designing slabs for shrinkage and temperature effects which avoid having to calculate design action effects. These simple rules are preferred by designers, and as mentioned above this approach will continue to be adopted for slabs.

3.4.2 Two-Way Action

A two-way slab is characterised by significant flexural action in two, normally orthogonal directions. Most floors fall into this category. An example is a wide slab subjected to dominantly concentrated loads. Flat plates, and flat slabs incorporating drop panels, both supported on columns, are cases when concentrated reactions give rise to two-way action. Another example is a rectangular slab supported on all four edges by walls or beams, with an aspect ratio less than 2.

Even in the more straightforward case of direct loading, generally designers make simplifying assumptions about the behaviour of a two-way slab when calculating design action effects. Torsional moment is normally ignored because of lack of a simple method to account for it. For example, torsional reinforcement is required at exterior corners of slabs, and in AS 3600-2000 the amount of reinforcement is determined by a deemed-to-comply provision (Clause 9.1.3.3) rather than direct calculation.

Complex behavioural issues, such as torsional effects, mean that flexural cracking in certain regions of two-way reinforced-concrete slabs will be different to that in one-way beams or slabs. Nawy [18] believes that the most important parameter to be considered to control cracking in two-way floors is reinforcement spacing. He argues that cover is normally small and therefore not a major parameter, and recommends that the maximum spacing of the reinforcement in both orthogonal directions, in the form of either bars or mesh, should not exceed 300 mm in any structural floor. Diagonal rather than orthogonal cracking patterns tend to form if the spacing is too large, noting that the orthogonal cracks are narrower. At higher loads, the diagonal cracks eventually form yield line patterns.

Imposed or restrained deformations lead to even more complex issues regarding the calculation of design action effects. To suggest that designers should use methods of analysis that attempt to accurately model the complex behaviour of two-way slabs for these actions is not realistic at this point in time.

Moreover, there is some controversy about using crack width formulae derived from tests on one-way acting beams and slabs to predict crack widths in two-way slabs. For example, Nawy [18] recommends using a different equation to the well-known Gergely-Lutz equation on which ACI 318 [19] is based, and which is used to calculate crack widths in beams and one-way slabs. However, Park and Gamble [20] explain that this alternative formula may be too conservative, and recommend against its use, preferring instead to use the Gergely-Lutz equation for two-way slabs too. They comment that “critical cracking takes place principally in negative moment regions above the faces of the beams in slab and beam floors, and near the columns in flat slab and flat plate floors, whereas the studies of two-way slabs reported by Nawy et al. have dealt principally with positive-moment regions”. They argue that “it would appear reasonable to use the Gergely-Lutz equation in regions of slabs where one-way bending was predominant, for example in negative-moment regions when slabs are supported on stiff beams and in slabs at column faces and in the critical positive-moment regions of rectangular panels”. In a similar fashion, in the absence of a better known approach, the method recommended in Eurocode 2 for calculating crack widths of one-way elements (see Section 3.5) will also be applied to two-way slabs, or more particularly, to regions of two-way slabs where one-way bending is predominant.

3.5 Crack Width Calculations in Accordance with Eurocode 2

3.5.1 Introduction

Eurocode 2 provides engineers with two methods for controlling cracking in reinforced-concrete slabs. The overall thickness of the slab can influence the design procedure. These methods are described as follows.

- (i) Calculation of Crack Widths (Clause 4.4.2.4) – formulae are provided for crack width calculations which apply to both beams and slabs for a range of design situations, and are applicable irrespective of the overall depth of the element; or
- (ii) Control of Cracking without Direct Calculation (Clause 4.4.2.3) – a simplified design method is allowed, the rules for which have been derived using the crack width formulae. Minimum reinforcement areas are determined, and limits are placed on bar diameter and bar spacing. Alternatively, for slabs with an overall depth, D_s , not exceeding 200 mm that are subjected to bending *without significant axial tension* (i.e. in a state of flexure), cracking is assumed to be satisfactory if the detailing rules in Clause 5.4.3 of Eurocode 2 are satisfied.

The design method involving crack width calculations is briefly addressed in the following Sub-sections. Some changes have been made to the terminology to suit this design booklet, while some design situations covered in Eurocode 2 have also been omitted for brevity, e.g. references to plain bars which are not permitted as main reinforcement in Australia.

Simplified design in accordance with Eurocode 2 is covered in Section 3.6, which forms the basis for the new design rules proposed for inclusion in AS 3600-2000 for both beams and slabs, although slabs are only designed for a state of flexure using this approach. Despite the fact that they have not been proposed for incorporation in AS 3600-2000, the crack width formulae are presented here to provide engineers with the opportunity to understand the background to the simplified method. Moreover, the derivation of the acceptable values of bar diameter and bar spacing as a function of maximum steel stress is presented in Section 3.6.

The alternative approach in Eurocode 2, which allows slabs in a state of flexure to be detailed for crack control, is not permitted in AS 3600-2000 for the reasons given in Section 3.7, so will not be discussed further here.

The fundamental principles behind the design approach adopted in Eurocode 2 are stated in Clause 4.4.2.1(9) as follows:

- (i) a minimum amount of bonded reinforcement is required;
- (ii) yielding of the reinforcement must not occur during crack formation (see Section 3.7.4); and
- (iii) crack control is achieved by limiting (the steel stress depending on) bar spacing and/or bar diameter.

3.5.2 Derivation of Crack Width Formulae

Beeby and Narayanan [12] can be referred to for a more detailed account of the derivation of the crack width formulae in Eurocode 2. They are presented here in relation to slabs.

Similar in principle to Eq. 3.3.2.1(6), it can be written that:

$$w_k = \beta s_{cr,m} \epsilon_{sm} \quad 3.5.2(1)$$

- where –
- w_k is the design crack width, which is a characteristic value with only a 5% probability of being exceeded;
 - β is a factor that relates the mean crack width in tests to the design value, e.g. it equals 1.7 for cracking due to direct loading;
 - $s_{cr,m}$ is the average final crack spacing; and
 - ϵ_{sm} is the average difference in strain between the steel and the concrete including the effects of bond stress, tension stiffening, concrete shrinkage, etc.

Concerning crack spacing, in the first instance Eq. 3.3.2.1(4) is used to estimate the transfer length. Then the average crack spacing is assumed to equal 1.5 times this value, i.e. the average of Eqs 3.3.2.1(1) and 3.3.2.1(2). As an initial estimate for s_{crm} , it follows from Eq. 3.3.2.1(4) that:

$$s_{\text{crm}} = 1.5 \frac{d_b f_t}{4\tau_m \rho_s} \quad 3.5.2(2)$$

This can be rewritten as:

$$s_{\text{crm}} = \frac{k_1 d_b}{4\rho_s} \quad 3.5.2(3)$$

where the constant $k_1 = 1.5f_t/\tau_m$. As mentioned in Section 3.3.2.1, a representative value of average bond stress for deformed bar is $\tau_m = 5.5$ MPa, and a minimum value of concrete tensile strength of $f_t = 3.0$ MPa is recommended in Eurocode 2 (see Section 3.5.3), which gives $k_1 = 0.82$, while a value of 0.8 is specified in Eurocode 2.

It follows from above that Equation 3.5.2(3) applies to elements subjected to pure tension. It was further modified for inclusion in Eurocode 2 to cover both of these cases. The average final crack spacing for beams subjected dominantly to flexure or tension, s_{crm} (in mm), can be calculated from the equation:

$$s_{\text{crm}} = 50 + \frac{k_1 k_2 d_b}{4\rho_r} \quad 3.5.2(4)$$

- where –
- k_1 is a factor that takes account of the bar bond properties, and =0.8 for deformed bars (see above);
 - k_2 is a factor that takes account of the form of the stress distribution, thus allowing elements in a state of flexure as opposed to tension to also be designed, and =0.5 for pure bending and 1.0 for pure tension, while intermediate values can be calculated for eccentric tension on the basis of a cracked section; and
 - ρ_r is the effective reinforcement ratio, $A_{\text{st}}/A_{\text{c,eff}}$, where A_{st} is the cross-sectional area of reinforcement within the effective tension area of concrete, $A_{\text{c,eff}}$ (see Fig. 3.5 for slabs).

The first term of 50 mm was added to include the effect of concrete cover, which has been shown to have a direct effect on crack spacing. This was not explicitly included in the formula since it was felt that it might encourage engineers to minimise cover when designing for crack control. However, this does not appear to be entirely logical since designers will normally only specify the minimum cover necessary for durability.

The average strain, ε_{sm} , is calculated at the section being considered as:

$$\varepsilon_{\text{sm}} = \frac{f_s}{E_s} \left[1 - \beta_1 \beta_2 \left(\frac{f_{\text{sr}}}{f_s} \right)^2 \right] \quad 3.5.2(5)$$

- where –
- f_s is the stress in the tension steel under the serviceability condition being considered, calculated on the basis of a cracked section, and equals f_{sr} in the case of restrained deformation giving rise to a state of tension (since then ε_{sm} must equal zero, at least for $\beta_2 = 1.0$);
 - f_{sr} is the stress in the tension steel under the relevant condition that just causes the tensile strength of the concrete to be reached, calculated on the basis of a cracked section;
 - β_1 is a factor that accounts for the bond properties of the reinforcement, =1.0 for deformed bars; and
 - β_2 is a factor that accounts for repeated stressing of the bars,

=1.0 if the bars are only stressed once (which is not really relevant to design), or
=0.5 for repeated stressing (the normal design situation).

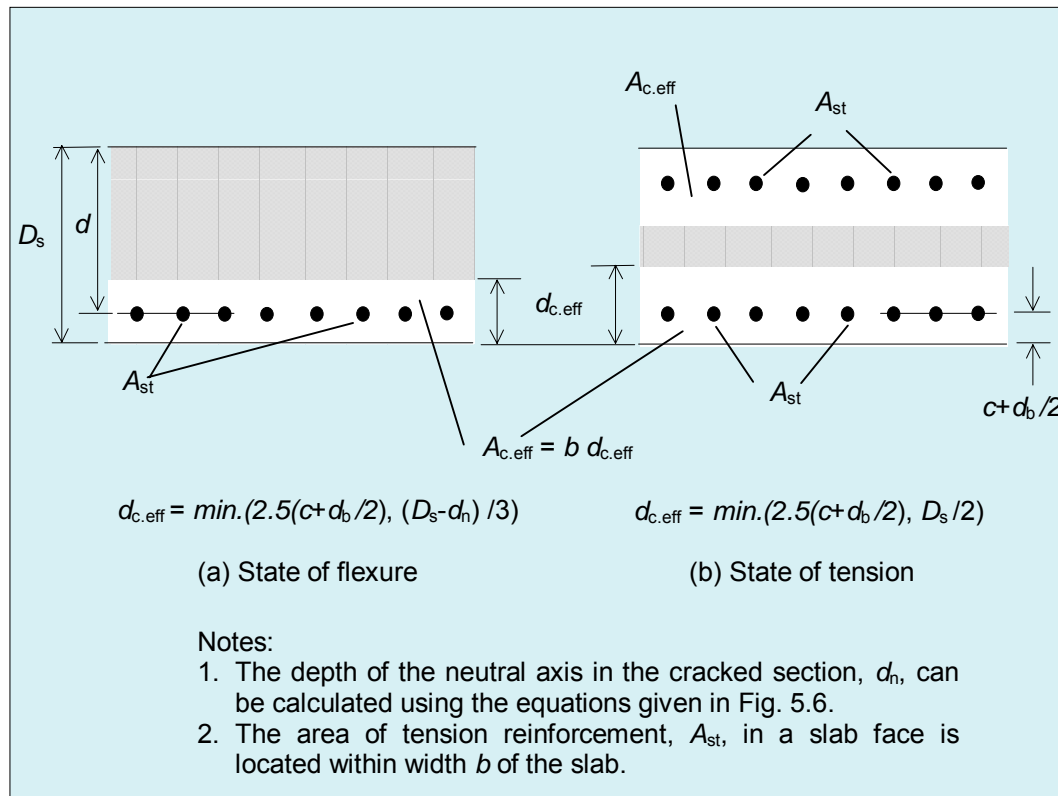


Figure 3.5 Definition of Effective Concrete Area, $A_{c,eff}$, for Slabs

It is assumed in the derivation of Eq. 3.5.2(5) that the reinforcement is completely unbonded next to each crack, and fully bonded in the middle region between adjacent cracks. The proportion of the distance between adjacent cracks, i.e. crack spacing, over which the reinforcement is assumed to be unbonded equals the term in the square brackets in Eq. 3.5.2(5). In the extreme, if this term equals zero, then the unbonded length equals zero and consequently ϵ_{sm} equals zero. This corresponds to behaviour prior to cracking. Equally, if this term equals a maximum value of 1.0, then the bar would be unbonded over its entire length resulting in $\epsilon_{sm} = f_s / E_s$. Equation 3.5.2(5) obviously has an empirical basis.

3.5.3 Minimum Reinforcement

Eurocode 2 requires that a minimum area of bonded reinforcement must be provided in beams and slabs subjected to restrained deformation where a state of tension is induced. This restraint might occur in combination with other actions. The steel must not yield while the cracks develop.

The equation for calculating this minimum area has been derived assuming equilibrium between the tensile forces in the steel and the concrete. It is similar in form to Eq. 3.3.2.2(2) except that, presumably for simplicity, the tensile force in the steel in the uncracked region has been ignored. Owing to this unconservative approximation, it seems appropriate to ignore the loss of area of concrete due to the steel reinforcement when applying this rule. The minimum area, $A_{st,min}$, is given by:

$$A_{st,min} = k_3 k_4 f_t A_{ct} / f_s \quad 3.5.3(1)$$

where – k_3 is a factor that allows for the effect of non-uniform self-equilibrating stresses, e.g. due to differential shrinkage or temperature,

- =0.8 for restrained deformation where a state of tension is induced, and may have other values for other situations (see Section 3.6.1);
- k_4 is a factor that takes account of the stress distribution at the section of concern immediately prior to cracking,
=1.0 for restrained deformation where a state of tension is induced, and may have other values for other situations (see Section 3.6.1);
- f_t is the (mean value of the) tensile strength of the concrete at the critical time when the cracks might occur, and a value of 3.0 MPa is recommended in Eurocode 2 for normal use;
- A_{ct} is the area of concrete in the tensile zone at the section of concern prior to cracking, and for the reason explained above, the cross-sectional area of the tension steel should not be subtracted from the gross area of the section in tension when calculating A_{ct} ; and
- f_s is the maximum stress permitted in the reinforcement immediately after the formation of the crack.

Concerning the value of f_s in Eq. 3.5.3(1), it must not exceed the nominal yield strength of the reinforcement, f_{sy} . As explained above, this requirement is intended to ensure that multiple cracks can form. However, it might be necessary for this stress to be reduced to less than f_{sy} in order to achieve acceptable crack widths. Equation 3.5.2(5) in association with Eq. 3.5.2(1) can be used to determine if this is necessary. Alternatively, a value of f_s can be estimated using the simplified design method described in the next Section.

It should be noted that for a state of tension, the minimum area of steel, $A_{st,min}$, determined using Eq. 3.5.3(1) would need to be distributed in the top and bottom faces of a slab.

3.6 Simplified Design Method in Eurocode 2

3.6.1 Minimum Reinforcement

It is a basic requirement of the design rules in Eurocode 2 for controlling cracking without requiring direct calculation, that the minimum area of reinforcement given by Eq. 3.5.3(1) is placed at the cross-section being designed. Since elements in a state of flexure must naturally be considered, the use of Eq. 3.5.3(1) needs to be broadened by the following additional description of some of its terms.

With reference to Eq. 3.5.3(1), it can be further written that:

- k_3 =1.0 for imposed deformation such as from support settlement; and
- k_4 =0.4 for pure bending, although a general equation is derived below for determining this value more accurately if desired (see Eq. 3.6.1(5)). Eurocode 2 does not provide values of k_4 for stress states other than pure bending ($k_4=0.4$) or pure tension ($k_4=1$), which is discussed in RCB-1.1(1).

For pure bending, immediately after the first crack is induced (assuming that the bending moment at the critical section does not reduce), it can be written that:

$$f_s = \frac{M_{cr}}{A_{st,min} z} \quad 3.6.1(1)$$

where z is the lever arm of the internal force couple.

However, immediately prior to cracking:

$$M_{cr} = f_t Z_t \quad 3.6.1(2)$$

where Z_t is the section modulus on the tension side of the uncracked section.

Substituting Eq. 3.6.1(1) into Eq. 3.6.1(2) and rearranging gives:

$$A_{st.min} = \frac{f_t Z_t}{f_s z} \quad 3.6.1(3)$$

From Eq. 3.5.3(1), with k_3 omitted it can be written that:

$$A_{st.min} = k_4 f_t A_{ct} / f_s \quad 3.6.1(4)$$

Substituting Eq. 3.6.1(3) into Eq. 3.6.1(4) and rearranging finally gives a general equation that applies for pure bending:

$$k_4 = \frac{Z_t}{A_{ct} z} \quad 3.6.1(5)$$

Considering a simple rectangular slab, ignoring the presence of the reinforcement when calculating Z_t and A_{ct} , i.e. $Z_t = bD^2/6$ and $A_{ct} = bD/2$, and assuming $z = 0.8D$ gives $k_4 = 0.42$. This explains the value of 0.4 in Eurocode 2.

3.6.2 Crack Width as a Function of Bar Diameter or Bar Spacing

The simplified design rules in Eurocode 2 are based on choosing an appropriate bar diameter or bar spacing. They will normally be preferred by design engineers rather than directly calculating crack width. They have been derived using the crack width formula Eq. 3.5.2(1) in parametric studies, as will be explained below.

Limits are placed on bar diameter and bar spacing to ensure that crack widths will not generally exceed 0.3 mm for reinforced-concrete elements. Specifically, it is stated that:

- (i) for cracking caused dominantly by restraint (herein interpreted to apply to slab sections in a state of flexure – see Note 1), the maximum diameters of deformed bars given in Table 3.6.2(1) (reproduced below) are not to be exceeded, where the steel stress, f_s , given in the table is the value obtained immediately after cracking (i.e. f_s in Eq. 3.5.3(1)); and
- (ii) for cracking caused dominantly by loading (herein interpreted to apply to slab sections in a state of flexure – see Note 2), either the maximum diameters of deformed bars given in Table 3.6.2(1) or their maximum spacings given in Table 3.6.2(2) (reproduced below) must be complied with. (Thus, in this case it is not necessary to satisfy both tables.)

Note 1: As shown in Fig. 3.4, restrained deformation can give rise to a state of flexure (Fig. 3.4(c)) or tension (Fig. 3.4(d)). It will be explained below that Table 3.6.2(1) was derived for a beam section in pure bending. It has been assumed that the table also applies to beams in tension as indicated in Eurocode 2. However, as explained in Section 3.4.1, slabs in a state of tension will not be designed using this approach. Therefore, it has been assumed that the table only applies to slabs in a state of flexure (see Section 5.3).

Note 2: For slabs, direct loading will normally give rise to a state of flexure (Fig. 3.4(a)). It will be seen from Table 3.6.2(2) that it covers both pure bending (column 2) and pure tension (column 3). It has been assumed that the values for pure bending are applicable for slabs in flexure (see Section 5.3).

For sections in a state of flexure, in Eurocode 2 the steel stress is calculated under the quasi-permanent combination of loading, i.e. long-term serviceability condition. Beeby and Narayanan [12] state that this stress can be estimated by multiplying the design yield strength of the steel (taken as 500/1.15 in Eurocode 2) by the ratio of the quasi-permanent load to the design ultimate load. However, this assumes that (i) the steel area is controlled by strength; (ii) the same load combinations apply for strength and serviceability design; and, most importantly for slabs, that (iii) the degree of moment redistribution at the strength limit state is zero. The designer should be aware of these assumptions. In particular, the designer may not know the amount of moment redistribution depending on the method of analysis being used to calculate the design action effects. An example of this is given in Section 7.3.

**Table 3.6.2(1) of Eurocode 2
Maximum Bar Diameters**

Steel stress (f_s) (MPa)	Maximum bar diameter (d_b) (mm)
450 (375)	6
400 (345)	8
360 (320)	10
320 (300)	12
280 (265)	16
240	20
200	25
160	32

Note: The values of stress shown in brackets are not given in Eurocode 2. They are the result of a parametric study described below, and are recommended for use, instead of the Eurocode 2 values, when designing slabs with an overall depth, D_s , less than or equal to 300 mm.

**Table 3.6.2(2) of Eurocode 2
Maximum Bar Spacings**

Steel stress (f_s) (MPa)	Maximum spacing – pure bending (mm)	Maximum spacing – pure tension (mm)
360	50	-
320	100	-
280	150	75
240	200	125
200	250	150
160	300	200

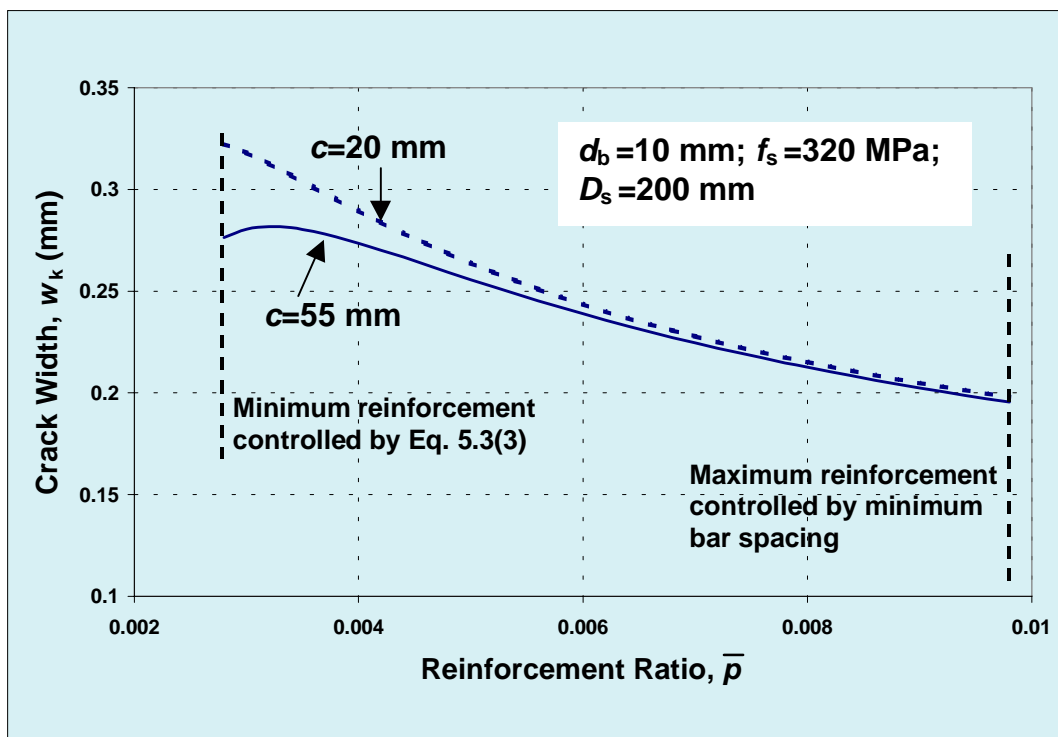
The background to both of these design tables from Eurocode 2 has been described in detail in Design Booklet RCB-1.1(1), which addresses the design of beams for crack control. In particular, the case of rectangular beams with an overall depth, D , of 400 mm was considered to be critical [11], crack control being less severe for deeper beams.

The overall depth, D_s , of slabs is generally less than 400 mm. Therefore, it was considered necessary to investigate the validity of using Tables 3.6.2(1) and 3.6.2(2) for their design. Firstly, a parametric study was conducted using the crack width formula Eq. 3.5.2(1). The following assumptions were made to firstly investigate whether the values of stress in Table 3.6.2(1) needed to be modified:

- (a) the concrete cover, c , to the longitudinal or transverse bars varied between 20 and 40 mm;
- (b) the diameter of the bars, d_b , varied between 6 and 20 mm;
- (c) the bars making up the main (longitudinal) reinforcement could be located in single layers in either the primary or secondary directions, whereby the dimension ($D_s - d$) varied between 23 and 70 mm but could not exceed $0.5D_s$;

- (d) the yield strength of the bars, f_{sy} , was 500 MPa;
- (e) the overall depth, D_s , of the slabs varied from 100 to 300 mm;
- (f) the characteristic compressive cylinder strength of the concrete, f_c , was either 25 or 32 MPa and the tensile strength, f_t , was 3.0 MPa;
- (g) the concrete was normal density with a density, ρ_c , of 2400 kg/m³;
- (h) the presence of any compression steel was ignored;
- (i) the area of tension steel, A_{st} , per unit length of slab was varied (for a given bar diameter), while still satisfying the minimum and maximum bar spacings appropriate for slabs (see Sections 5.4 and 5.3, respectively), as well as satisfying the minimum reinforcement requirements of Eqs 5.3(2) and 5.3(3) and not exceeding a k_u value of 0.4 appropriate for under-reinforced cross-sections;
- (j) the reinforcement was treated as bars not mesh, even though solutions for bars less than 10 mm in diameter were not possible if the bar spacing did not match that of a standard mesh;
- (k) in Eq. 3.5.2(4), $k_1=0.8$ and a state of flexure was assumed whereby $k_2=0.5$ and $A_{c,eff}$ was calculated in accordance with Fig. 3.5(a); and
- (l) in Eq. 3.5.2(5), $\beta_1=1.0$, $\beta_2=0.5$, $f_{sr}=nM_{cr}(d-d_n)/I_{cr}$ (where M_{cr} equals $f_t Z_t$), and by trial and error the maximum value of f_s ($=nM_s^*(d-d_n)/I_{cr}$) was determined for a given bar diameter, d_b , to keep the maximum value of w_k equal to about 0.30 mm, over the full range of values of the parameters described above (see Fig. 3.6 for an example). The serviceability bending moment, M_s^* , was approximately calculated as $f_s/f_{sy} \times (1/\phi)$, where $\phi=0.8$ for bending of under-reinforced cross-sections. Thus, moment redistribution was ignored.

The results of the study showed that the steel stresses in Table 3.6.2(1) of Eurocode 2 had to be reduced for bars less than 20 mm in diameter, for slabs with an overall depth, D_s , of up to 300 mm. The values of steel stress recommended have been added to Table 3.6.2(1) and are shown in brackets. A typical, critical result obtained from the study is shown in Fig. 3.6.



**Figure 3.6 Derivation of Modifications to Table 3.6.2(1) from Eurocode 2
for Slabs in Pure Bending and Reinforced in One Layer**

The crack width formula in Eurocode 2 (Eq. 3.5.2(1)) was also applied directly to investigate the validity of using Table 3.6.2(2) to design slabs in a state of flexure. For this purpose, it was assumed that a reasonably critical case was defined by $D_s=200$ mm, $c=20$ mm, $f_t=3.0$ MPa, and the steel reinforcement ratio was calculated as:

$$\bar{\rho} = \frac{A_b}{s_b D} = \frac{\pi d_b^2}{4 s_b D} \quad 3.6.2(1)$$

where A_b and s_b are the cross-sectional area and spacing of the bars, respectively.

For each value of steel stress, f_s , in Table 3.6.2(2), the crack width was calculated using Eq. 3.5.2(1) for each bar spacing in the middle column of the table, and for each bar diameter in Table 3.6.2(1). The results of these calculations are shown in Fig. 3.7. It can be noted that all non-practical solutions have been excluded. For example, slabs that do not satisfy either the minimum or maximum reinforcement requirements described above for the parametric investigation into Table 3.6.2(1). In this way, solutions leading to negative values of w_k have been omitted. It is apparent from Fig. 3.7 that designs based on Table 3.6.2(2) for pure bending give maximum crack widths of around 0.30 mm, and should generally be satisfactory. Therefore, no change to this table has been proposed in this design booklet. However, it should not be overlooked that by using this table from Eurocode 2, the value of the reinforcement ratio, $\bar{\rho}$, can be quite high, particularly if large diameter bars are used at close centres. This implies that solutions satisfying Table 3.6.2(2) will not necessarily be practical or economical, especially for thin slabs.

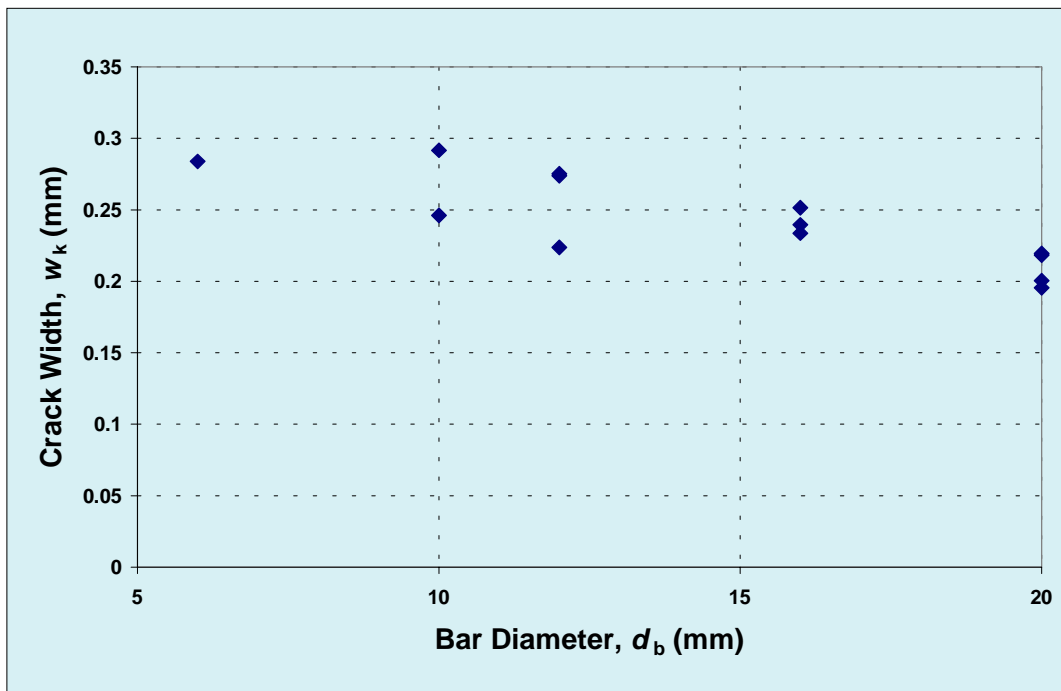


Figure 3.7 Check on Table 3.6.2(2) from Eurocode 2 for Slabs in Pure Bending

3.7 Other Relevant Design Requirements in Eurocode 2

3.7.1 Introduction

As stated in Section 3.5.1, in Eurocode 2 cracking of slabs less than 200 mm deep and in a state of flexure (i.e. without significant axial tension) is assumed to be controlled if the detailing rules in Clause 5.4.3 “Cast In-situ Solid Slabs” of Eurocode 2 are satisfied. According to this approach, it is not necessary to calculate crack widths. Nor is it necessary to satisfy either Table 3.6.2(1) or Table 3.6.2(2). However, it was felt that slabs normally develop significant axial tension due to concrete shrinkage and restraint, and that, therefore, it would be unconservative for designers to assume

otherwise. Thus, it was felt that this concession could be misleading and that the proposed details would be insufficient in themselves to overcome the types of problems observed in the field (see Fig. 1.2). Therefore, the approach was not included in AS 3600-2000 (see Section 5). Nevertheless, some of the more worthy detailing rules for slabs proposed in Eurocode 2 are repeated here for consideration by designers. Some are met when using the rules in AS 3600-2000, which are summarised in Section 5.3.

3.7.2 Minimum Reinforcement Percentage (Clause 5.4.2.1.1(1))

The cross-sectional area of the longitudinal tension reinforcement must not be less than that required to control cracking, nor less than $0.0015b_t d$ (intended to avoid brittle failure or sudden collapse) where b_t is the mean width of the tension zone, cf. $A_{st}/bd=0.8/f_{sy}$ derived from Eq. 3.3.3.2(2) in Section 3.3.3.2.

3.7.3 Reinforcement in Slabs Near Supports (Clause 5.4.3.2.2(2))

When partial fixity occurs along a side of a slab, but is not taken into account in the analysis, then the top reinforcement should be designed for a bending moment of at least 25 per cent of the maximum (positive) bending moment in the adjacent span.

3.7.4 Avoiding Yielding of Reinforcement (Clause 4.4.1.1(6-7))

An essential condition for the crack width formulae given in Section 3.5.2 to be valid is that the reinforcement remains elastic. The condition immediately after each new crack is formed is critical, since yielding at a crack may prevent further cracks from forming. Also, should yielding, (or more precisely, significant non-linearity along the stress-strain relationship) occur at a crack at any stage, then this crack will become excessively wide and render the structure unserviceable.

Stresses in the steel under serviceability conditions that could lead to inelastic deformation of the reinforcement should be avoided as this will lead to large, permanently open cracks. It is assumed that this requirement will be satisfied, provided under the rare combination of loads (i.e. full, unfactored dead and live loads) that the tensile stress in the reinforcement does not exceed $0.8f_{sy}$. The intention here is that the effect of actions ignored in design (such as restrained deformation) will not be enough to cause the steel to yield. When a major part of the stress is due to restrained deformation, e.g. restrained shrinkage, then a maximum stress of f_{sy} is deemed acceptable in Eurocode 2.

4. DESIGN APPROACHES

4.1 General

The simplified design rules for crack control in Eurocode 2 give rise to a minimum area of reinforcement and a limitation on bar diameter or bar spacing, depending on the magnitude of the steel stress calculated under service loads. They have been proposed for inclusion in AS 3600-2000. This has principally come about with the move from 400 MPa to 500 MPa grade reinforcing bars, and 450 MPa to 500 MPa mesh. This is because an increase in the design yield stress of the steel is basically intended to reduce the area of steel required at the strength limit state. This in turn increases the need to have reliable design methods for ensuring that the structure will remain serviceable. This is because reducing the amount of steel in a reinforced-concrete slab or beam, even if it is of higher yield strength, will generally increase the possibility of serviceability problems such as cracking.

A major objective of the design approach presented in this booklet is to achieve a significant reduction in the area of reinforcing steel (nominally up to 20 per cent) required when designing slabs for bending strength and crack control. This will require the design engineer to thoroughly understand the effect that bar spacing and bar diameter can have on the maximum allowable steel stress, while still keeping crack widths to an acceptable level. Consideration should also be given to other design criteria such as deflection control.

If moment redistribution is ignored at the strength limit state, at critical sections in bending the stress in the steel under serviceability conditions can be approximately calculated using the equation:

$$f_s = \phi f_{sy} \frac{G + \psi_s Q}{1.25G + 1.5Q} \quad 4.1(1)$$

where ϕ is the capacity factor for bending (normally 0.8) and ψ_s is the short-term live load factor (see Section 3.6.2 of RCB-1.1(1) for explanation in choosing short-term rather than long-term loading). Therefore, at critical locations the serviceability steel stress, f_s , can vary between about 0.4 and 0.65 f_{sy} , depending on the ratio of dead and live loads. Thus, f_s can vary from 180 to 293 MPa for $f_{sy}=450$ MPa, and from 200 to 325 MPa for $f_{sy}=500$ MPa.

If moment redistribution is taken into account, Eq. 4.1(1) can be generalised further. Let η equal the degree of moment redistribution (away from a critical moment region) assumed during strength design. It will be assumed that $\eta=0$ implies no redistribution, and $\eta=1$ means 100% redistribution, noting that at least for elastic design, AS 3600 limits the absolute value of η to 0.3. Then it can be approximately written that:

$$f_s = \phi f_{sy} \frac{G + \psi_s Q}{1.25G + 1.5Q} \frac{1}{(1 - \eta)} \quad 4.1(2)$$

If $\eta=0.3$, it is clear that the serviceability steel stress, f_s , can be more than 40 percent higher than the values just cited, conceivably reaching 0.93 f_{sy} or 465 MPa for 500 MPa steel. Naturally η can be even higher if plastic methods of analysis, such as yield line theory, are used to calculate design action effects (see Section 7.3).

In view of these potentially high values of f_s , the discussion in Section 3.6.2 and the information contained in Tables 3.6.2(1) and 3.6.2(2) of Eurocode 2, it is clear that the maximum bar spacing specified for slabs in AS 3600-1994 (viz. lesser of 500 mm and $2.5D_s$ – see Sections 4.3 and 5.2) cannot be relied upon to control cracking. As illustrated by Fig. 1.2, there are many avoidable cases of excessively cracked structures. The rules in AS 3600-1994 regarding crack control of slabs in flexure have been over-simplified, and the necessary checks on serviceability steel stresses are not being made. This is shown by the flowcharts in Section 4.3 where the design approach in AS 3600-1994 is presented, as well as the new improved approach proposed for AS 3600-2000.

4.2 Simplified Design Rules vs Crack Width Calculation

It is explained in the commentary to AS 3600-1994 (AS 3600 Supp1-1994) that the calculation of crack widths can be used as an alternative procedure to the specific detailing rules for controlling cracking. The conditions that apply for when this more detailed design work might be required are not explained. However, it is stated in the commentary that designers should aim to minimise the cover and distance between bars to control flexural crack widths. It follows that if cover or bar spacing was larger than normal, then designers should have been cautious and calculated crack widths.

Reference is made in the commentary to AS 3600-1994 to accepted procedures in the American and British Standards: an earlier version of the current ACI 318 [19] (although the method has remained largely unchanged); and BS 8110: Part 2 [21]. It can be assumed that the design procedure in Eurocode 2 will eventually supersede that in BS 8110: Part 2, when Eurocode 2 becomes more widely adopted.

Designers generally prefer not having to calculate crack widths. This is partly because these calculations can cause the design to become iterative in nature. The simplified method in Eurocode 2 described in Section 3.6 provides reasonable solutions directly without iteration. However, it does require steel stresses under serviceability conditions to be calculated, which merits the use of computer software. Computer software also allows multiple solutions to be considered without requiring tedious hand calculations. Computer program 500PLUS-SCC™ (Slab Crack Control) has been developed for this purpose and is described in Section 6.

4.3 Flowcharts for Simplified Design Approaches

AS 3600 - 1994

The rules in AS 3600-1994 for crack control design of slabs in flexure are stated in Section 5.2. The design approach is very straightforward because maximum bar spacing is the only criterion considered. It is presented in the flowchart in Fig. 4.1. Crack control for shrinkage and temperature effects must be considered separately in accordance with Clause 9.4 of AS 3600-1994, noting that Clause 9.4.3.2 may require the reinforcement in the primary direction (for flexure) to be increased depending on the degree of restraint to in-plane movement.

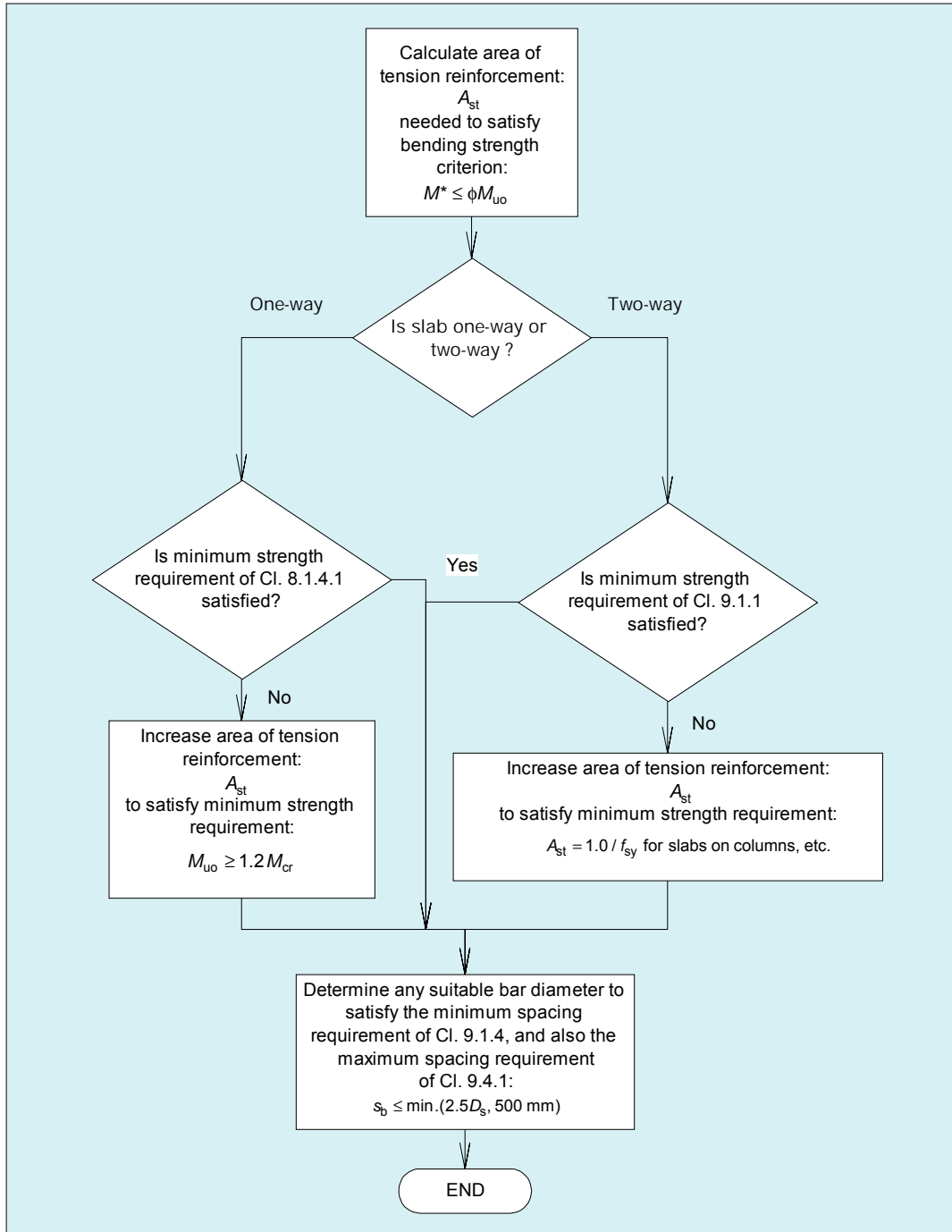
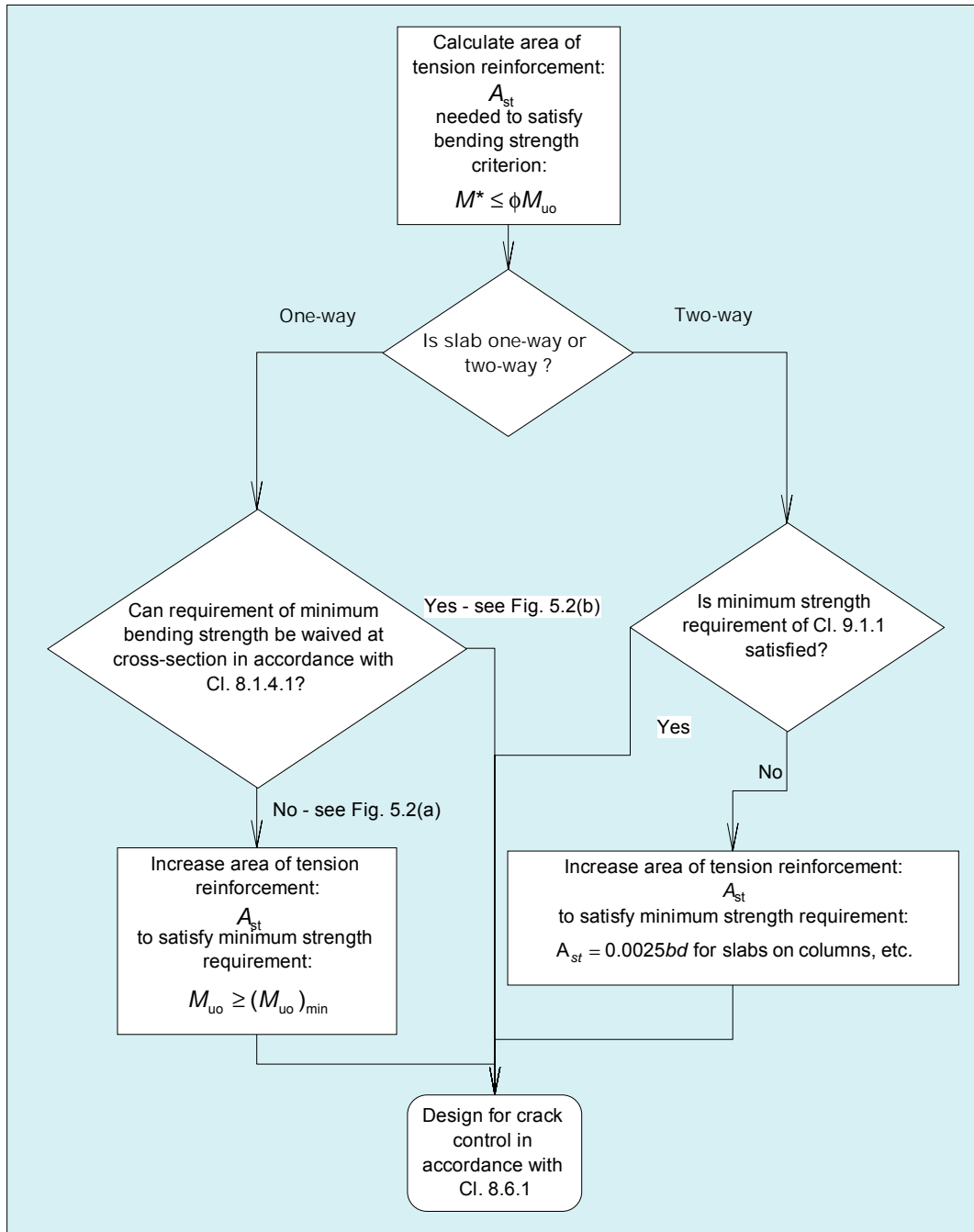


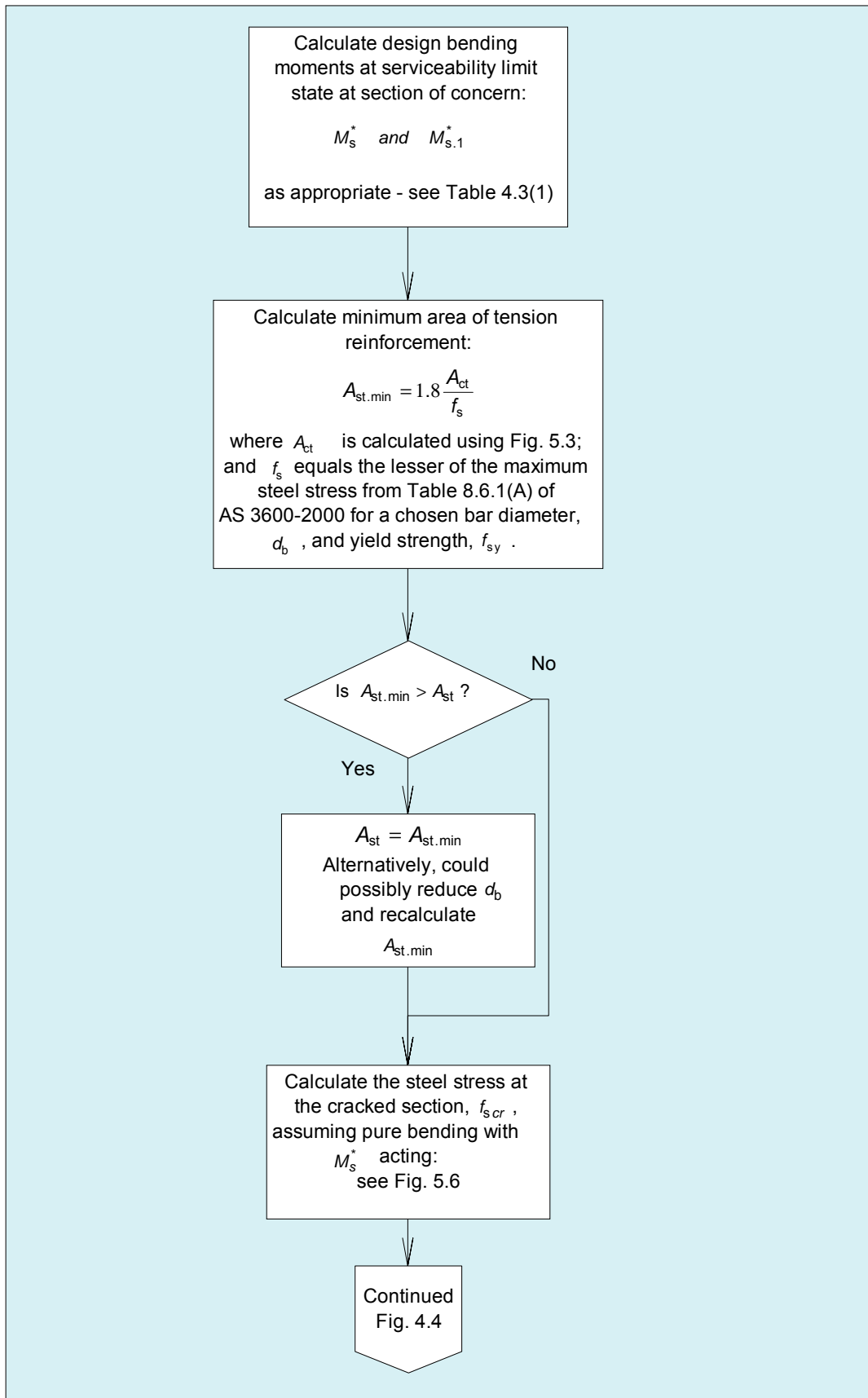
Figure 4.1 Flowchart of Simplified Design Approach in AS 3600-1994 for Crack Control of Reinforced Slabs in Flexure

AS 3600 - 2000

The rules for crack control design of slabs in flexure proposed for inclusion in AS 3600-2000 are stated in Section 5.3. The design approach is explained in the flowcharts in Figs 4.2, 4.3 and 4.4. Crack control for shrinkage and temperature effects must be considered separately in accordance with Clause 9.4 of AS 3600-2000, noting that Clause 9.4.3.2 may require the reinforcement in the primary direction (for flexure) to be increased depending on the degree of restraint to in-plane movement.



**Figure 4.2 Flowchart of Simplified Design Approach in AS 3600-2000
 – Part A: Minimum Strength Requirements for Slabs in Flexure**



**Figure 4.3 Flowchart of Simplified Design Approach in AS 3600-2000
– Part B: Crack Control for Flexure (cont.)**

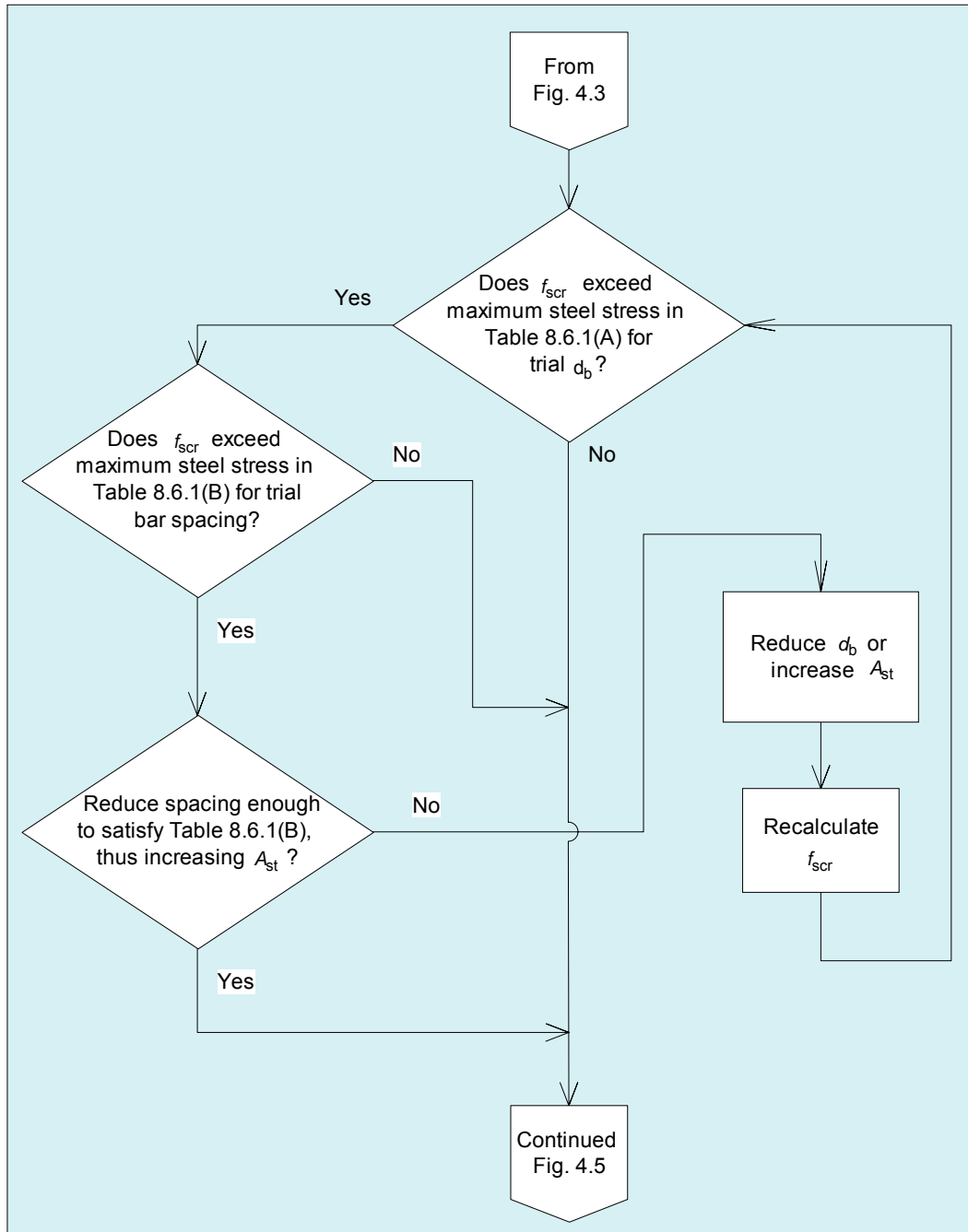


Figure 4.4 Flowchart of Simplified Design Approach in AS 3600-2000
– Part B: Crack Control for Flexure (cont.)

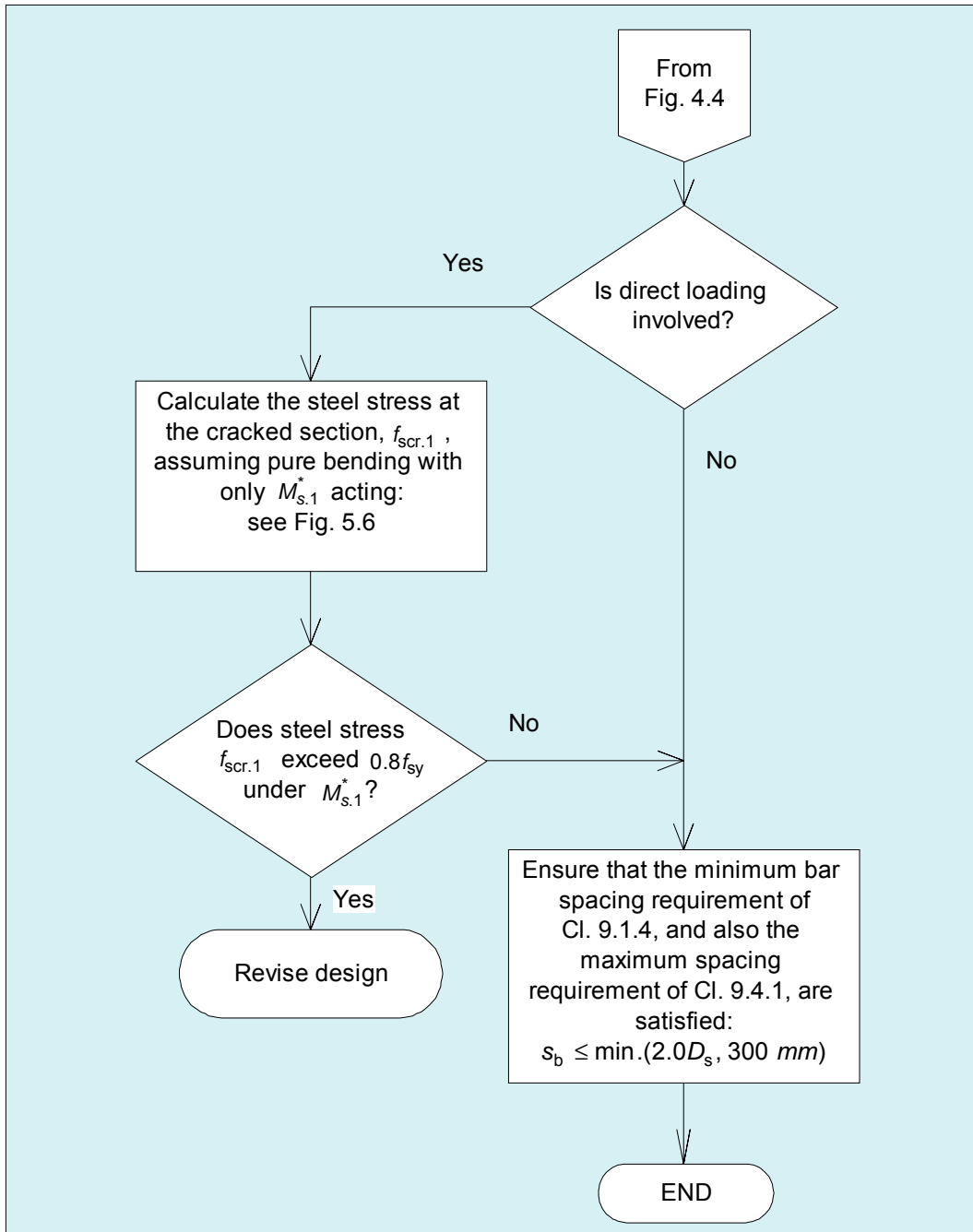


Figure 4.5 Flowchart of Simplified Design Approach in AS 3600-2000
– Part B: Crack Control for Flexure

**Table 4.3(1)
Design Action Effects at Serviceability Limit State for
Crack Control Design of Slabs in Flexure
(Refer to Figs 4.3 and 4.5)**

State	Design action (see Fig. 3.4)	Design action effects	
		Uncracked section (Note 1)	Cracked section (Note 2)
Flexure	Direct loading	M_s^*	$M_s^*, M_{s,1}^*$ (Note 3)
“	Imposed deformation	M_s^*	M_s^*
“	Restrained deformation	M_s^*	M_s^*

Notes:

1. The design action effect/s that act on the cracked section are used to calculate the stress in the steel under serviceability conditions. They are normally assumed to equal the design action effects that act on the uncracked section, which in turn may be calculated directly from the values determined for the strength limit state (see Eq. 4.1(2)), provided that η is known (see Section 7.3).
2. Under direct loading, the stresses in the steel under both $G+\psi_s Q$ (corresponding to M_s^*) and $G+Q$ (corresponding to $M_{s,1}^*$) must be checked.

5. DESIGN RULES

5.1 General

The existing rules in AS 3600-1994 and the rules proposed for AS 3600-2000³ relating to the design of slabs for crack control are presented in this Section. In AS 3600-1994, for slabs in flexure it was deemed sufficient to limit the maximum spacing of the main tension bars. However, it is proposed in AS 3600-2000 that steel stresses under serviceability conditions will have to be calculated for slabs in flexure using elastic cracked-section theory, and then a suitable bar diameter or bar spacing chosen. The equations needed to perform these stress calculations are provided in this Section as an aid to designers. The rules for designing slabs for crack control for shrinkage and temperature effects (Clause 9.4.3) have essentially remained unchanged.

5.2 AS 3600 - 1994

The design rules in AS 3600-1994 relevant to crack control of slabs in flexure are as follows. Background information to the rules can be found in the commentary to AS 3600-1994 (AS 3600 Supp1).

Clause 2.4.4 Cracking (Clause 2.4, Design for Serviceability)

The cracking of beams or slabs under service conditions shall be controlled in accordance with the requirements of Clause 8.6 or 9.4 as appropriate.

It follows from Clause 9.1.1 (see below) that the following clause for beams also applies to the design of one-way reinforced slabs for strength in bending.

Clause 8.1.4.1 General (Clause 8.1.4, Minimum Strength Requirements)

The ultimate strength in bending, M_{uo} , at critical sections shall be not less than 1.2 times the cracking moment, M_{cr} , given by-

$$M_{cr} = Z(f'_{cf} + P/A_g) + Pe \quad 5.2(1)$$

where –

- Z = section modulus of the uncracked section, referred to the extreme fibre at which flexural cracking occurs; and
- f'_{cf} = characteristic flexural tensile strength of the concrete.

For the purpose of this Clause, the critical section to be considered for negative moment shall be the weakest section in the negative moment region (i.e. where $\phi M_{uo}/M^*$ is least).

For rectangular reinforced concrete sections, this requirement shall be deemed to be satisfied if minimum tension reinforcement is provided such that –

$$A_{st}/bd \geq 1.4/f_{sy} \quad 5.2(2)$$

For reinforced T-beams or L-beams where the web is in tension, b shall be taken as b_w .

³ AS 3600-2000 is expected to be published this year by Standards Australia. The design rules proposed by the authors for possible inclusion in AS 3600-2000 are presented in Section 5.3. They are the result of further development work performed after the rules were first drafted, and released publicly in the Standards Australia Public Comment Draft DR 99193 CP, Amendment 2 to AS 3600. The clauses proposed in Section 5.3 can be compared directly with clauses with the same numbers in DR 99193 CP, which are reproduced in Appendix D of design booklet RCB-1.1(1). Any differences between the clauses in Section 5.3 and Appendix D of RCB-1.1(1) reflect the authors' latest recommendations.

Clause 9.1.1 General (Clause 9.1, Strength of Slabs in Bending)

The strength of a slab in bending shall be determined in accordance with Clauses 8.1.1 to 8.1.6 except that for two-way reinforced slabs, the minimum strength requirements of Clause 8.1.4.1 shall be deemed to be satisfied by providing minimum tensile reinforcement such that A_{st}/bd is not less than one of the following:

- (a) Slabs supported by columns 1.0/ f_{sy}
- (b) Slabs supported by beams or walls 0.8/ f_{sy}
- (c) Slab footings 0.8/ f_{sy}

Clause 9.1.4 Spacing of reinforcement and tendons (Clause 9.1, Strength of Slabs in Bending)

The minimum clear distance between parallel bars (including bundled bars), ducts and tendons shall be such that the concrete can be properly placed and compacted in accordance with Clause 19.1.3 (Handling, Placing and Compacting of Concrete). The maximum spacing of longitudinal reinforcement and tendons shall be determined in accordance with Clause 9.4 (Crack Control of Slabs).

Clause 9.4.1 Crack control for flexure in reinforced slabs (Clause 9.4, Crack Control of Slabs)

Flexural cracking in reinforced slabs, shall be deemed to be controlled if the centre-to-centre spacing of bars in each direction does not exceed the lesser of 2.5D or 500 mm. For the purpose of this Clause, bars with a diameter less than half the diameter of the largest bar in the cross-section shall be ignored.

Clause 9.4.3 Crack control for shrinkage and temperature effects (Clause 9.4, Crack Control of Slabs)

For brevity, see AS 3600-1994.

Clause 9.4.4 Crack control in the vicinity of restraints (Clause 9.4, Crack Control of Slabs)

In the vicinity of restraints, special attention shall be paid to the internal forces and cracks which may be induced by prestressing, shrinkage or temperature.

Clause 9.4.5 Crack control at openings and discontinuities (Clause 9.4, Crack Control of Slabs)

For crack control at openings and discontinuities in a slab, additional properly-anchored reinforcement shall be provided if necessary.

5.3 AS 3600 – 2000 (Proposed Rules)

A simplified approach to design for crack control in Eurocode 2 – Clause 4.4.2.3 Control of Cracking without Direct Calculation, which avoids the calculation of crack widths, has been proposed for inclusion in AS 3600-2000, although with some modifications. The revised rules in AS 3600-2000 proposed for design for crack control of slabs in flexure are as follows. Design of slabs for crack control for shrinkage and temperature effects shall be deemed to be met by Clause 9.4.3.

Clause 2.4.4 Cracking (Clause 2.4, Design for Serviceability)

The cracking of beams or slabs under service conditions shall be controlled in accordance with the requirements of Clause 8.6 or 9.4 as appropriate.

It follows from Clause 9.1.1 (see below) that the following clause for beams also applies for the design of one-way reinforced slabs.

Clause 8.1.4.1 General (Clause 8.1.4, Minimum Strength Requirements)

The ultimate strength in bending, M_{uo} , at critical sections shall not be less than $(M_{uo})_{min}$, given by-

$$(M_{uo})_{min} = 1.2 [Z (f'_{cf} + P/A_g) + Pe] \quad 5.3(1)$$

where –

- Z = section modulus of the uncracked section, referred to the extreme fibre at which flexural cracking occurs;
- f'_{cf} = characteristic flexural tensile strength of the concrete; and
- e = the eccentricity of the prestressing force (P), measured from the centroidal axis of the uncracked section.

This requirement may be waived at some critical sections of an indeterminate member provided it can be demonstrated that this will not lead to sudden collapse of a span (see Fig. 5.2).

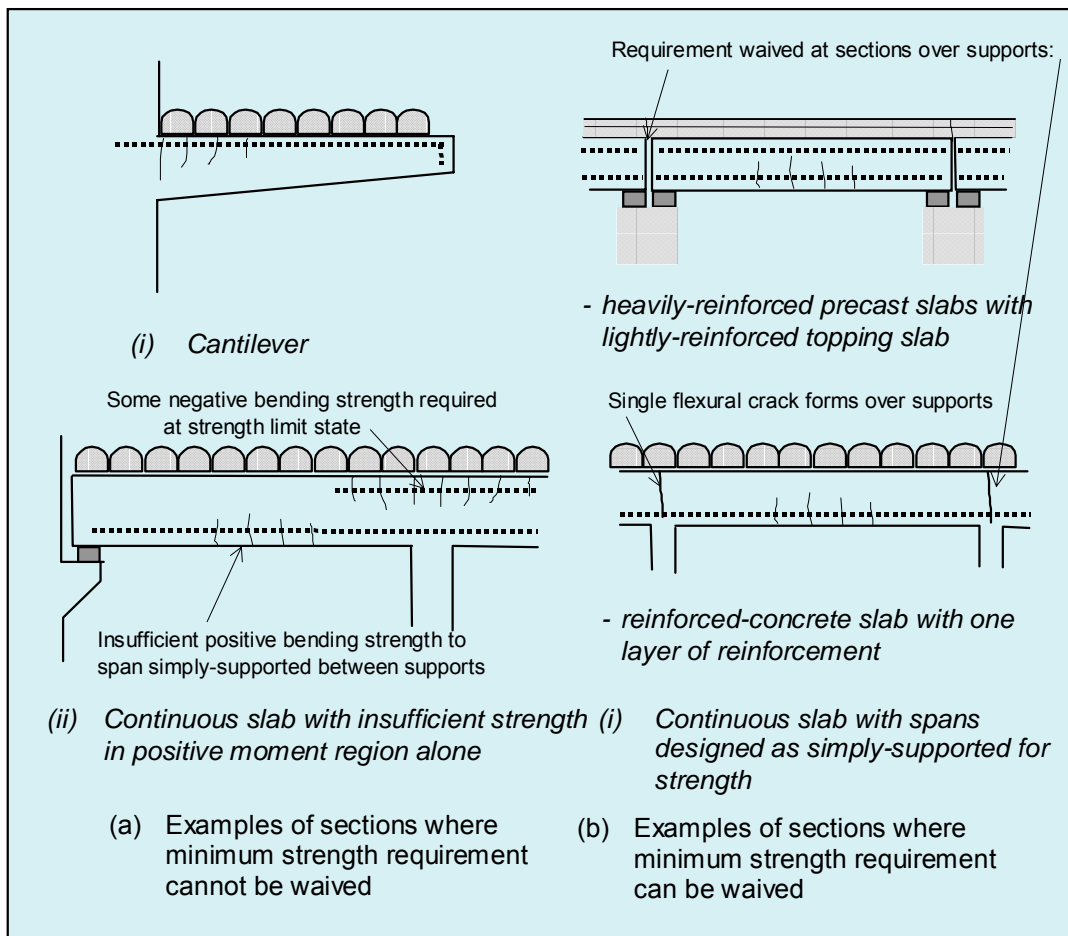


Figure 5.2 Sudden Collapse and the Minimum Strength Requirement for One-Way Reinforced Slabs

For prismatic, rectangular reinforced concrete sections, the requirement that $M_{uo} \geq (M_{uo})_{min}$ shall be deemed to be satisfied for the direction of bending being considered if minimum tension reinforcement is provided such that –

$$A_{st}/bd \geq 0.22 (D/d)^2 f'_{cf}/f_{sy} \quad 5.3(2)$$

For reinforced T-beams or L-beams where the web is in tension, b shall be taken as b_w .

Clause 8.6.1 Crack control for tension and flexure in reinforced beams (Clause 8.6, Crack Control of Beams)

Cracking in reinforced beams subject to tension or flexure shall be deemed to be controlled if the appropriate requirements in items (a), (b) and (c), and either item (d) for beams primarily in tension or item (e) for beams primarily in flexure, are satisfied. In cases when the reinforcement has different yield strengths, in this clause the yield strength (f_{sy}) shall be taken as the lowest yield strength of any of the reinforcement.

For the purpose of this Clause, the resultant action is considered to be *primarily tension* when the whole of the section is in tension, or *primarily flexure* when the tensile stress distribution within the section prior to cracking is triangular with some part of the section in compression.

- (a) The minimum area of reinforcement in the tensile zone ($A_{st,min}$) shall be:

$$A_{st,min} = 3 k_s A_{ct} / f_s \quad 5.3(3)$$

where –

- k_s = a coefficient that takes into account the shape of the stress distribution within the section immediately prior to cracking, as well as the effect of non-uniform self-equilibrating stresses, and equals 0.8 for tension and 0.6 for flexure;
- A_{ct} = the area of concrete in the tensile zone, being that part of the section in tension assuming the section is uncracked (see Fig. 5.3 for the case of flexure in a solid rectangular slab); and
- f_s = the maximum tensile stress permitted in the reinforcement immediately after formation of a crack, which shall be the lesser of the yield strength of the reinforcement (f_{sy}) and the maximum steel stress given in Table 8.6.1(A) of AS 3600 (see also Fig. 5.4) for the largest nominal diameter (d_b) of the bars in the section.

Note: This equation has been derived from Eq. 3.5.3(1) by putting $f_t=3.0$ MPa, and $k_s=k_3 \times k_4$, whereby $k_3=0.8 \times 1.0$ for a state of tension, and $k_3=1.0 \times 0.6$ for a state of flexure (see Section 3.6.1).

- (b) The distance from the side or soffit of a beam to the centre of the nearest longitudinal bar does not exceed 100 mm. Bars with a diameter less than half the diameter of the largest bar in the section shall be ignored. The centre-to-centre spacing of bars near a tension face of the beam shall not exceed 300 mm.
- (c) When direct loads are applied:
- the load combination for serviceability design with short-term effects shall be used in items (d) and (e) as appropriate, when calculating the steel stress, f_{scr} ; and
 - the steel stress assuming a cracked section, f_{scr} , shall also be calculated using the load combination for serviceability design with short-term effects, but with load factors of unity, and for this case shall not exceed a stress of $0.8f_{sy}$.
- (d) For beams subject to tension, the steel stress (f_{scr}) calculated assuming the section is cracked does not exceed the maximum steel stress given in Table 8.6.1(A) for the largest nominal diameter (d_b) of the bars in the section.
- (e) For beams subject to flexure, the steel stress (f_{scr}) calculated assuming the section is cracked does not exceed the maximum steel stress given in Table 8.6.1(A) for the largest nominal diameter (d_b) of the bars in the tensile zone. Alternatively, the steel stress does not exceed the maximum stress determined from Table 8.6.1(B) for the largest centre-to-centre spacing of adjacent parallel bars in the tensile zone (see Fig. 5.5). Bars with a diameter less than half the diameter of the largest bar in the section shall be ignored when determining spacing. (See Fig. 5.6 for equations to calculate f_{scr} for a solid rectangular slab.)

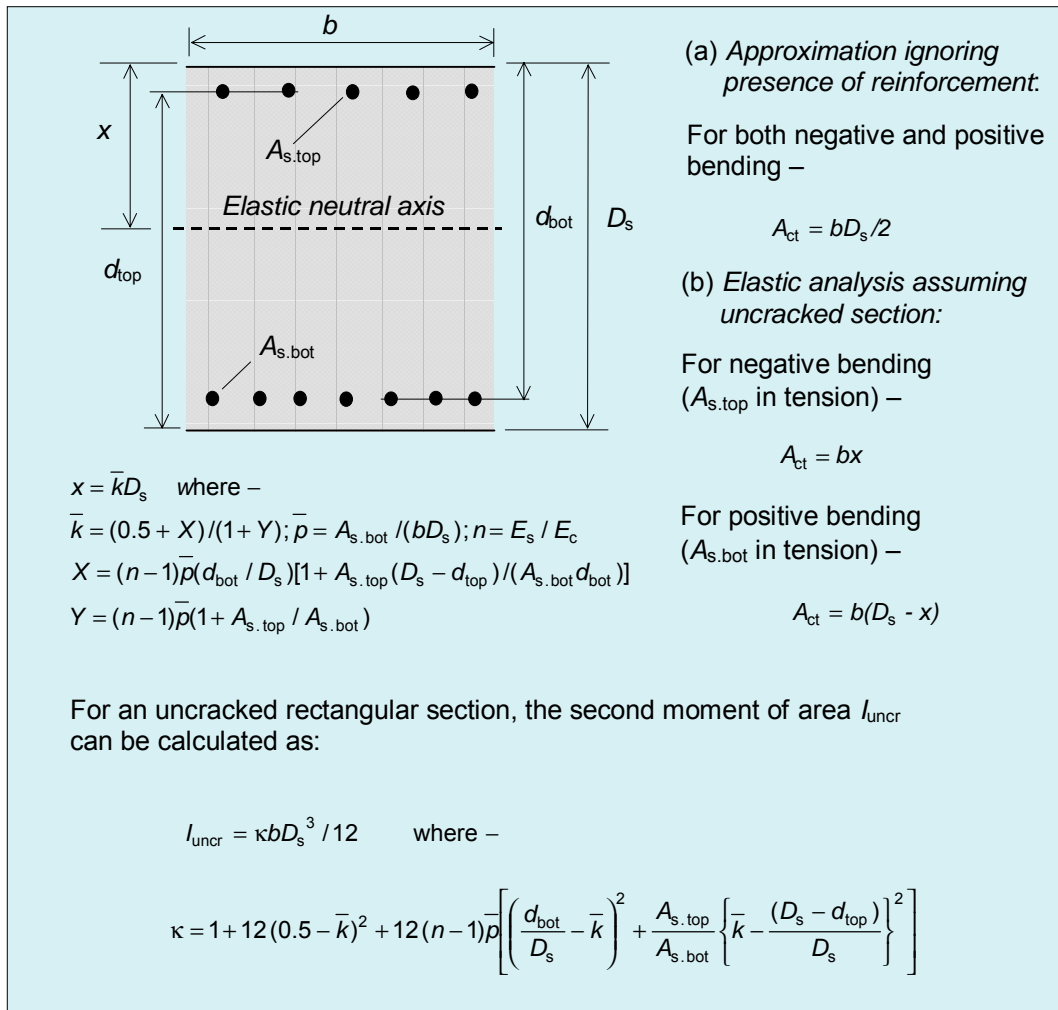


Figure 5.3 Area of Concrete (A_{ct}) in Tensile Zone of a Solid Slab

TABLE 8.6.1(A) of AS 3600 – 2000, **Modified for Slabs Only**
MAXIMUM STEEL STRESS FOR TENSION OR FLEXURE (see Fig. 5.4)

Nominal bar diameter (d_b) (mm)	Maximum steel stress (MPa) for overall depth, D_s (mm) of:	
	≤ 300 mm	> 300 mm
6	375	450
8	345	400
10	320	360
12	300	330
16	265	280
20	240	
24	210	
28	185	
32	160	
36	140	
40	120	

NOTES:

- Values of stress shown in the shaded area are recommended in this design booklet for slabs when $D_s \leq 300$ mm and $d_b < 20$ mm. (See Section 3.6.2 for derivation. The crack width equation Eq. 3.5.2(1) was used, with the values of the parameters in this equation calculated for a solid slab. Therefore, this change does not apply to beams, irrespective of their depth.
- The values in the table can be accurately calculated using the equations (also see dashed curves in Fig. 5.4):

Maximum steel stress equals:

$$-173 \log_e (d_b) + 760 \text{ MPa for } d_b \geq 20 \text{ mm}$$

$$-173 \log_e (d_b) + 760 \text{ MPa for } d_b < 20 \text{ mm and } D_s > 300 \text{ mm}$$

$$-114 \log_e (d_b) + 580 \text{ MPa for } d_b < 20 \text{ mm and } D_s \leq 300 \text{ mm}$$

- Sizes 6 and 8 mm are not available as separate bars.

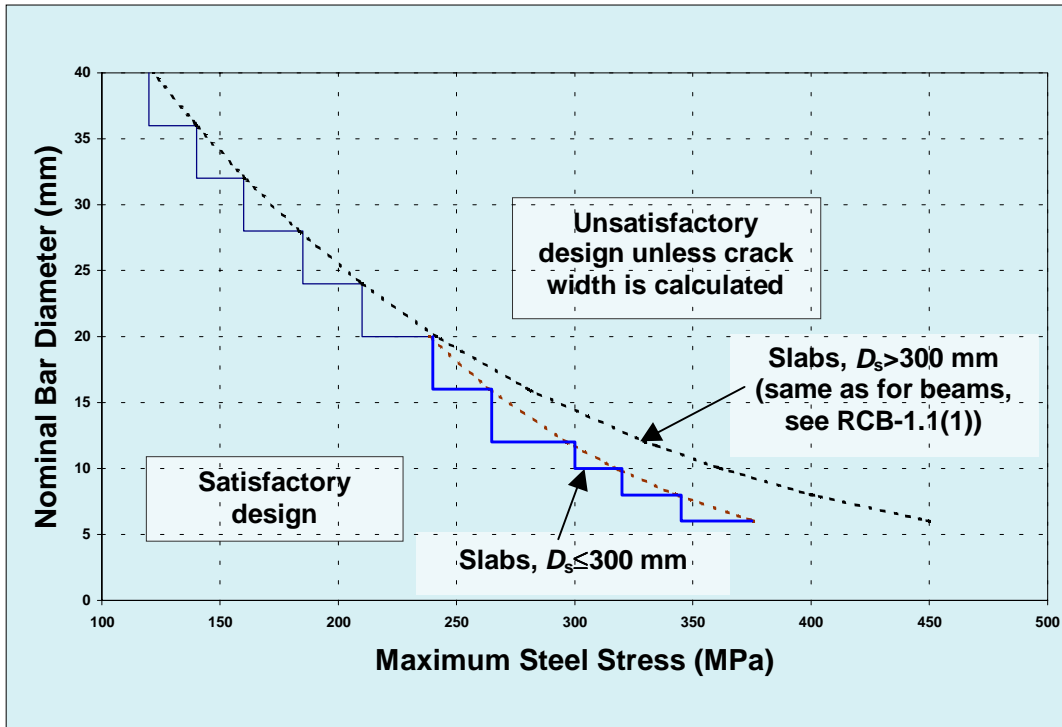


Figure 5.4 Maximum Steel Stress as a Function of Nominal Bar Diameter (Tension or Flexure) – Table 8.6.1(A) of AS 3600-2000, Modified for Slabs According to Their Overall Depth, D_s

TABLE 8.6.1(B) of AS 3600 - 2000
 MAXIMUM STEEL STRESS FOR FLEXURE
 (see Fig. 5.5)

Centre-to-centre spacing (mm)	Maximum steel stress (MPa)
50	360
100	320
150	280
200	240
250	200
300	160

NOTE: Linear interpolation may be used with the equation:

$$\text{Maximum steel stress} = -0.8 \times \text{centre-to-centre spacing} + 400 \text{ MPa}$$

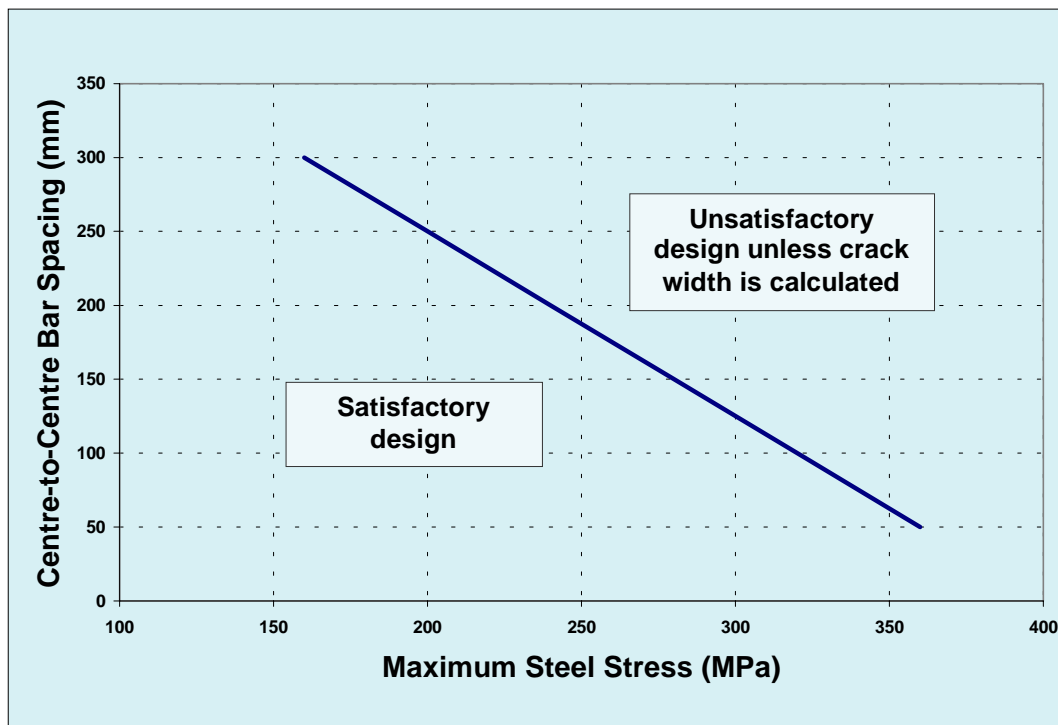


Figure 5.5 Centre-to-Centre Bar Spacing as a Function of Maximum Steel Stress (Flexure)
 – Table 8.6.1(B) of AS 3600-2000

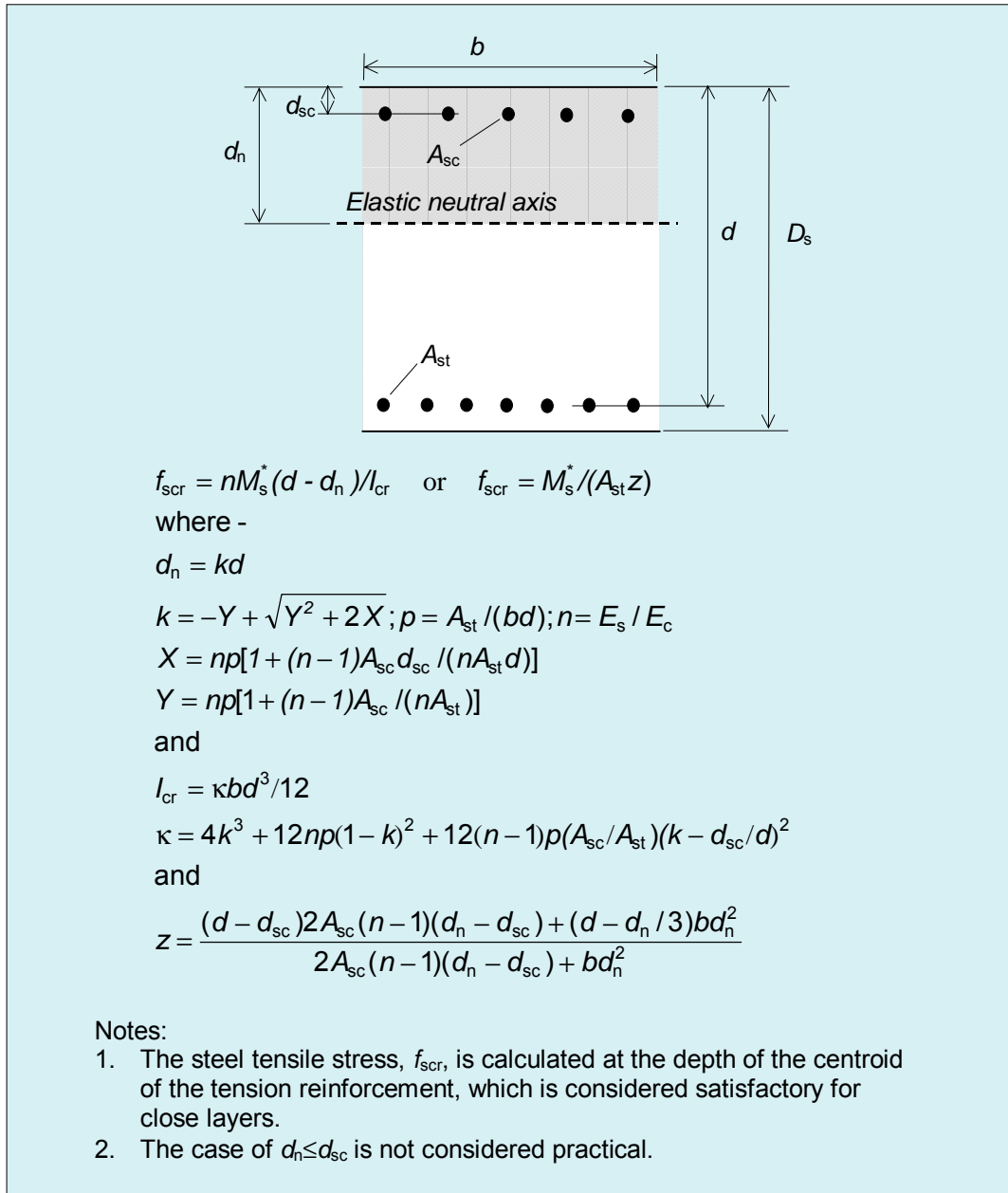


Figure 5.6 Calculation of f_{scr} for a Rectangular Section including a Solid Slab – Positive or Negative Bending

Clause 9.1.1 General (Clause 9.1, Strength of Slabs in Bending)

The strength of a slab in bending shall be determined in accordance with Clauses 8.1.1 to 8.1.6 except that for two-way reinforced slabs, the minimum strength requirements of Clause 8.1.4.1 shall be deemed to be satisfied by providing minimum tensile reinforcement such that A_{st}/bd is not less than one of the following:

- (a) Slabs supported by columns 0.0025
- (b) Slabs supported by beams or walls 0.0020
- (c) Slab footings 0.0020

Note: These values, now independent of f_{sy} , were calculated using the formulae in AS 3600-1994 directly and assuming $f_{sy}=400$ MPa.

Clause 9.1.4 Spacing of reinforcement and tendons (Clause 9.1, Strength of Slabs in Bending)

The minimum clear distance between parallel bars (including bundled bars), ducts and tendons shall be such that the concrete can be properly placed and compacted in accordance with Clause 19.1.3 (Handling, Placing and Compacting of Concrete). The maximum spacing of longitudinal reinforcement and tendons shall be determined in accordance with Clause 9.4 (Crack Control of Slabs).

Clause 9.4.1 Crack control for flexure in reinforced slabs (Clause 9.4, Crack Control of Slabs)

Cracking in reinforced slabs shall be deemed to be controlled if the following requirements are satisfied:

- (a) the minimum tensile reinforcement complies with Clause 9.1.1;
- (b) the centre-to-centre spacing of bars in each direction shall not exceed the lesser of $2.0D_s$ or 300 mm (ignoring bars with a diameter less than half the diameter of the largest bar in the cross-section); and
- (c) the requirements of Clause 8.6.1 for beams.

Note: When applying the requirements of Clause 8.6.1 for beams, a state of flexure shall be assumed in the slab. Design of slabs for crack control for shrinkage and temperature effects, which can induce a state of tension in a slab, shall be in accordance with Clause 9.4.3.

Clause 9.4.3 Crack control for shrinkage and temperature effects (Clause 9.4, Crack Control of Slabs)

For brevity, see AS 3600-2000.

Note: The values of minimum reinforcement given in AS 3600-2000, now independent of f_{sy} , were calculated using the formulae in AS 3600-1994 directly and assuming $f_{sy}=400$ MPa.

Clause 9.4.4 Crack control in the vicinity of restraints (Clause 9.4, Crack Control of Slabs)

In the vicinity of restraints, special attention shall be paid to the internal forces and cracks which may be induced by prestressing, shrinkage or temperature.

Clause 9.4.5 Crack control at openings and discontinuities (Clause 9.4, Crack Control of Slabs)

For crack control at openings and discontinuities in a slab, additional properly-anchored reinforcement shall be provided if necessary.

5.4 Additional Design Rules

The following design rules, which are additional to those in AS 3600-1994 or those proposed to date for AS 3600-2000, are required to design and detail slabs for crack control.

Exposure Classifications Requiring Crack Control

The requirements of AS 3600-1994 for crack control of slabs for flexure apply to all exposure classifications. For AS 3600-2000, it is proposed that the requirement given under item (c) of Clause 9.4.1 shall only apply to Exposure Classification A1 when crack control is required for aesthetic reasons.

Minimum Spacing of Reinforcing Bars

A minimum clear distance between parallel bars is not specified in AS 3600, but is needed to allow the concrete to flow into place. It is recommended that the minimum clear distance is restricted to the larger of 1.5 times the maximum nominal size of aggregate (normally max. aggregate size is 20 mm) and the diameter of the largest reinforcing bar [1].

Maximum Bar Diameter

A maximum bar diameter of 20 mm is recommended for the main tension reinforcement in slabs when crack control is important. Nevertheless, bars of larger diameter can be used in accordance with AS 3600-2000, but their cross-sectional area in tensile regions is likely to be governed by the maximum serviceability stress, f_s , rather than yield strength, f_{sy} , unless they are closely spaced.

Monolithic Construction

The possibility of partial fixity at simple supports should be considered (see Section 3.7.3).

Methods of Analysis

Some of the methods of analysis in Section 7 of AS 3600-1994 are based on strength considerations only, and should be used with care if designing for crack control (see Section 7.3). Some of these methods are: the Simplified Method for Reinforced Two-Way Slabs Supported on Four Sides (Clause 7.3) – see Section 7.3; and Plastic Methods of Analysis for Slabs (Clause 7.9).

6. COMPUTER SOFTWARE

6.1 General

A computer program described in the next Sub-section has been written to assist with the design of reinforced-concrete slabs for crack control while under a state of flexure. The designs are in accordance with the rules in Section 5.3, which have been proposed for inclusion in AS 3600-2000.

The simple nature of the crack control design rules in AS 3600-1994 has meant that crack control has not formed a major part of the overall design process of a reinforced-concrete structure or member. Satisfactory crack control of slabs in flexure has been assumed to be assured by simply spacing main bars at 500 mm centres or less, depending upon the overall thickness of the slab (Clause 9.4.1 of AS 3600-1994). Many other design factors that can significantly increase the likelihood of cracking have been ignored, e.g. the degree of moment redistribution assumed at the strength limit state (see Sections 4.1 and 5.4).

Following on from Section 4.1, a major objective of using the software is, whenever possible, to find satisfactory solutions for crack control which provide the full benefit of the increase in steel strength to 500 MPa.

It follows that the software can be usefully used to check designs that are otherwise complete and ready for final detailing. Some designs might have been performed assuming 400 or 450 MPa steel will be used, leaving crack control a major design issue to address before converting to 500PLUS Rebar or OneMesh500. As necessary, critical regions must be checked for crack control, and the diameter and spacing of the main reinforcing bars selected accordingly. It is assumed that the overall depth of the concrete slab, the concrete compressive strength and the details of any compression steel are known prior to running the software.

6.2 Computer Program 500PLUS-SCC™

The design approach presented in Figs 4.2 to 4.5 in Section 4.3 explains the steps a designer will normally follow when performing detailed calculations for crack control of slabs in flexure. Of course, a major difficulty with performing these calculations manually is that a number of iterations may have to be performed before finding a satisfactory solution. At the very least, it would seem prudent for a designer to program the equations in Fig. 5.3 in order to reduce the amount of manual calculation. However, the success of designing using this approach will depend somewhat on the suitability of the initial estimate of bar diameter. The designer should also check that the solution is the most efficient and, if possible, minimises steel area.

Computer program 500PLUS-SCC allows quite a different approach to be taken during design. No manual calculations are required once the input data has been determined. Solutions that allow the full benefit to be gained from using 500PLUS Rebar or OneMesh500 are referred to as “strength governs” cases. They are identified in the tables of solutions by not being shaded in white. These are the solutions for which crack control design does not govern the area of tension reinforcement. Otherwise, the solutions are identified as “serviceability governs” cases, and they may not be the most suitable. These solutions are easily identifiable, as they are shaded in white.

It is not necessary to specify the actual area of tensile reinforcement in the slab. If this is known beforehand, then it can be compared with and adjusted as necessary against the results from the crack control analysis. However, if the area of compressive reinforcement is known, then this should be input for each section since this is taken into account by the computer program.

The program is structured such that the user first defines the cross-section details. The slab is assumed to have a prismatic section and to be continuous with both sagging (positive) and hogging (negative) moment regions. The maximum design action effects for both of these regions are then defined. Sagging bending results in tension in the bottom face.

The steps to follow when using the program to design the critical sections of a reinforced-concrete slab are briefly explained as follows (see Figs 6.1 to 6.6).

1. The cross-section details must be input first (see Fig. 6.1). Only solid, rectangular sections are allowed. The overall depth, D_s , and the top and bottom covers to the longitudinal bars are required. The cover specified must take account of the presence of any reinforcement closer either the top or bottom surface. In the case of mesh, the cover is also assumed to be to the main longitudinal bar. The value of D_s cannot be less than 100 mm, with no upper limit applying. Only single layers of top and bottom reinforcement are considered. The horizontal gap (clear distance) between bars (mesh excluded) must be defined using the Options feature (see Fig. 6.2, and Section 5.4, Minimum Spacing of Reinforcing Bars). The type of concrete (normal-weight or lightweight) is also defined using this feature, since the density of the concrete, ρ_c , (assumed to equal 2000 kg/m³ for lightweight and 2400 kg/m³ for normal weight) is used in the calculation of modular ratio, n . All bars are equi-spaced across the width of the slab, and because the slab is assumed to be wide, edge distance is not considered. The compressive strength of the concrete, f'_c (20 to 50 MPa) and the type of steel 400Y bar (f_{sy} =400 MPa), 500PLUS Rebar or OneMesh500 (f_{sy} =500 MPa) must also be specified.
2. The design action effects for both the strength and serviceability limit states must have been determined separately from analysis (see Table 4.3(1)). Only sections deemed to be in a state of flexure are allowed. Tension arising from shrinkage and temperature effects must be catered for separately using Clause 9.4.3 of AS 3600 – see Section 7.3. Under direct loading conditions, it is necessary to input the bending moment, $M_{s,1}^*$, corresponding to the serviceability overload condition with full live load applied to the beam, i.e. G+Q loading. If a value for $M_{s,1}^*$ is not input, then it will automatically be assigned equal to M_s^* . This is a satisfactory way to run the software for the uncommon cases when imposed deformation or restrained deformation have given rise to flexure, and strictly $M_{s,1}^*=0$. The critical sections for the sagging and hogging moment regions are considered separately (see Figs 6.3 to 6.6).
3. The type of slab system must be nominated. This affects the calculation of minimum reinforcement. For a one-way slab system, it is necessary to further define whether or not the minimum strength requirement concerning sudden collapse and $(M_{uo})_{min}$ can be waived (see Figs 4.2 and 5.2). Since this option is assumed to only apply to hogging moment regions, i.e. the regions where the first hinges are assumed to form, it can be waived by ticking the box in the screen for Hogging (see Fig. 6.6). If a two-way slab system is chosen, the calculation of $(M_{uo})_{min}$ is not relevant (being for beams and one-way slabs only), but it is necessary to define the slab support condition, since this instead affects the minimum amount of tensile reinforcement required in each face for bending strength. Using the software, two-way slabs may be supported by columns, or by beams or walls. Two-way slab footings are not considered, but in any case have the same minimum requirement as slabs supported by beams or walls.
4. Once the data has been input, the program will automatically perform the necessary calculations and immediately display the results. These can be viewed in the table of feasible solutions in both the Sagging and Hogging screens. The cross-sectional area and depth of any compression steel for either sagging or hogging bending must be defined separately. This will change the results, which again are automatically altered once this data is input.
5. The table of solutions extends over the following range of 400Y or 500PLUS Rebar sizes produced by OneSteel Reinforcing: viz. 10 (500PLUS only), 12, 16, 20 mm. The 10 mm bar is particularly suited to economically satisfying minimum reinforcement requirements and controlling cracking in slabs. The OneMesh500 solutions include the new range of square (SL) and rectangular (RL) Class L meshes produced by OneSteel Reinforcing. Up to three solutions are given for each type of mesh (SL or RL), these being the “lightest” feasible solutions.

A feature of the table is that it contains every feasible bar solution that satisfies design for strength, and design for crack control in accordance with all of the requirements of Section 5.3. It is up to the designer to choose the design that best suits any given situation. A dash is placed where no feasible bar solution exists. For example, this can occur for the smaller diameter bars when there is insufficient space between the bars. Mesh solutions are obviously less frequent than for bars since there are only two bar pitches, viz. 100 and 200 mm. BAMTEC® reinforcing carpets (see Section 7.3) provide a way for a designer to specify the most economical bar

solutions, and achieve very rapid reinforcement placement without wastage, making it a superior system to using mesh in many situations.

As mentioned above, solutions shown in the white-shaded area of the table (see Figs 6.3 to 6.6) indicate that the crack control design rules governed the cross-sectional area of the tension steel. In such cases, the yield strength, f_{sy} , of the reinforcing bars cannot be fully utilised at the strength limit state. Although the solution is feasible, it may not be the most economical since a larger steel area is required than just for strength.

6. The values of some of the terms used in the calculations to produce each solution given in a table are shown under the heading “Section Properties”. They are mainly concerned with the crack control calculations in Section 5.3, and are obtained by “clicking” on the particular size of bar or mesh given in the table where a solution is given. No values are shown for non-feasible situations (where there is a dash for bars). Sample results are shown in Figs 6.3 and 6.4 for the solutions involving 10 and 20 mm bars, respectively. The cross-sectional area of the tensile steel, A_{st} , as well as its effective depth, d , are given to the right side of the section properties table. (By comparing the values of these parameters in Figs 6.3 and 6.4, it is possible to see that the 10 mm bars required much less cross-sectional area than the 20 mm bars. It can be seen that this was also helped by a slightly larger effective depth for the smaller bar.) The tensile stress in the steel reinforcing bars, f_{scr} , calculated under $G+\psi_s Q$, is output. If the value of f_{scr} equals the maximum allowable steel stress, $f_{s,max}$ (given below f_{scr} , and which equals the larger of the steel stresses determined from Tables 8.6.1(A) and 8.6.1(B) in Section 5.3), then it is obvious that serviceability governed the design. Other conditions when “serviceability” is considered to govern are if $f_{scr,1}$ equals $0.8f_{sy}$ (direct loading only), or if minimum reinforcement controls. The value of $f_{scr,1}$ is given directly below f_{scr} , noting that $f_{scr,1}$ will equal f_{scr} if $\psi_s=1.0$ but otherwise may exceed $f_{s,max}$.
7. A complete set of results can be printed directly by choosing “File” then “Print”.
8. The program can be terminated by clicking on the “Close” button at any time, although the data file needs to be saved beforehand by choosing “File” then “Save As”.

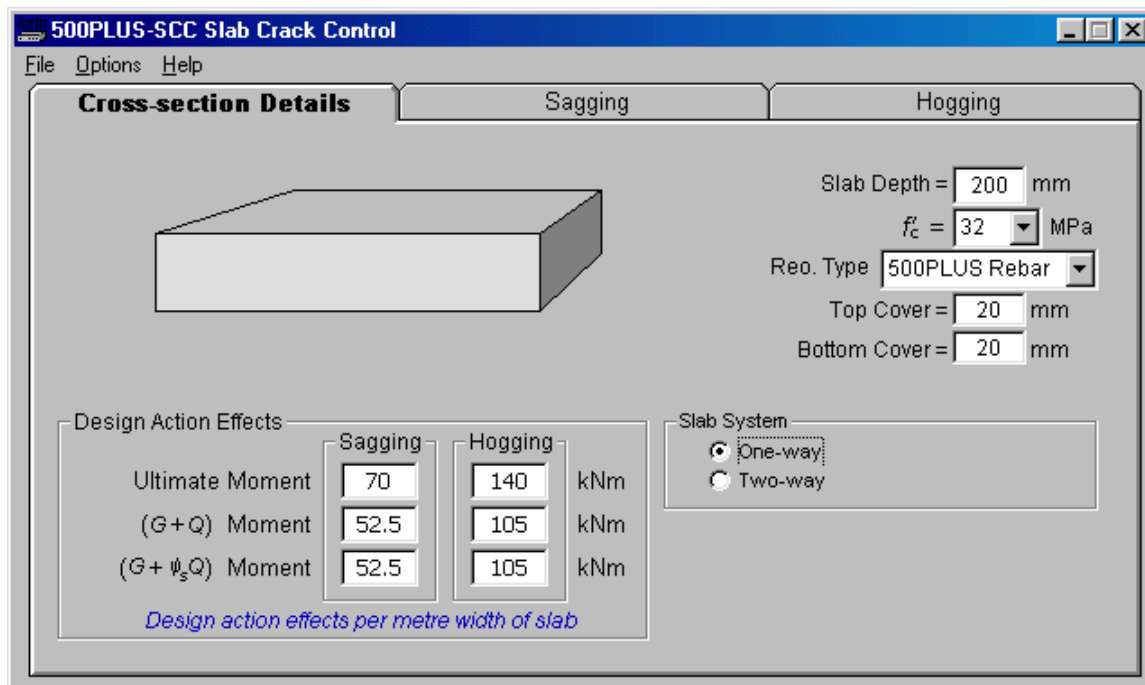


Figure 6.1 Program 500PLUS-SCC™ – Cross-section Details

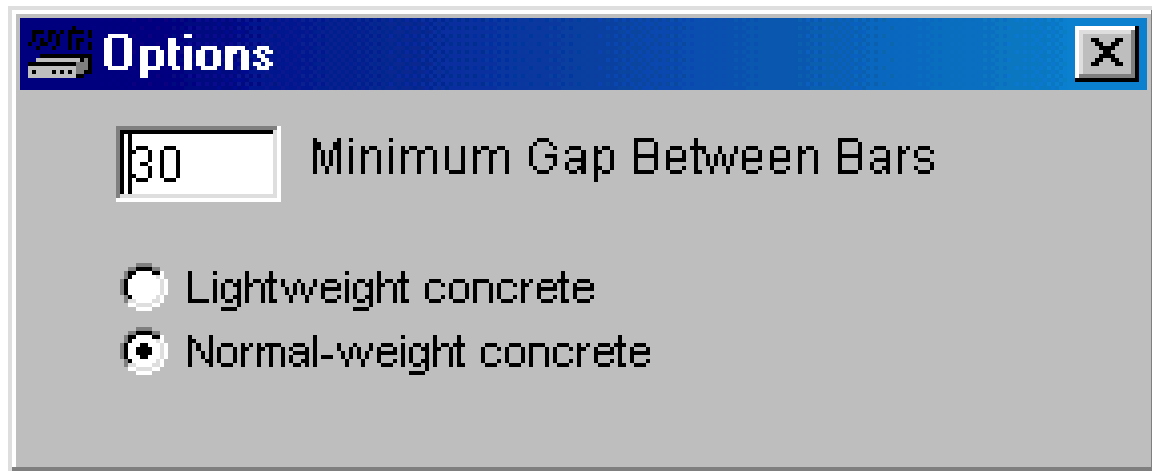


Figure 6.2 Program 500PLUS-SCC™ – Cross-section Detail Options

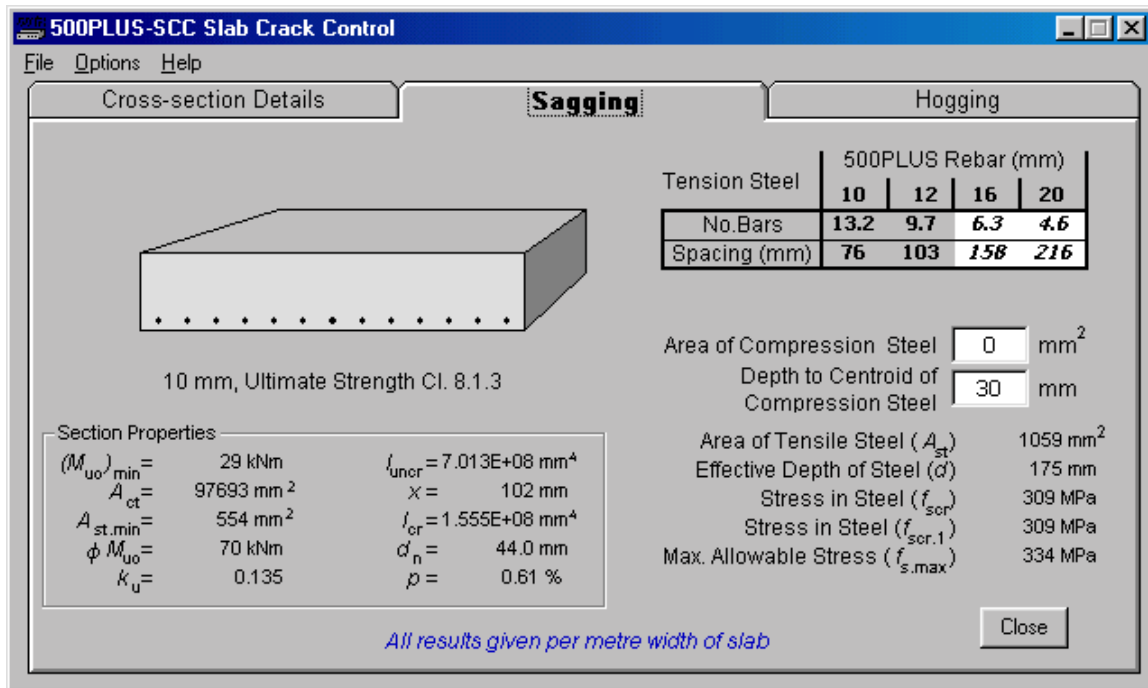


Figure 6.3 Program 500PLUS-SCC™ – Sagging Bending Calculated Properties ($d_b=10$ mm)

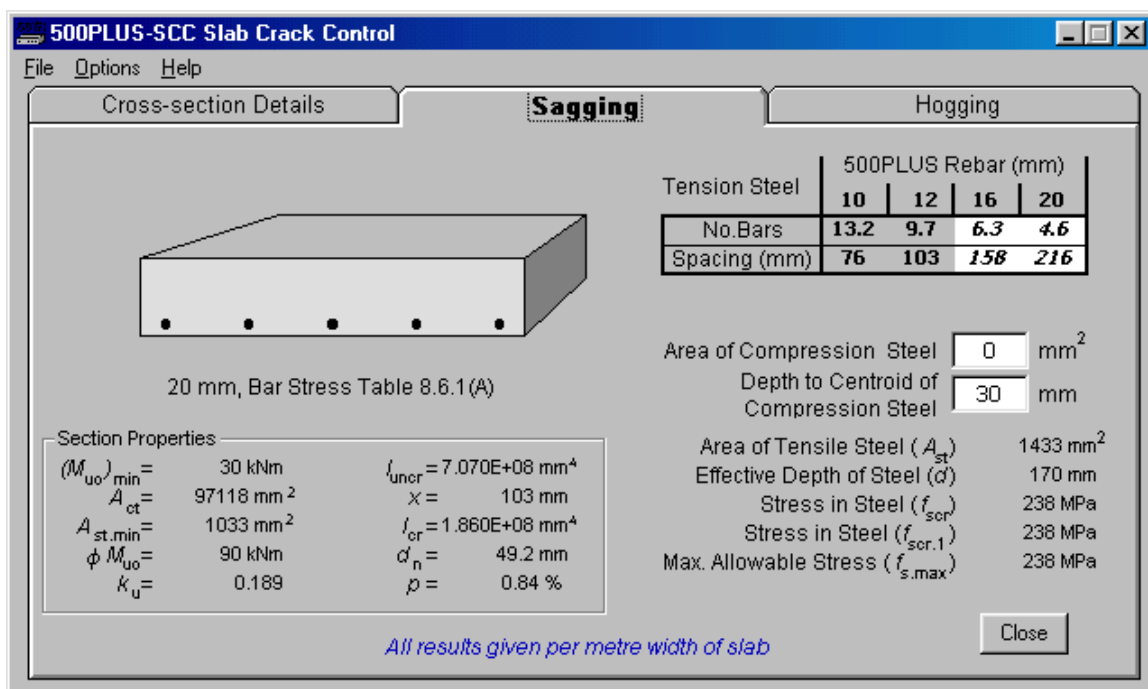


Figure 6.4 Program 500PLUS-SCC™ – Sagging Bending Calculated Properties ($d_b=20$ mm)

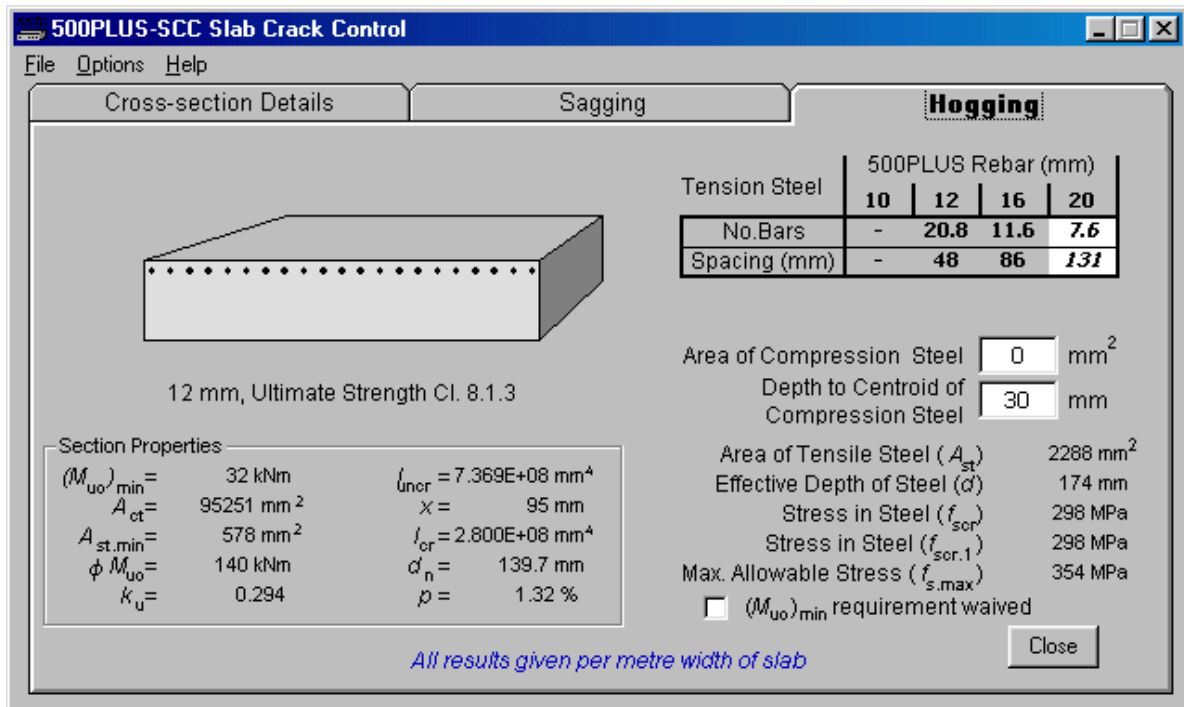


Figure 6.5 Program 500PLUS-SCC™ – Hogging Bending Calculated Properties ($d_b=12$ mm)

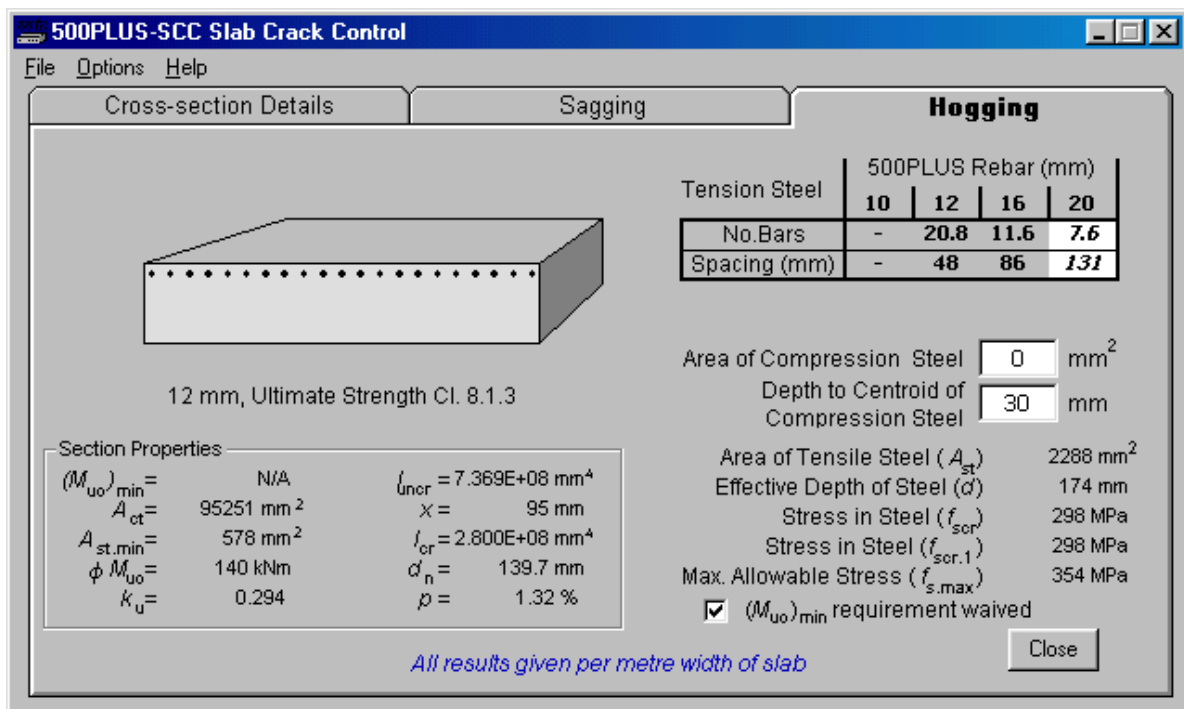


Figure 6.6 Program 500PLUS-SCC™ – $(M_{uo})_{min}$ Requirement Waived for One-Way Action

7. WORKED EXAMPLES

7.1 General

Several worked examples are used to illustrate to design engineers the way they should use the new rules proposed for inclusion in AS 3600-2000 to design reinforced-concrete slabs for crack control. Some typical design situations are examined. The opportunity is also taken to show how significant benefits can be obtained from using the higher strength 500 MPa reinforcing steels, viz. 500PLUS Rebar and OneMesh500, in slabs. A major benefit to be gained is a significant reduction in the quantity of reinforcing steel required, for which the associated cost savings can be large. The first example illustrates how this can be achieved by designers (see Section 7.2). This can also result in fewer and/or smaller bars to lay, and possibly reduced congestion. However, these benefits may not lead to direct savings per se if the new, advanced construction technique of using BAMTEC reinforcing carpets is employed, which is described in the second example (see Section 7.3).

Another major benefit to be gained from using this booklet is that the likelihood of structures experiencing the damage from cracking like that shown in Fig. 1.2 should be greatly reduced. Moreover, experience is showing that the design information contained in this booklet should be used irrespective of whether or not grade 500 MPa reinforcement is being used.

Detailed calculations that show how to directly apply the design rules and equations presented in Section 5, have not been included in this booklet. Instead, the reader is referred to Section 7 of design booklet RCB-1.1(1) for this information, noting that (at least for a state of flexure) the calculations needed to design beams for crack control are in principle the same as those for slabs.

The reader can also refer to other documents for design examples that illustrate the use of the design provisions in Eurocode 2 for crack control of slabs [11,12,22].

7.2 Example 1 – Reinforced Slab Cross-sections in Flexure Designed to AS 3600-2000 Using 500PLUS® Rebar

This example is used to illustrate the way a designer should approach reinforcing critical sections of a slab in flexure, while satisfying the new crack control provisions of AS 3600-2000 given in Section 5.3. An important objective is that using 500PLUS Rebar will lead to significant benefits compared with 400Y bar.

The details of the slab sections to be studied are given in Fig. 7.1. It can be observed that, for simplicity, they are the same in both hogging and sagging bending, whereby the direction of bending is not a consideration. For generality, it is assumed that the slab section is subjected to a wide range of bending moments. They vary, starting from a very small value such that minimum reinforcement requirements come into play, increasing up to values near the limit of what an under-reinforced section can carry, i.e. $k_u \approx 0.4$. Specifically, it has been assumed that $D_s = 200$ mm, $c = 20$ mm, $f_c = 32$ MPa, and $M_s^* = M_{s,1}^* = 0.75M^*$. So that comparisons can be made between using 400 and 500 MPa reinforcement, solutions have also been generated for 400Y bar as well as 500PLUS Rebar. Bar diameters of 10 mm (only available in 500PLUS Rebar), 12 mm, 16 mm and 20 mm have been considered.

For this example, it makes no difference whether one-way or two-way action is assumed. This is because the minimum quantity of tensile reinforcement required is governed either by Eq. 5.3(3) for crack control, or, as d_b increases, by the maximum bar spacing limit of 300 mm. The minimum strength requirement defined by Eqs 5.3(1) and 5.3(2) doesn't govern for any of the bar diameters being considered.

Typical outputs for the problem in hand, obtained from running computer program 500PLUS-SCC, are shown in Figs 6.3 and 6.4 ($M^* = 70$ kNm/m) and in Fig. 6.5 ($M^* = 140$ kNm/m). The results are summarised in Figs 7.2 to 7.5 for bar diameter, d_b , equal to 10, 12, 16 and 20 mm, respectively, which were generated using 500PLUS-SCC. The following comments are made with respect to all of these figures.

- (a) All the curves lie between the minimum and maximum amounts of reinforcement, defined by the reinforcement ratio, $p=A_{st}/bd$, the values of which depend on the bar diameter, d_b . The maximum amount of reinforcement has been chosen to correspond to when $k_u=0.4$. As mentioned above, the minimum value of p is determined either by the crack control formula Eq. 5.3(3) or by the maximum allowable bar spacing of 300 mm (Clause 9.4.1 of AS 3600-2000).
- (b) The “base line” to the curves generated for 500PLUS Rebar is labelled “500PLUS Rebar; M^* ”, and was produced by calculating the design moment capacity, ϕM_{uo} , as a function of reinforcement ratio, p , without giving any consideration to crack control. This curve, therefore, corresponds to when the tensile strength ($A_{st}f_{sy}$) of the 500PLUS Rebar is fully utilised in bending. It is shown as a dashed line. However, it is not visible in Fig. 7.2 since it coincides exactly with the solid line labelled “500PLUS Rebar; ϕM_{uo} ”.
- (c) The dashed curve labelled “500PLUS Rebar; M^*_s ” shows the values of the serviceability moments assumed for the example, viz. $M^*_s = M^*_{s,1} = 0.75M^*$.
- (d) The solid line in Figs 7.2 to 7.5 labelled “500PLUS Rebar; ϕM_{uo} ” shows the design moment capacity, ϕM_{uo} , which results for a given value of M^* in order to satisfy both requirements of bending strength and crack control. This value is output by 500PLUS-SCC in the Section Properties table for sagging or hogging bending, and exceeds M^* when crack control governs the design. A clear example of this is shown in Fig. 7.5 where the solid line departs above the line labelled “500PLUS Rebar; M^* ” for all values of bending moment less than 142 kNm/m.
- (e) Where the solid line “500PLUS Rebar; ϕM_{uo} ” lies above the dashed line “500PLUS Rebar; M^* ”, the area of reinforcement is greater than that required to satisfy bending strength. Therefore, the tensile strength of the steel ($A_{st}f_{sy}$) is not fully utilised which is to be avoided whenever possible. It follows that in this situation, the value of the reinforcement ratio, p , corresponding to any particular value of M^* , will have to increase by approximately the ratio of ϕM_{uo} to M^* .
- (f) As an example of item (e), consider the case of $M^*=70$ kNm/m and 500PLUS Rebar with $d_b=20$ mm. The horizontal dashed line in Fig. 7.5 intersects the line “500PLUS Rebar; M^* ” at Point A, where $p=0.0064$. This defines the minimum amount of steel required to satisfy bending strength. However, where the vertical dashed line through Point A intersects the line “500PLUS Rebar; ϕM_{uo} ”, at Point B, $\phi M_{uo}=90$ kNm/m which is greater than $M^*=70$ kNm/m. The reinforcement ratio, p , will need to be increased approximately to $0.0064 \times 90/70 = 0.0082$ to satisfy crack control. This correction is automatically made to the solution when the computer program 500PLUS-SCC is run (which actually gives $p=0.0084$). It will be obvious when this occurs, because ϕM_{uo} will exceed M^* in the program output.
- (g) Consider now the different solutions one obtains for p , for $M^*=70$ kNm/m for $d_b=10, 12, 16$ or 20 mm from Figs 7.2 to 7.5, respectively. These are $p=0.0061, 0.0061, 0.0074$ and 0.0084 . It is clear from this that crack control does not govern the value of p provided 10 or 12 mm diameter bars are used. A 12 mm diameter bar is preferred since fewer bars are required.
- (h) In a similar fashion, the bar diameter preferred for each range of design bending moment, M^* , is shown in Figs 7.2 to 7.5 as (see “*Recommended Solution*”): $d_b=10$ mm for $M^*<65$ kNm/m; $d_b=12$ mm for $65 \leq M^* < 105$ kNm/m; $d_b=16$ mm for $105 \leq M^* < 142$ kNm/m; and $d_b=20$ mm for $M^* \geq 142$ kNm/m.
- (i) It is worth noting (see Fig. 7.2) that with the advent of the 10 mm diameter 500PLUS Rebar, it has been possible to achieve full utilisation of the tensile strength of the reinforcing steel, while still maintaining crack control, even for the most lightly reinforced situations.
- (j) Similar curves, to those explained above for 500PLUS Rebar, have also been included in Figs 7.2 to 7.5 for 400Y bar. It can be observed that 400Y bar would be fully utilised in the slab except for when $M^* < 83$ kNm/m and $d_b=20$ mm (see Fig. 7.5). This highlights the need for the new design provisions for crack control given in this booklet with the move to 500 MPa reinforcement.

- (k) Provided the bar diameters are chosen in accordance with the recommended solutions described in item (h) above, it is clear that a saving of at least 20% in the cross-sectional area of the reinforcing steel can be achieved by changing from 400Y bar to 500PLUS Rebar. This saving can be achieved for all values of M^* greater than about 39 kNm/m, which is only a small amount above $(M_{uo})_{min}=28$ kNm/m.
- (l) In an actual design, additional reinforcement may be required in the primary or secondary direction, to control cracking due to shrinkage and temperature effects. In accordance with Clause 9.4.3 of AS 3600-2000, this will depend on the exposure condition, the degree of restraint to in-plane movement, the overall slab depth, D_s , and possibly the width of the slab. The way in which the program 500PLUS-SCC can be used to design for crack control for flexure, and the additional requirements of Clause 9.4.3 are then met separately, is illustrated in the next worked example (see Section 7.3).

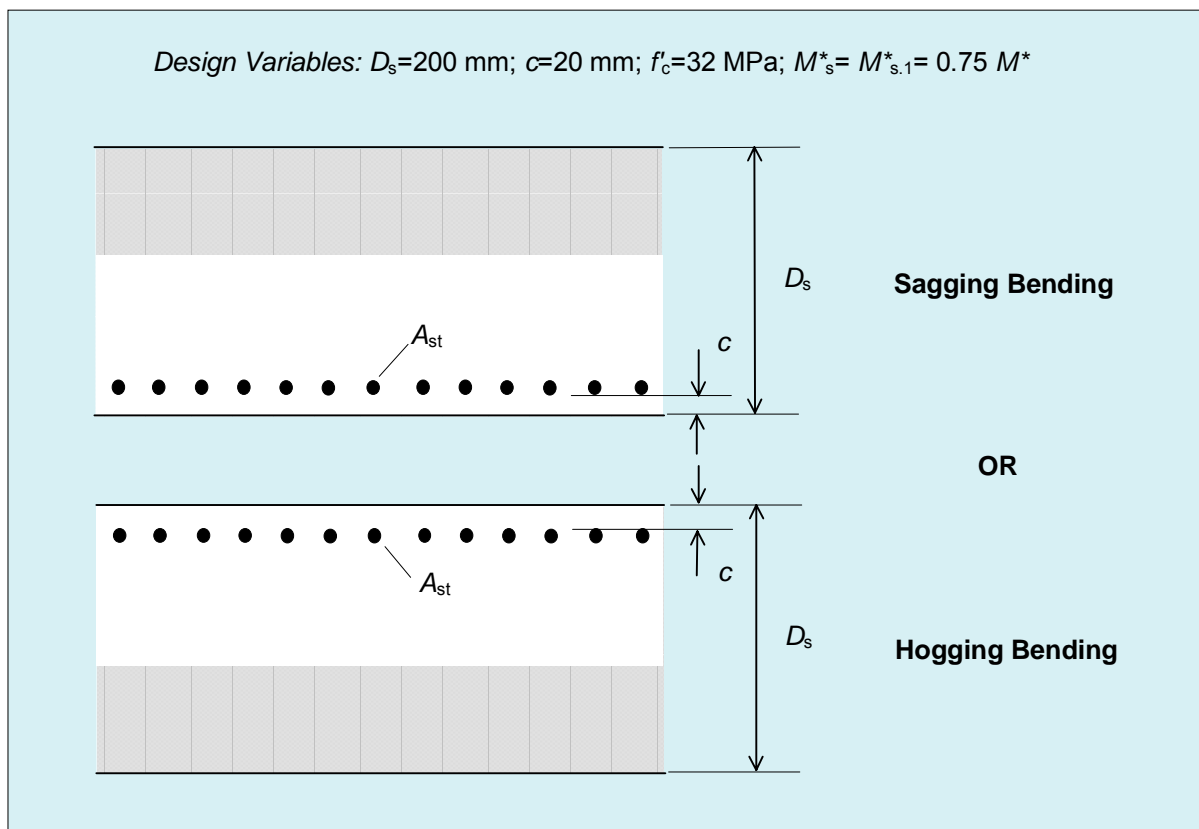


Figure 7.1 Reinforced Slab Cross-section Details

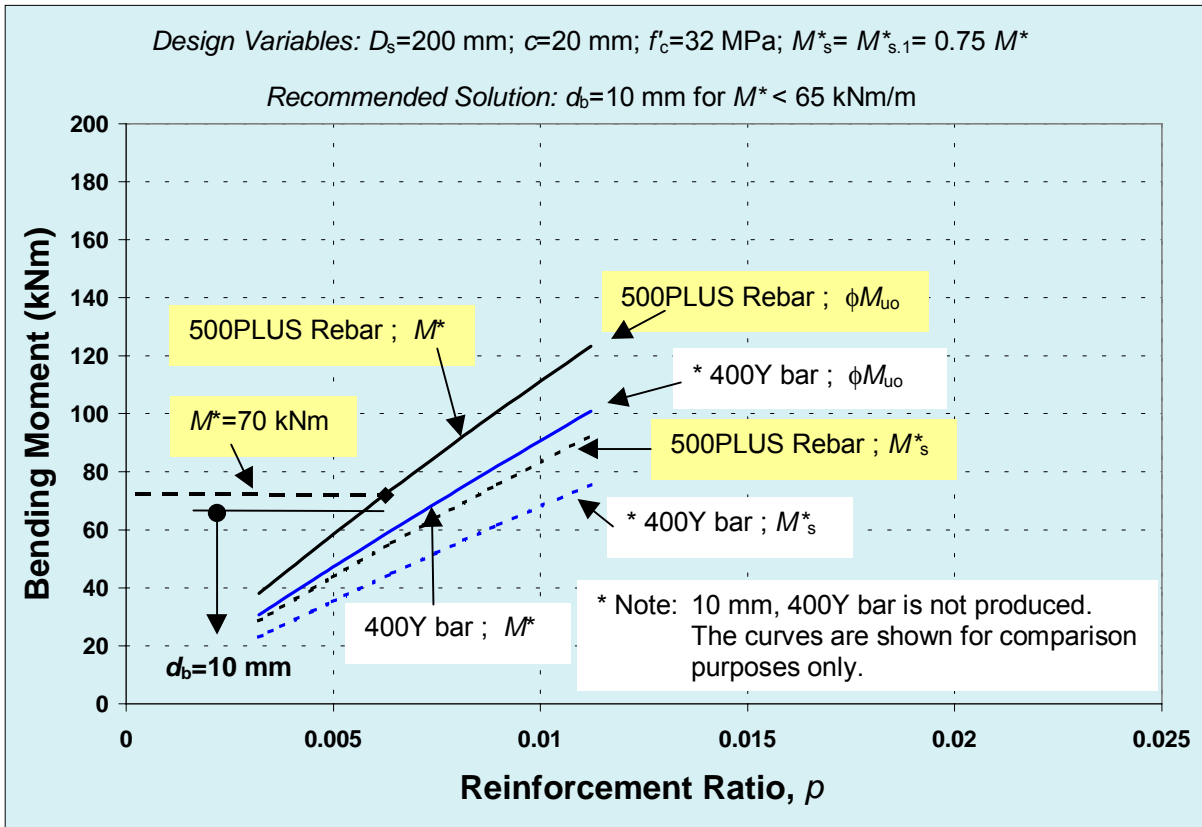


Figure 7.2 Moment vs Reinforcement Ratio – Bar diameter $d_b = 10$ mm

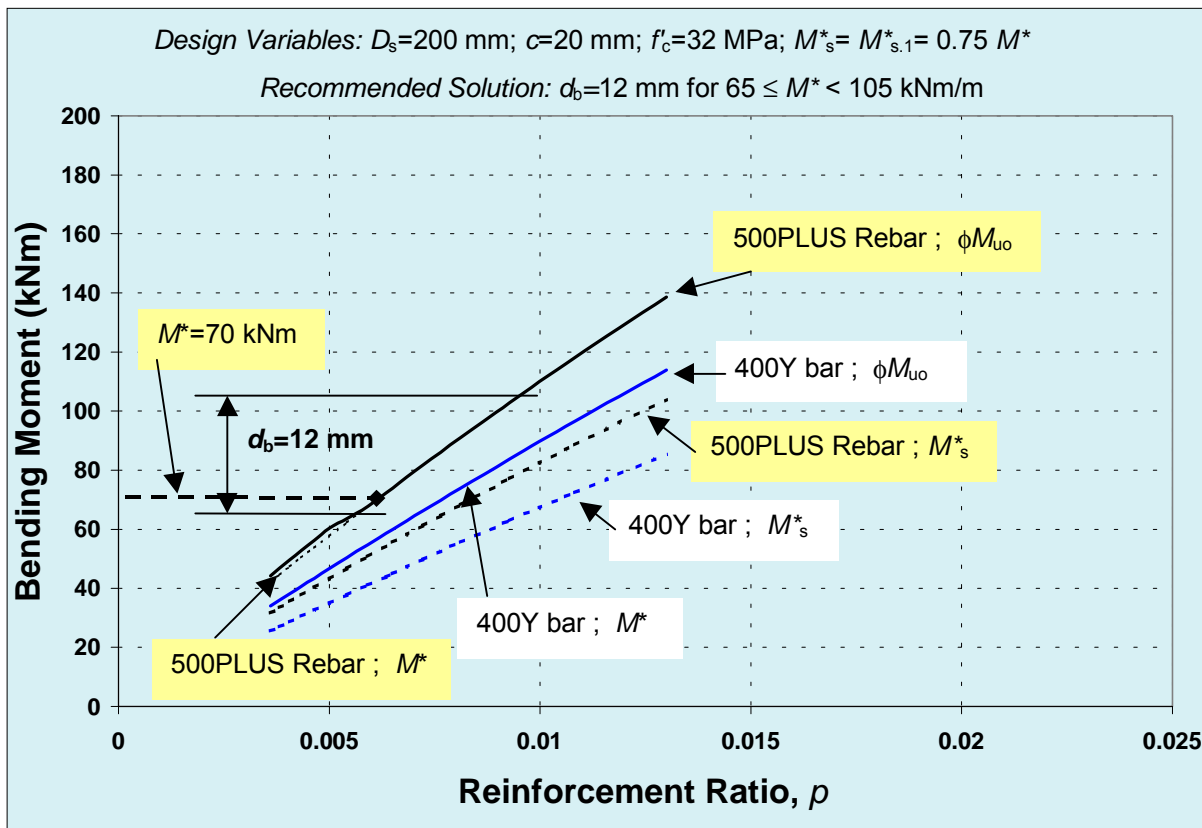


Figure 7.3 Moment vs Reinforcement Ratio – Bar diameter $d_b = 12$ mm

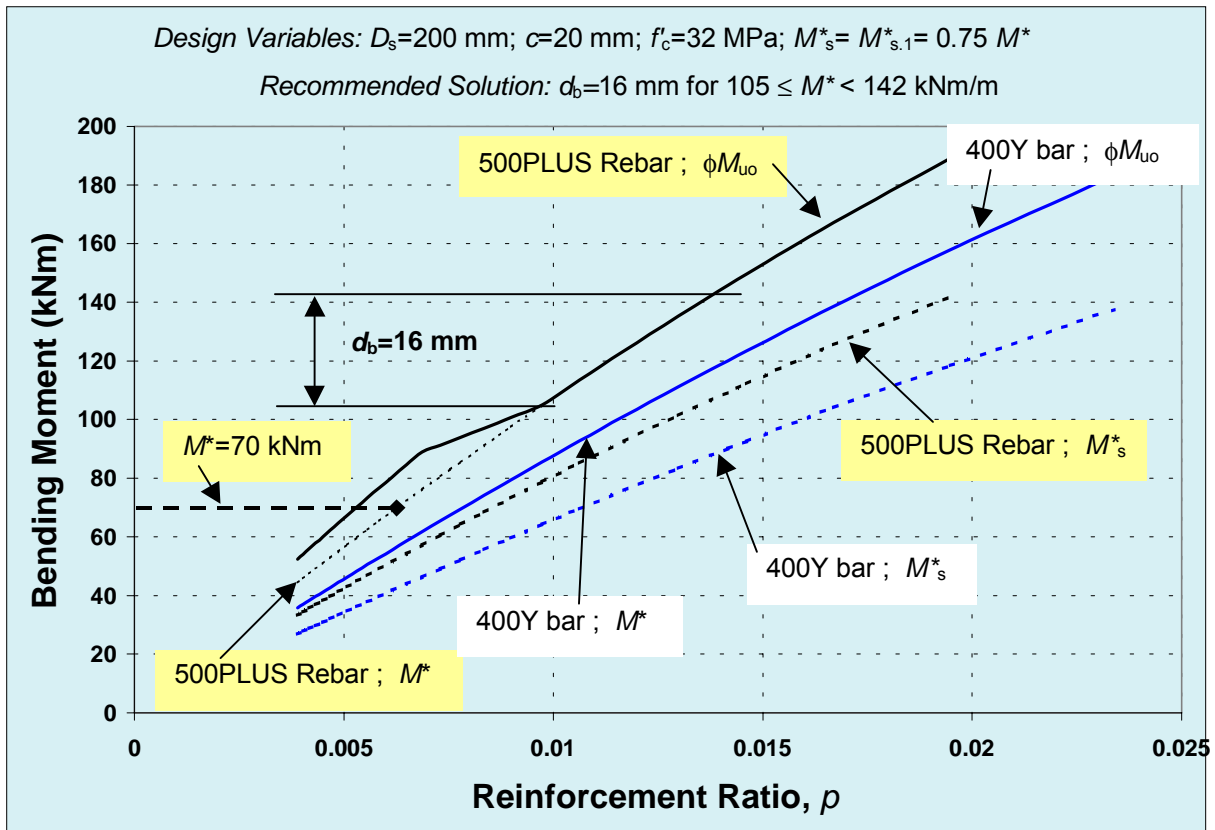


Figure 7.4 Moment vs Reinforcement Ratio – Bar diameter $d_b = 16$ mm

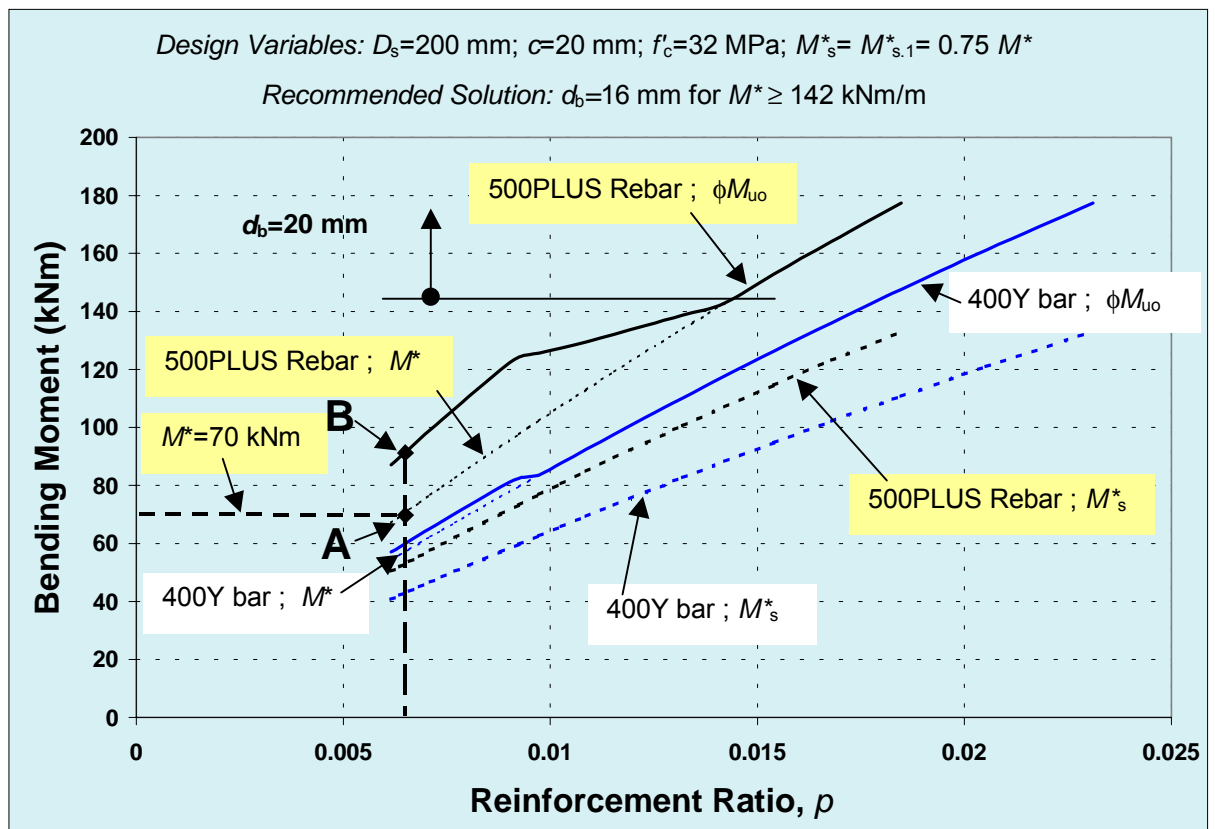


Figure 7.5 Moment vs Reinforcement Ratio – Bar diameter $d_b = 20$ mm

7.3 Example 2 – Rectangular Two-Way Slab Designed to AS 3600-2000 Using 500PLUS[®] Rebar and BAMTEC[®] Reinforcing Carpets

The rectangular slab shown in Fig. 7.6 is to be designed for strength and crack control. It has already been designed for deflection control taking into account two-way action using finite element analysis, but these calculations are beyond the scope of this booklet. However, it is necessary to calculate the design bending moments for both the serviceability and strength limits states, and finite element analysis will be used for this purpose for the reasons explained below.

The slab will be cast on 200 mm thick concrete walls that run continuously along each of its sides. L-shaped bars will be positioned in the outer face of each wall while the slab reinforcement is being placed. Further, it will be assumed that these bars will have sufficient strength to tie the slab edges down, preventing any uplift or relative rotation with respect to the walls.

The slab will be assumed to be “fully” restrained in its horizontal plane by the walls. It follows that the slab will also have to be designed for crack control due to shrinkage and temperature effects in accordance with Clauses 9.4.3.2 and 9.4.3.4 of AS 3600-2000. Moderate control, “where cracks will be seen but can be tolerated” (see Section 3.2), will be deemed sufficient for the slab.

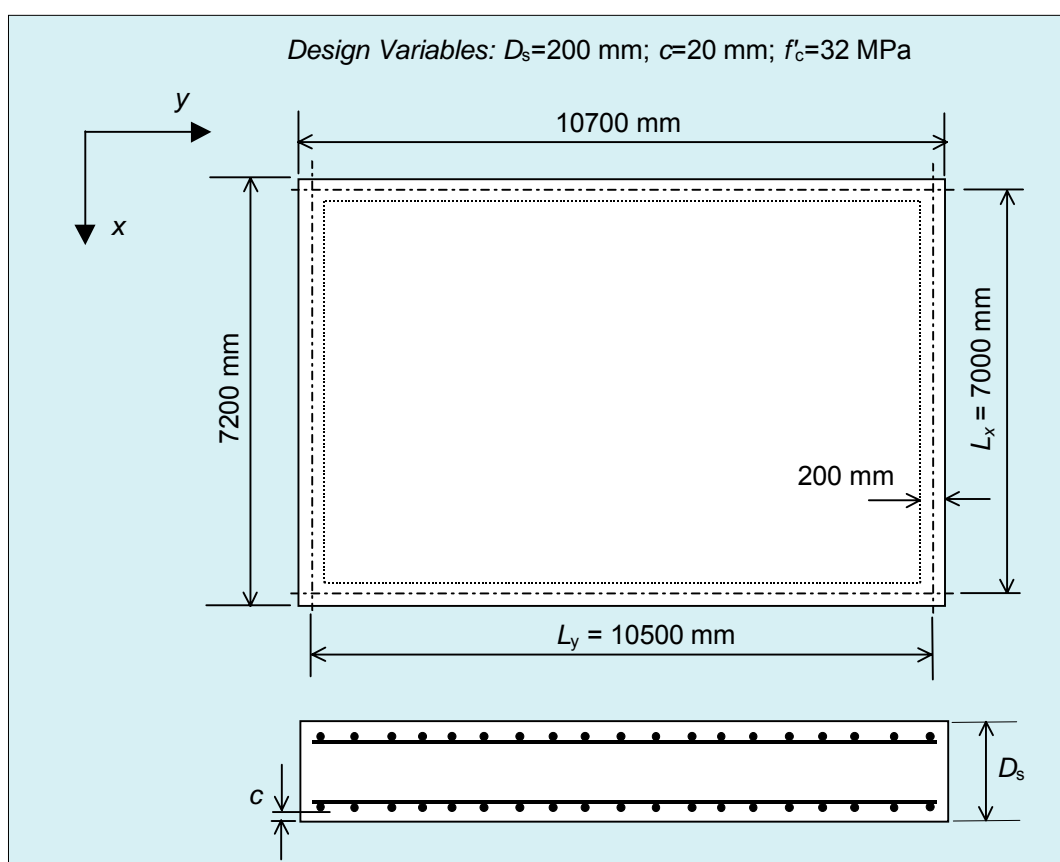


Figure 7.6 Rectangular Two-Way Slab Supported on Four Sides

Part 1 – Design criteria

For simplicity, construction loads that occur after the falsework has been removed will not be considered critical, and the slab will only be designed for the in-service condition. The design loads for strength design and serviceability design can be calculated using the following information:

- Superimposed dead load, G_{sup} = 1.5 kPa
- Live load, Q = 5.0 kPa – storage area

(Note: in accordance with AS 1170.1, $\psi_s=1.0$ and $\psi_l=0.6$ for storage areas.)

Concrete density, ρ_c = 2400 kg/m³

Allowance for reinforcing steel = 100 kg/m³

(Note: this estimate will not be readjusted in light of the final design, since ρ_c is only a nominal value. Also, for simplicity, displacement of concrete by the steel is ignored.)

Additional design parameters are as follows:

Concrete strength grade, f'_c = 32 MPa

Main steel grade = 500 MPa (500PLUS Rebar)

Main steel ductility class = N (Normal ductility)

(Note: with Class N reinforcement, moment redistribution at the strength limit state is allowed, and in accordance with Clause 7.6.8.2 of AS 3600-2000, the amount depends on the value of the neutral axis parameter, k_u .)

Exposure classification = A1

Deflection limit = L/250 long-term, total deflection

Fire rating (FRL) = 2 hours (120/120/120)

Part 2 – AS 3600-2000 design requirements

The slab could be readily designed using the simplified method for reinforced two-way slabs supported on four sides given in Section 7.3 of AS 3600-2000. It is instructive to carry out some of the calculations required by this design method, which only considers design for strength.

The values of the effective spans, L_x and L_y , are shown in Fig. 7.6 and have been calculated in accordance with Clause 1.7 of AS 3600-2000 as 7000 and 10500 mm, respectively. Therefore, $L_y/L_x=1.5$.

For the strength limit state, the uniformly-distributed design load per unit area, F_d , is calculated as:

$$\begin{aligned}G_{cs} &= 0.2 \times (2.4 + 0.1) \times 9.81 \\ &= 4.9 \text{ kPa}\end{aligned}$$

$$G_{sup} = 1.5 \text{ kPa}$$

$$\begin{aligned}G &= G_{cs} + G_{sup} \\ &= 6.4 \text{ kPa}\end{aligned}$$

$$Q = 5 \text{ kPa}$$

$$\begin{aligned}F_d &= 1.25G + 1.5Q \\ &= 1.25 \times 6.4 + 1.5 \times 5 \\ &= 15.5 \text{ kPa}\end{aligned}$$

In accordance with Clause 7.3.2 of AS 3600-2000, the positive and negative design bending moments are calculated as follows, using $\beta_x=0.040$ and $\beta_y=0.024$ from Table 7.3.2:

$$\begin{aligned}M_x^{*+} &= \beta_x F_d L_x^2 \\ &= 0.040 \times 15.5 \times 7.0^2 \\ &= 30.4 \text{ kNm/m}\end{aligned}$$

$$\begin{aligned}M_x^{*-} &= -1.33M_x^{*+} \\ &= -1.33 \times 30.4 \\ &= -40.4 \text{ kNm/m}\end{aligned}$$

$$\begin{aligned} M_y^{*+} &= \beta_y F_d L_x^2 \\ &= 0.024 \times 15.5 \times 7.0^2 \\ &= 18.2 \text{ kNm/m} \end{aligned}$$

$$\begin{aligned} M_y^{*-} &= -1.33 M_y^{*+} \\ &= -1.33 \times 18.2 \\ &= -24.2 \text{ kNm/m} \end{aligned}$$

The design bending moments M_x^* and M_y^* are to be applied over central regions of the slab equal in width to $0.75 L_y$ and $0.75 L_x$, respectively. Minimum reinforcement ($\rho=0.002$) is required in both faces of the slab in all edge regions.

It is known that the positive moment coefficients in Table 7.3.2 of AS 3600-2000 were derived from yield-line theory [1]. Therefore, moment redistribution has affected the values of the positive and negative design bending moments, and the serviceability design bending moments (M_s^* and $M_{s,1}^*$) cannot be estimated from these values without knowing the degree of moment redistribution, η . However, η is unknown, so some other means is required to calculate values for M_s^* and $M_{s,1}^*$. Elastic finite element analysis will be performed for this purpose. This is consistent with Clause 7.6 of AS 3600-2000, which allows linear elastic analysis to determine the design action effects in a structure for strength and serviceability design.

Part 3 – Calculation of design bending moments using finite element analysis

At the serviceability limit state, for crack control design:

$$\begin{aligned} F_{d,ef} &= G + \psi_s Q \\ &= 6.4 + 1.0 \times 5 \\ &= 11.4 \text{ kPa} \end{aligned}$$

Since $\psi_s=1.0$, $M_s^*=M_{s,1}^*$, while from above $F_d=15.5$ kPa, and therefore in the absence of moment redistribution, M_s^* and $M_{s,1}^*$ would both equal $F_{d,ef}/F_d=11.4/15.5=0.74$ times M^* .

The slab was modelled as an elastic isotropic plate with fixed edges, consistent with Clause 7.6.5 of AS 3600-2000. The factored design load at the strength limit state, F_d , was applied over the whole of the slab top surface to give strength design bending moments M^* . If moment redistribution is assumed not to occur, i.e. the effects of cracking are ignored, it follows that the serviceability design bending moments (M_s^* and $M_{s,1}^*$) can simply be calculated by multiplying the M^* values by 0.74. The following results were obtained from the finite element analysis.

At the strength limit state, with F_d applied, the maximum bending moments were:

$$\begin{aligned} M_x^{*+} &= 26.6 \text{ kNm/m cf. } 30.4 \text{ kNm/m by AS 3600-2000} \\ M_x^{*-} &= -58.8 \text{ kNm/m cf. } -40.4 \text{ kNm/m by AS 3600-2000} \\ M_y^{*+} &= 12.0 \text{ kNm/m cf. } 18.2 \text{ kNm/m by AS 3600-2000} \\ M_y^{*-} &= -42.0 \text{ kNm/m cf. } -24.2 \text{ kNm/m by AS 3600-2000} \end{aligned}$$

(Note: These finite element results have been checked with solutions derived by numerically solving the differential equations for the bending of isotropic plates with small deflections [23]. There is very close agreement, within 3%, using the coefficients in Table 1.12 of [23].)

At the serviceability limit state, for crack control design with $F_{d,ef}$ ($=0.74 F_d$) applied:

$$\begin{aligned} M_{xs}^{*+} &= M_{xs,1}^{*+} = 0.74 \times 26.6 = 19.7 \text{ kNm/m} \\ M_{xs}^{*-} &= M_{xs,1}^{*-} = 0.74 \times -58.8 = -43.5 \text{ kNm/m} \\ M_{ys}^{*+} &= M_{ys,1}^{*+} = 0.74 \times 12.0 = 8.9 \text{ kNm/m} \\ M_{ys}^{*-} &= M_{ys,1}^{*-} = 0.74 \times -42.0 = -31.1 \text{ kNm/m} \end{aligned}$$

It is apparent from these results that designing for strength in accordance with the simplified method in Section 7.3 of AS 3600-2000 would give rise to large amounts of moment redistribution. For example, the serviceability bending moment $M_{xs}^{*-} = -43.5$ kNm/m is greater than the strength bending moment $M_x^{*-} = -40.4$ kNm/m calculated using AS 3600-2000. Clearly, yielding of the reinforcement under service loads is possible if the simplified method in Clause 7.3 of AS 3600-2000 is followed. However, closer examination of the finite element results shows that at the edges of the central regions, the design negative and positive bending moments in each direction are only about half of the maximum values given above. Thus yielding would only occur over central portions of the sides at the supports. The results of the finite element analysis could be used to calculate the width of these portions, which would lead to savings in negative reinforcement. However, for simplicity, this has not been done in this example. It does, however, illustrate that Clause 7.3 of AS 3600-2000 should be used with care if crack control is important (see Section 5.4).

Part 4 – Strength and crack control calculations using 500PLUS-SCC

As a consequence of the calculations in Part 3 above, the central regions of the slab will be designed for the following bending moments using 500PLUS-SCC:

$$\begin{aligned}
 M_x^{*+} &= 26.6 \text{ kNm/m} & \text{and} & & M_{xs}^{*+} &= M_{xs,1}^{*+} = 19.7 \text{ kNm/m} \\
 M_x^{*-} &= -58.8 \text{ kNm/m} & \text{and} & & M_{xs}^{*-} &= M_{xs,1}^{*-} = -43.5 \text{ kNm/m} \\
 M_y^{*+} &= 12.0 \text{ kNm/m} & \text{and} & & M_{ys}^{*+} &= M_{ys,1}^{*+} = 8.9 \text{ kNm/m} \\
 M_y^{*-} &= -42.0 \text{ kNm/m} & \text{and} & & M_{ys}^{*-} &= M_{ys,1}^{*-} = -31.1 \text{ kNm/m}
 \end{aligned}$$

In the main spanning x-direction of the slab, across the shorter span, $c=20$ mm. Once all of the details of the reinforcement in the x-direction are known, it will be possible to calculate the cover to the bars in the y-direction, and to then finalise the remaining details of this reinforcement.

The results obtained from running computer program 500PLUS-SCC in order to determine the main reinforcement in the x-direction are shown in Figs 7.7 to 7.9. The presence of any compression steel has conservatively been ignored. The results show that in this direction the solutions requiring the least amount of steel are with $d_b=10$ mm, for both sagging and hogging bending.

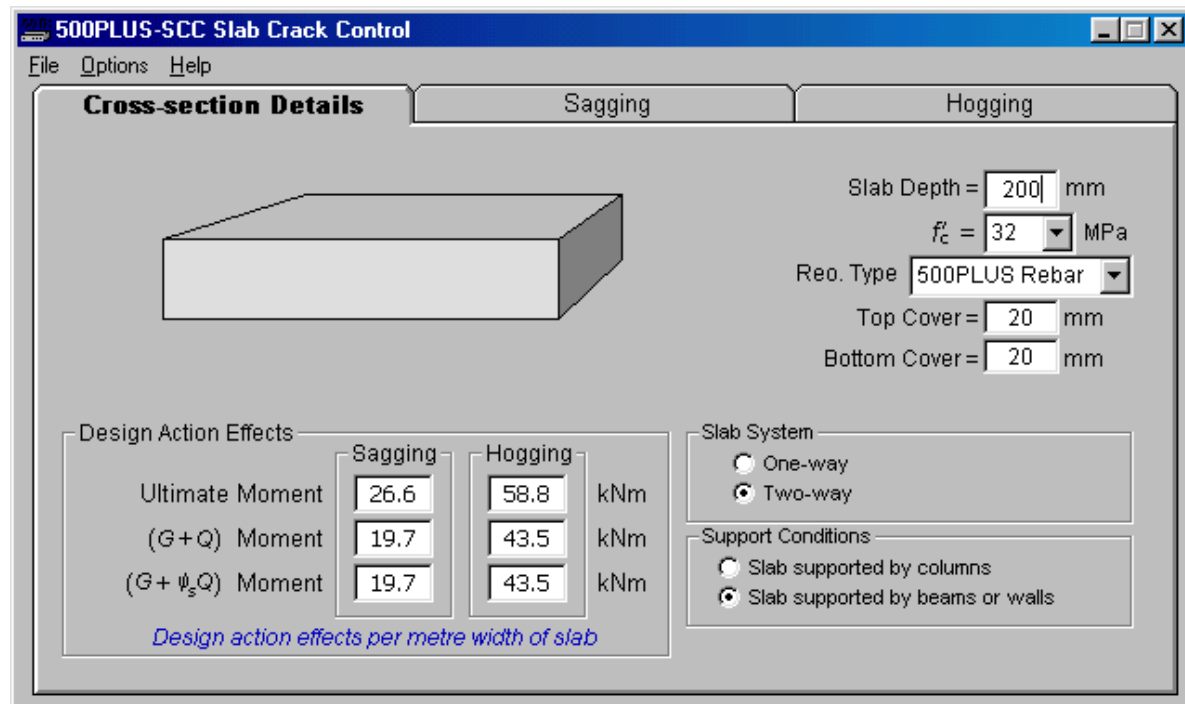


Figure 7.7 Example 2 – Cross-section Details for Bending in x-Direction

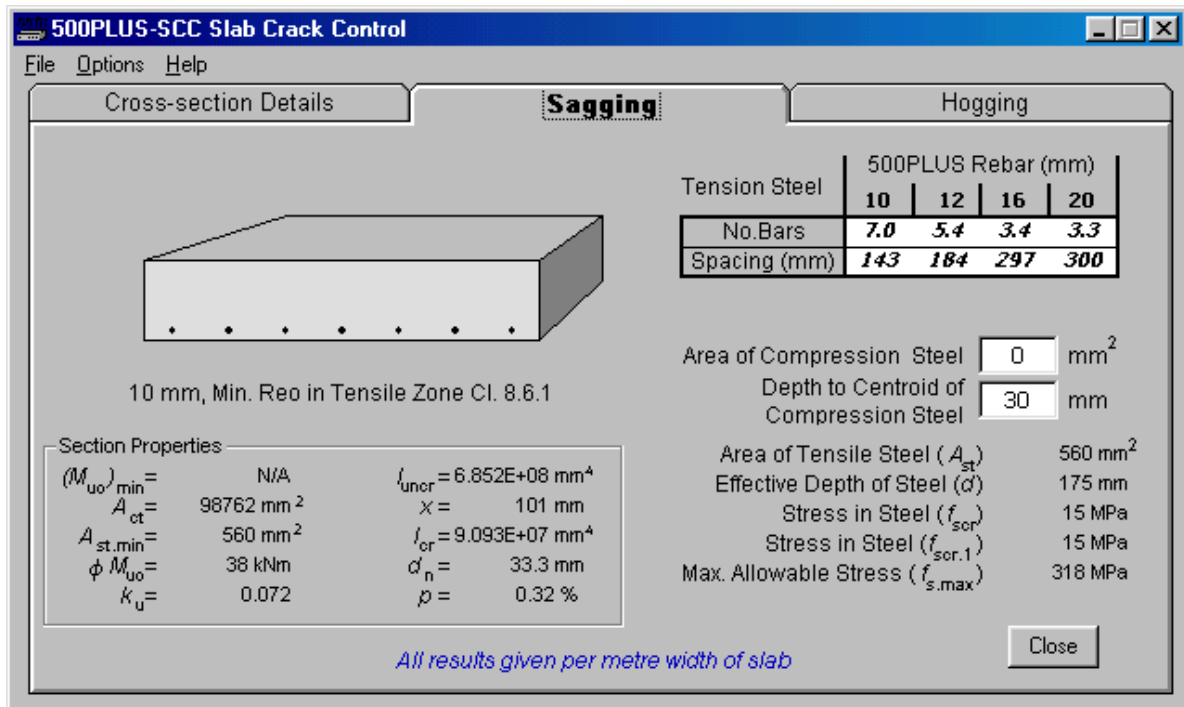


Figure 7.8 Example 2 – Sagging Bending in x-Direction (d_b=10 mm)

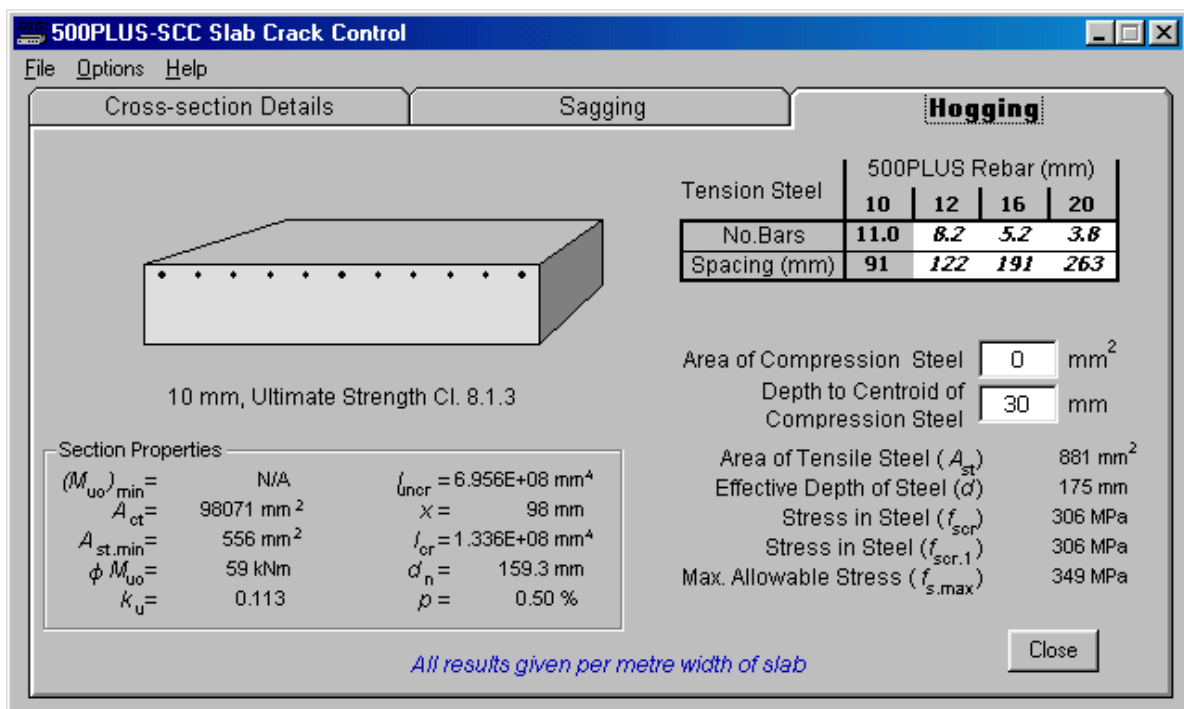


Figure 7.9 Example 2 – Hogging Bending in x-Direction (d_b=10 mm)

By using 10 mm diameter bars in the x-direction, the cover to the bars in the y-direction equals 30 mm. The results obtained from running computer program 500PLUS-SCC in order to determine the reinforcement in the y-direction are shown in Figs 7.10 to 7.12. The results show that in this direction the solutions requiring the least amount of steel are also with d_b=10 mm, for both sagging and hogging bending.

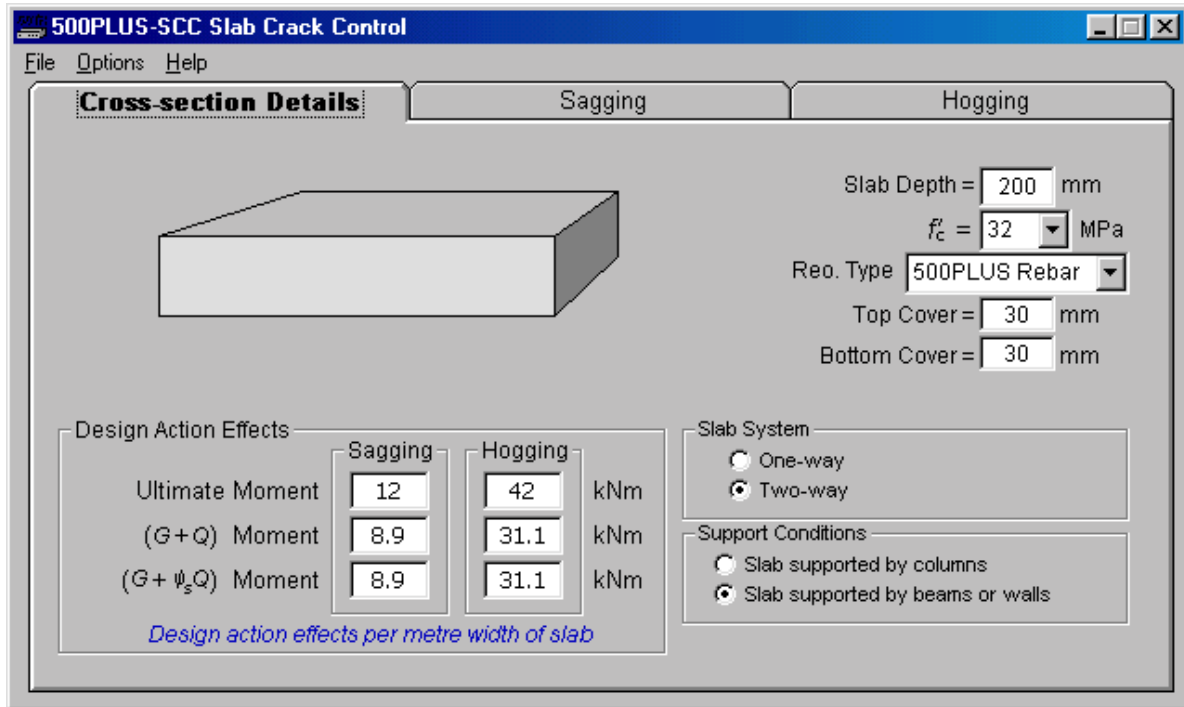


Figure 7.10 Example 2 – Cross-section Details for Bending in y-Direction

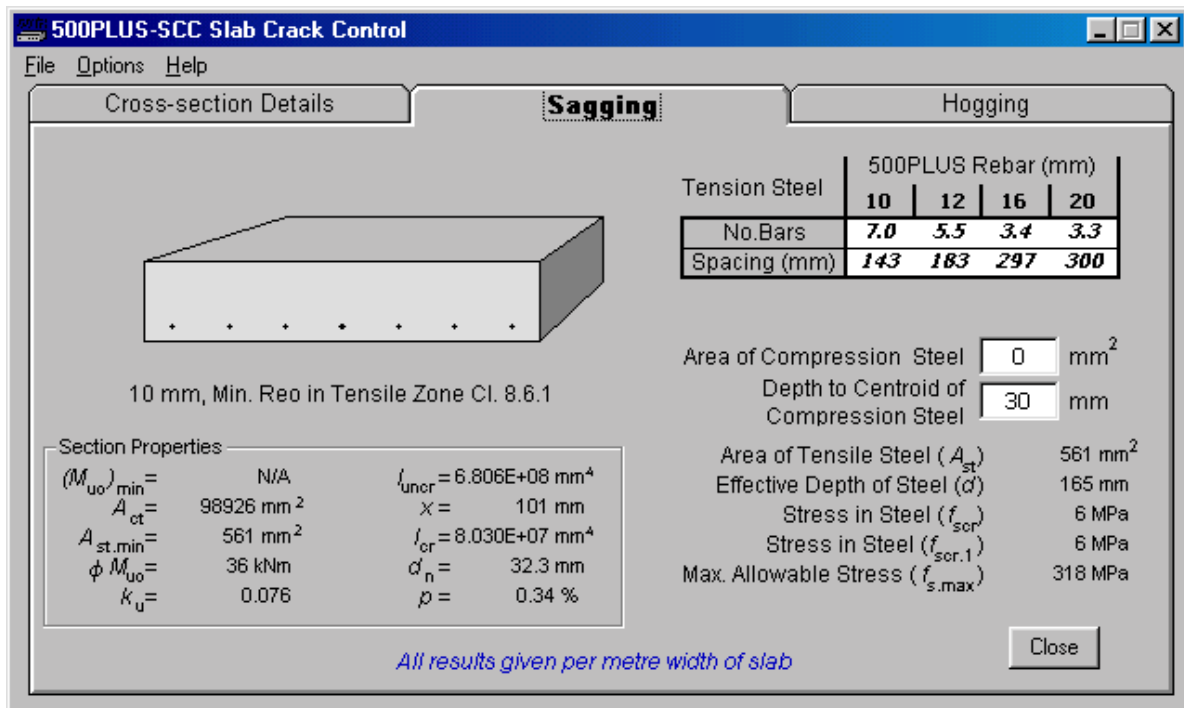


Figure 7.11 Example 2 – Sagging Bending in y-Direction ($d_b=10$ mm)

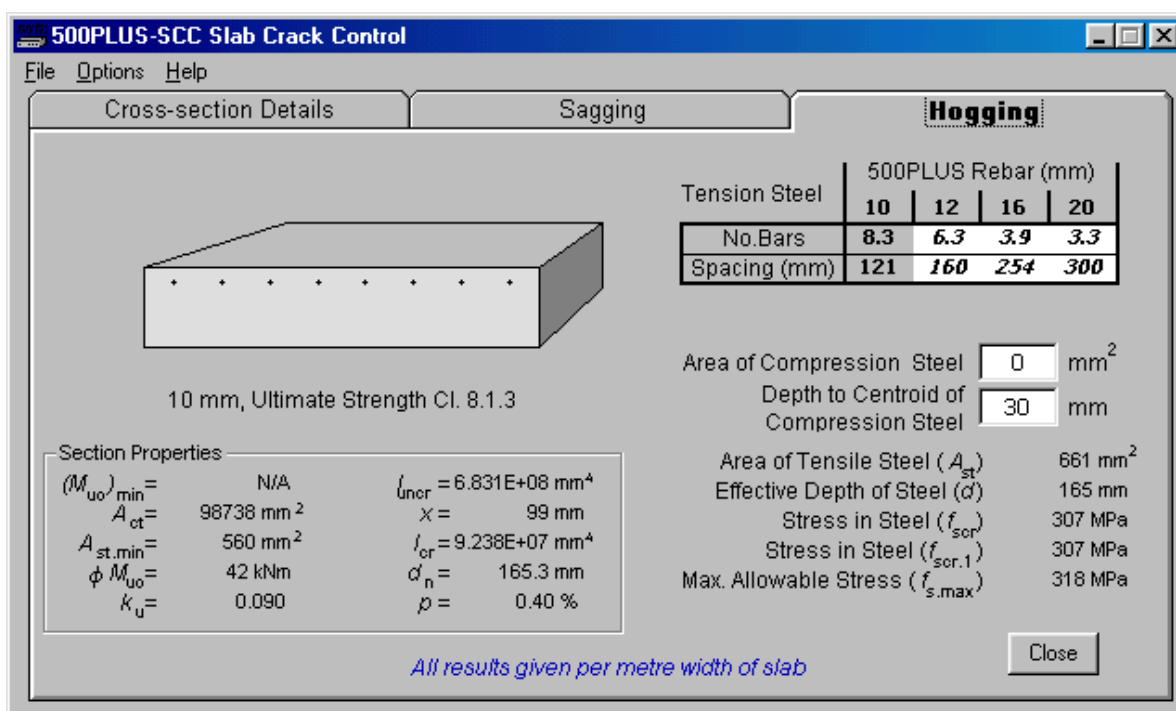


Figure 7.12 Example 2 – Hogging Bending in y-Direction ($d_b=10$ mm)

Part 5 – Detailing requirements for the reinforcement

The reinforcement in the bottom and top faces of the slab is shown detailed in Figs 7.13 and 7.14, respectively, with the following brief explanation. It should be noted that all the N10 500PLUS Rebar chosen will be placed in BAMTEC reinforcing carpets, the final details of which are given in Part 6. This system gives the designer freedom to optimise the design, without being overly concerned about bar spacings and numbers.

- (a) In accordance with Clause 9.1.1 of AS 3600-2000, minimum tension reinforcement for bending strength in the x-direction equals $0.002 \times 1000 \times 175 = 350 \text{ mm}^2/\text{m} = \text{N10}@230 \text{ mm}$.
- (b) Similarly, minimum tension reinforcement for bending strength in the y-direction equals $0.002 \times 1000 \times 165 = 330 \text{ mm}^2/\text{m} = \text{N10}@240 \text{ mm}$.
- (c) In accordance with Clause 9.4.1 of AS 3600-2000, the maximum bar spacing equals $\min.(300 \text{ mm}, 2D_s=400\text{mm})=300 \text{ mm}$, which both items (a) and (b) satisfy.
- (d) In accordance with Clause 9.4.3.2 of AS 3600-2000, for control of cracking due to shrinkage and temperature effects, the minimum area of reinforcement required in the x-direction equals the larger of that required for strength, i.e. $0.002bd$ as above in item (a), and 0.75 times that required by Clause 9.4.3.4, i.e. $0.75 \times 0.0035 \times 1000 \times 200 = 525 \text{ mm}^2/\text{m}$ for exposure classification A1, which requires $\text{N10}@300 \text{ mm}$ in each face. It follows that the requirement for minimum bending strength controls, i.e. $\text{N10}@230 \text{ mm}$.
- (e) In accordance with Clause 9.4.3.2 of AS 3600-2000, for control of cracking due to shrinkage and temperature effects, the minimum area of reinforcement required in the y-direction also equals the larger of that required for strength, i.e. $0.002bd$ as above in item (b), and 0.75 times that required by Clause 9.4.3.4, i.e. $0.75 \times 0.0035 \times 1000 \times 200 = 525 \text{ mm}^2/\text{m}$, which requires $\text{N10}@300 \text{ mm}$ in each face. It follows that the requirement for minimum bending strength controls, i.e. $\text{N10}@240 \text{ mm}$. For simplicity, this will be increased to $\text{N10}@230$ to make it the same as the minimum reinforcement in the x-direction.
- (f) The width of the central region in the x-direction equals $0.75L_y = 0.75 \times 10500 = 7875 \text{ mm}$. This must be reinforced in the bottom face with $\text{N10}@143$ (see Fig. 7.8), which can be rounded to $\text{N10}@140$. The width of this central band of reinforcement can be calculated as $\text{int.}(7875/140) \times 140 = 56 \times 140 = 7840 \text{ mm}$, with 57 bars required. To the sides of this central

- band, N10@230 are required, which equates to $\text{int}((10700-7840)/2/230)=6$ bars, but this will be reduced to 5 bars, leaving the bar out over the concrete wall so that it doesn't clash with the vertical bars. A separate bar in the side face of the slab can be added to make up for this bar.
- (g) The width of the central region in the y-direction equals $0.75L_x=0.75 \times 7000=5250$ mm. This must be reinforced in the bottom face with N10@143 (see Fig. 7.11), which can be rounded to N10@140. The width of this central band of reinforcement can be calculated as $\text{int}((5250/140) \times 140)=37 \times 140=5180$ mm, with 38 bars required. To the sides of this central band, N10@230 are required, which equates to $\text{int}((7200-5180)/2/230)=4$ bars, but this will be reduced to 3 bars, leaving the bar out over the concrete wall so that it doesn't clash with the vertical bars. A separate bar in the side face of the slab can be added to make up for this bar.
- (h) The detailing of the tensile reinforcement should comply with Clause 9.1.3 of AS 3600. Therefore, for simplicity all the bottom bars will extend past the internal face of the walls. The amount of extension will be minimal, and will equal 50 mm so as not to clash with any vertical reinforcement in the concrete walls. The clear spans in the x- and y-directions are $L_{nx}=6800$ mm and $L_{ny}=10300$ mm, whereby the overall lengths of the bars will be taken as $L_{nx}+100$ mm = 6900 mm and $L_{ny}+100$ mm = 10400 mm.
- (i) A fundamental requirement when detailing the negative reinforcement in the top face is that its curtailment should be based on the distribution of elastic bending moments. The finite element analysis shows that the contraflexure band is approximately 1400 mm out from the boundary of the slab, which is a little over $0.2L_{nx}$. Since Clause 9.1.3.1 of AS 3600 must also be satisfied, which requires a hypothetical envelope of bending moments to be considered, it is clear that the deemed-to-comply arrangement of the top steel shown in Fig. 9.1.3.2 of AS 3600 will be satisfactory. Therefore, and for simplicity, the top face reinforcement in both the x- and y-directions will be continued approximately $0.3L_{nx}=2040$ mm past the inside face of the concrete walls. This can be achieved satisfactorily by using N10 bars 2200 mm long around the perimeter of the slab in the top face.
- (j) Again, the width of the central region in the x-direction equals $0.75L_y=0.75 \times 10500=7875$ mm. The regions near the walls must be reinforced in the top face with N10@91 (see Fig. 7.9), which can be rounded to N10@90. As explained in item (d) at least N10@230 is required in the top face for bending strength, noting that only N10@300 is required for crack control due to shrinkage and temperature effects. Therefore, since negative bending will not occur over the central region, a slight concession will be made to assist with efficient bar placement. Namely, N10@270, 7100 mm long bars will extend across the entire width of the slab, with 2N10@90, 2200 mm long bars placed between each adjacent pair of long bars. The width of this central band of reinforcement can be calculated as $\text{int}((7875/90) \times 90)=87 \times 90=7830$ mm, with 88 bars required (starting and finishing with a long bar). To the sides of this central band, N10@230 are required, which equates to $\text{int}((10700-7830)/2/230)=6$ bars, but this will be reduced to 5 bars, leaving the bar out over the concrete wall so that it doesn't clash with the vertical bars. A separate corner bar can be added to make up for this bar.
- (k) Again, the width of the central region in the y-direction equals $0.75L_x=0.75 \times 7000=5250$ mm. This must be reinforced in the top face with N10@121 (see Fig. 7.12), which can be rounded to N10@115, since according to item (e) minimum transverse top-face reinforcement equals N10@230. The width of this central band of reinforcement can be calculated as $\text{int}((5250/115) \times 115)=45 \times 115=5175$ mm, with 46 bars required. To the sides of this central band, N10@230 are required, which equates to $\text{int}((7200-5180)/2/230)=4$ bars, but this will be reduced to 3 bars, leaving the bar out over the concrete wall so that it doesn't clash with the vertical bars. A separate corner bar can be added to make up for this bar.
- (l) The L-bars that lap with the N10 top face bars can be detailed as follows. Firstly, in order to limit their number, N16 bars will be used, which can be bent on site if necessary. They are placed along the entire length of the long side, and in the central region of the short side (see Fig. 7.15). They would be positioned immediately after the two bottom reinforcing carpets are rolled into place. The top carpets can then be rolled out on top of the L-shaped bars. Running 500PLUS-SCC shows that N16 bars at 170 mm and 230 mm centres along the long and short sides, respectively, will provide sufficient strength and control flexural cracking. This is also

ample steel to control cracking due to shrinkage and temperature effects and restraint.

- (m) No torsional reinforcement is required in the slab since all the corners are interior.
- (n) The vertical shear strength of the slab has been checked separately, and is satisfactory without requiring additional reinforcement.

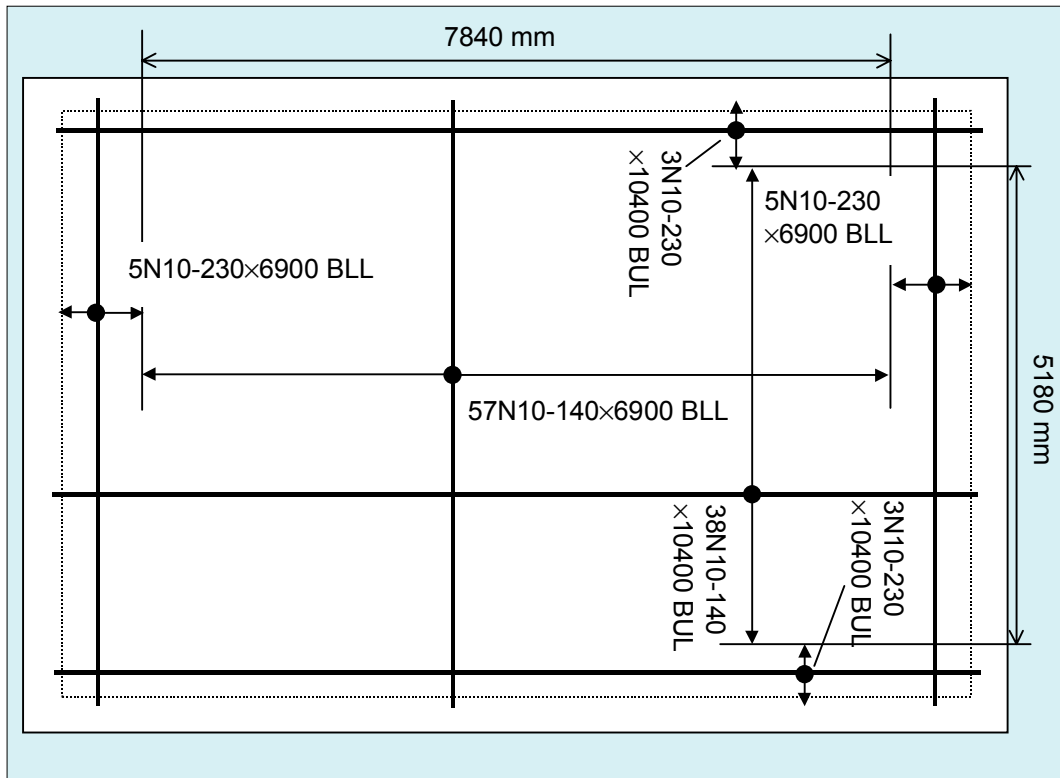


Figure 7.13 Example 2 – Bottom Reinforcement ($d_b=10$ mm)

(Note: BLL= bottom lower layer, BUL= bottom upper layer)

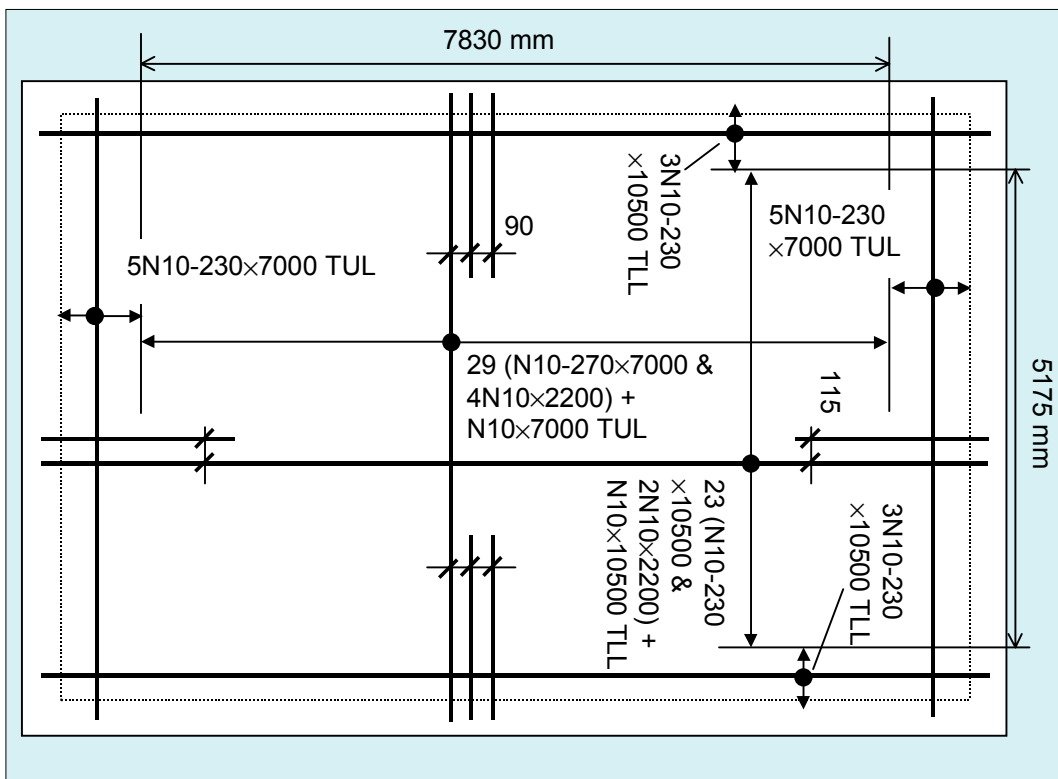


Figure 7.14 Example 2 – Top Reinforcement ($d_b=10$ mm)

(Note: TLL= top lower layer, TUL= top upper layer)

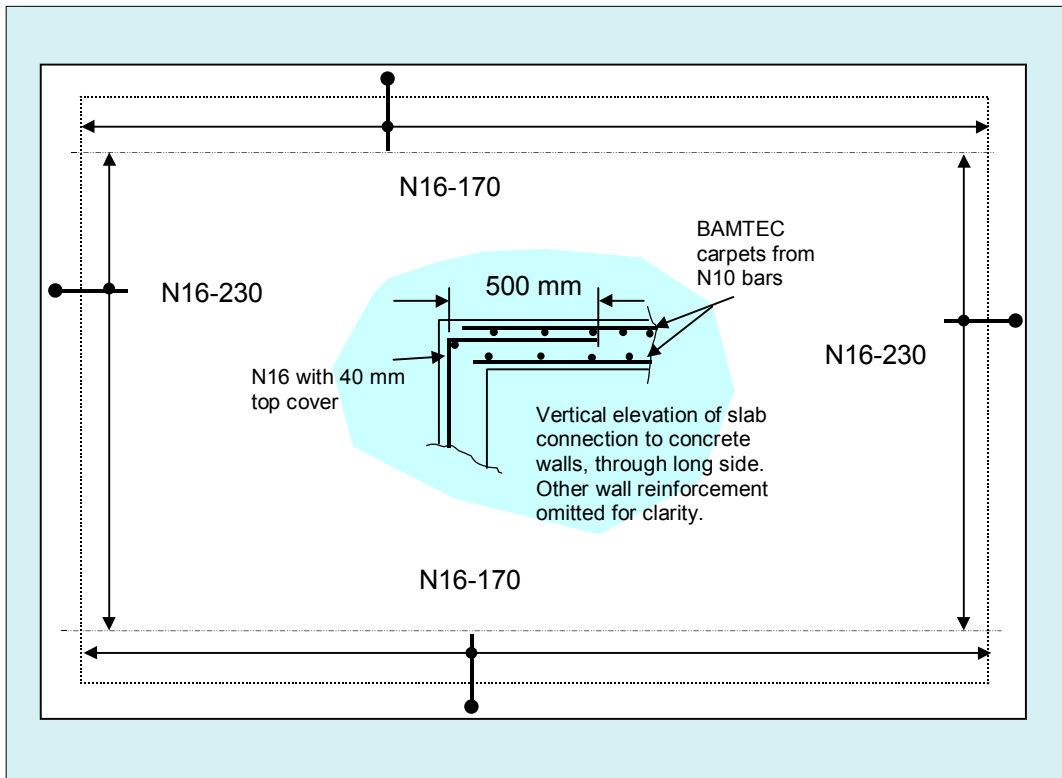


Figure 7.15 Example 2 – L-shaped Splice Bars Around Slab Perimeter ($d_b=16$ mm)

Part 6 – BAMTEC reinforcing carpets

BAMTEC reinforcing carpets are manufactured from 500PLUS Rebar, Class N reinforcement. The bars are welded in the factory to regularly spaced, thin steel straps that are used to hold the bars in place in the carpet. The carpets are first rolled up for transport to site, lifted into position by crane, and then simply and rapidly rolled out in successive layers that are normally orthogonal to each other. Strip bar chairs are used to support and separate the carpets as necessary.

To conclude this example, the reinforcement in Figs 7.13 and 7.14 has been detailed to form BAMTEC reinforcing carpets. They are laid according to the details shown in Figs 7.16 to 7.19. It was mentioned in Part 3 of this example that the results of the finite element analysis could have been applied more accurately, which would have led to savings in reinforcement. Software is being developed to support the use of the BAMTEC system, which will allow this type of saving to be readily gained. This will enhance the benefits to be had from using BAMTEC reinforcing carpets. Further information about this system can be obtained from OneSteel Reinforcing or found at www.reinforcing.com.

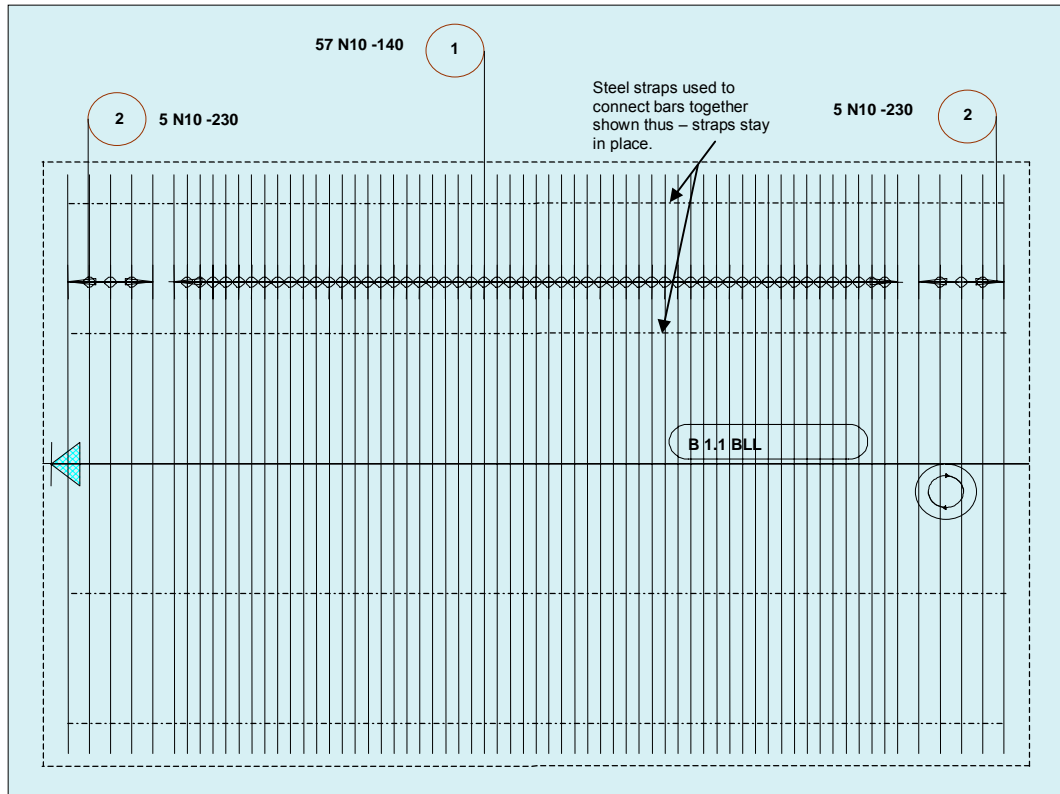


Figure 7.16 Example 2 – BAMTEC Carpet (Laid 1st – BLL)

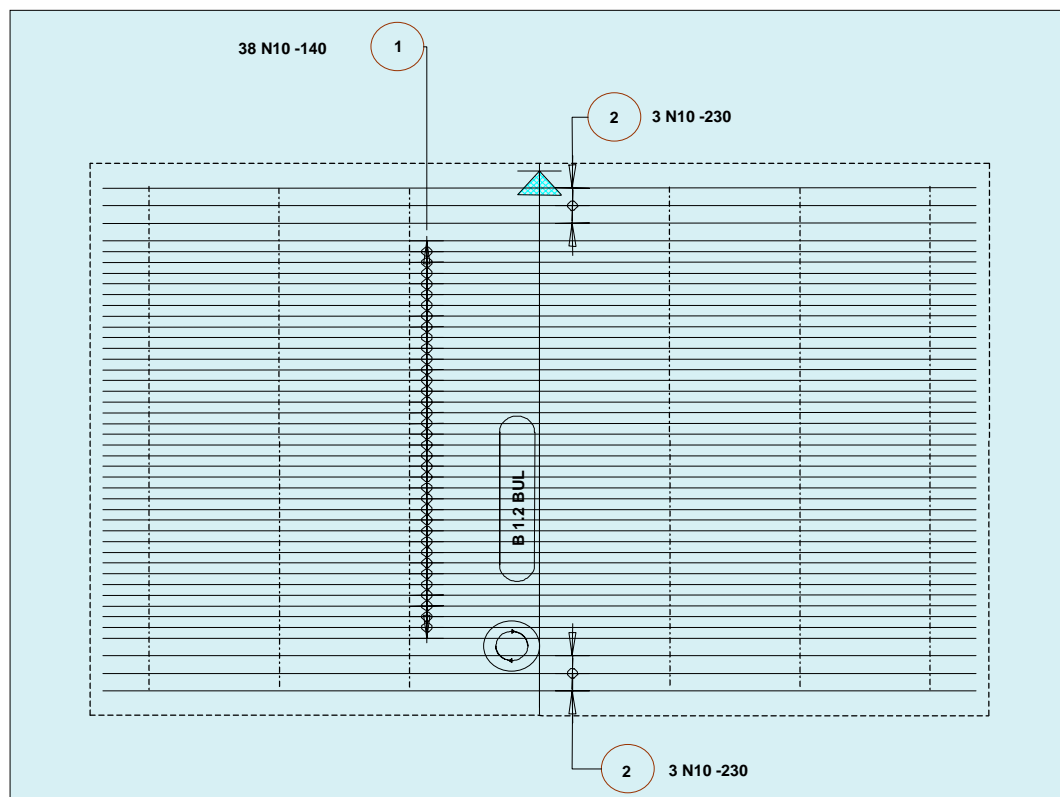


Figure 7.17 Example 2 – BAMTEC Carpet (Laid 2nd – BUL)

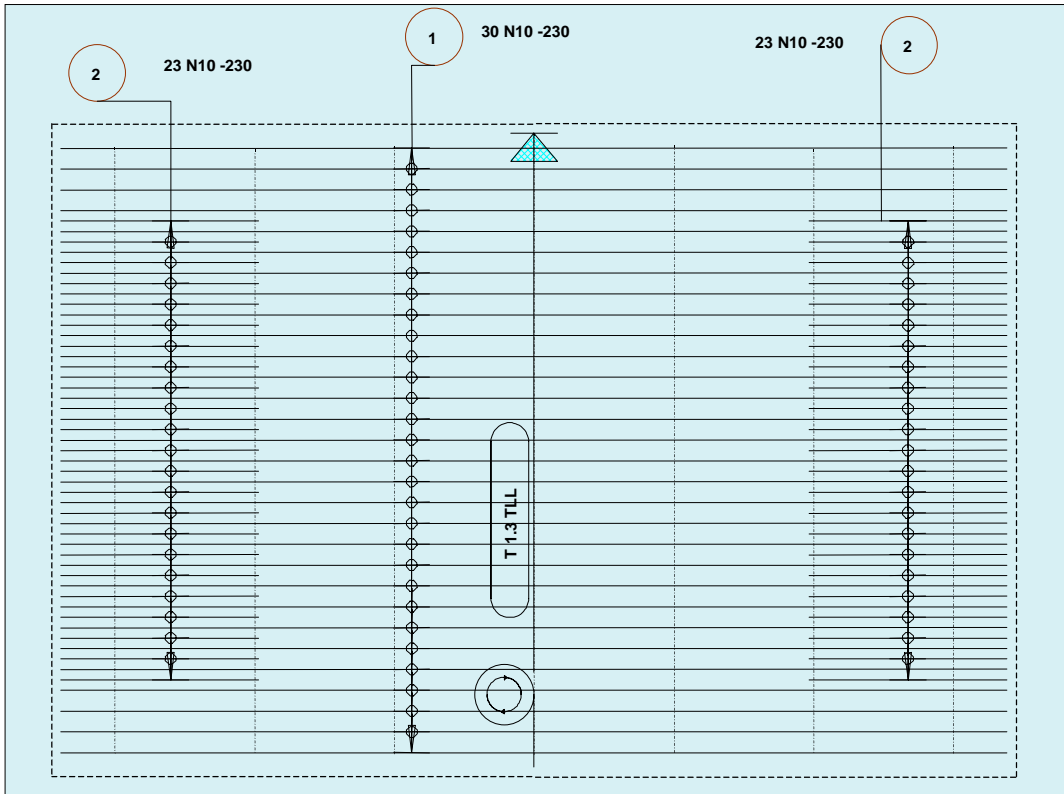


Figure 7.18 Example 2 – BAMTEC Carpet (Laid 3rd – TLL)

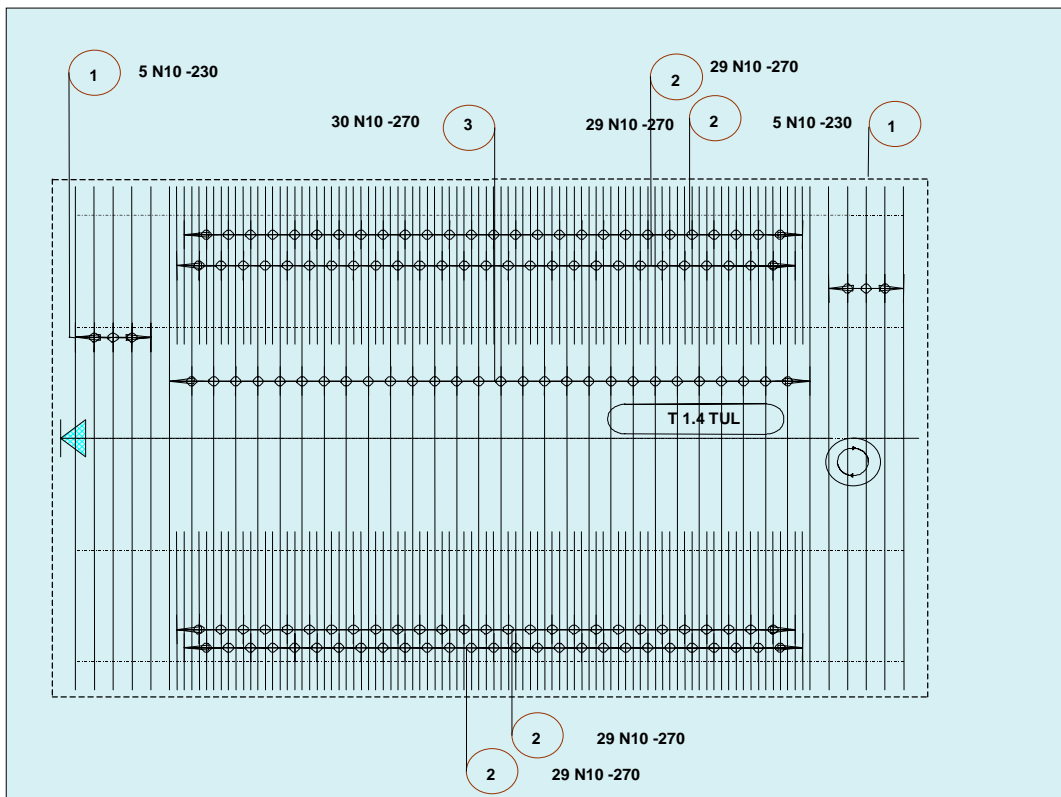


Figure 7.19 Example 2 – BAMTEC Carpet (Laid 4th – TUL)

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APPENDIX A

REFERENCED AUSTRALIAN STANDARDS

REFERENCE NO.	TITLE
AS 1170.1-1989	Minimum Design Loads on Structures (known as the SAA Loading Code), Part 1: Dead and Live Loads and Load Combinations
AS 1302-1991	Steel Reinforcing Bars for Concrete
AS 1303-1991	Steel Reinforcing Wire for Concrete
AS 3600-1994	Concrete Structures
AS 3600/Amdt 1/1996-08-05	Amendment No. 1 to AS 3600-1994 Concrete Structures, August, 1996
AS 3600 Supp1-1994	Concrete Structures – Commentary
AS 3600/Amdt 1/1996-12-05	Amendment No. 1 to AS 3600-1994 Concrete Structures – Commentary, December, 1996
DR 99193 CP	Combined Postal Ballot/Draft for Public Comment Australian Standard, Amendment 2 to AS 3600-1994 Concrete Structures, Issued 1 May, 1999
AS 3600-2000 ⁵	Concrete Structures (including Amendments Nos 1 & 2)

⁵ This Standard is yet to be published.

APPENDIX B

NOTATION

The notation used in this booklet has been taken from AS 3600-1994 when appropriate.

Latin Letters

A_b	cross-sectional area of a steel bar
A_c	cross-sectional area of concrete (steel excluded)
$A_{c,eff}$	effective cross-sectional area of concrete in tension that surrounds the tension reinforcement
A_{ct}	cross-sectional area of concrete in the tensile zone assuming the section is uncracked
A_g	gross cross-sectional area of concrete
$A_{s,bot}$	cross-sectional area of steel in bottom face
$A_{s,top}$	cross-sectional area of steel in top face
A_{sc}	cross-sectional area of compression steel
A_{st}	cross-sectional area of tension steel
$A_{st,min}$	minimum area of reinforcement permitted in tensile zone
b	slab width
b_t	mean width of tension zone (see Section 3.7.2)
c	concrete cover
d	effective depth of reinforcement at a section in bending
d_b	nominal diameter of reinforcing bar
d_{bot}	effective depth of bottom reinforcement at a section in bending
$d_{c,eff}$	depth of effective tension area of concrete (see Fig. 3.5)
d_n	depth of elastic neutral axis below compressive face at a cracked section
d_{sc}	depth of centroid of compression reinforcement below compression face
d_{top}	effective depth of top reinforcement at a section in bending
D	overall depth of beam
D_s	overall depth of slab
e	eccentricity of prestressing force (see Eq. 5.3(1))
e_T	eccentricity of design tensile force (see Section 3.6.1)
E_c	modulus of elasticity of concrete, calculated in accordance with Clause 6.1.2 of AS 3600-1994 for design
E_s	modulus of elasticity of steel reinforcement (=200 GPa)
f'_c	characteristic compressive cylinder strength of concrete at 28 days
f'_{ct}	characteristic flexural tensile strength of concrete calculated in accordance with Clause 6.1.1.2 of AS 3600-1994
f_{cm}	mean compressive strength of concrete in accordance with AS 3600-1994 (see p. 149 of AS 3600 Supp1-1994)
f_{ct}	tensile stress in concrete (see Fig. 3.2)

f_s	tensile stress in reinforcement (as a general term), or (in AS 3600-2000) maximum tensile stress permitted in the reinforcement immediately after the formation of a crack (see Eqs 3.5.3(1) and 5.3(3))
$f_{s,max}$	maximum tensile stress permitted in the reinforcement based on both Table 8.6.1(A) and Table 8.6.1(B) of AS 3600-2000 for crack control design
f_{scr}	tensile stress in reinforcement at a cracked section
$f_{scr,1}$	tensile stress in reinforcement at a cracked section, calculated with $\psi_s=1.0$
f_{sr}	stress in tension steel that just causes the tensile strength of the concrete to be reached (see Eq. 3.5.2(5))
f_{sy}	yield strength of steel reinforcement
f_t	tensile strength of concrete (mean value in Eurocode 2 – see Eq. 3.5.3(1)), assumed to equal 3.0 MPa during design for states of either tension or flexure when using Eq. 5.3(3)
F_d	uniformly-distributed design load per unit area factored for strength
$F_{d,ef}$	uniformly-distributed effective design service load per unit area
G	total dead load (including G_{cs} and G_{sup})
G_{cs}	dead load of concrete and reinforcing steel supported by beam
G_{sup}	superimposed dead load supported by beam
I_{av}	average second moment of area calculated in accordance with AS 3600-1994
I_{cr}	second moment of area of a cracked section
I_{uncr}	second moment of area of an uncracked section
k	ratio of depth of elastic neutral axis, d_n , to effective depth, d , at a cracked section (see Figs 5.7, 5.8 and 5.9)
k_u	neutral axis parameter (strength limit state)
\bar{k}	ratio of depth of elastic neutral axis, x , to overall depth, D , at an uncracked section (see Figs 5.3 and 5.4)
k_s	a coefficient that takes into account the shape of the stress distribution within the section immediately prior to cracking, as well as the effect of non-uniform self-equilibrating stresses (see Eq. 5.3(3)) ($=k_3 \times k_4$)
k_1	a factor that takes account of the bar bond properties (see Eq. 3.5.2(4)), or a coefficient used in the equation to calculate I_{av}
k_2	a factor that takes account of the stress distribution (see Eq. 3.5.2(4))
k_3	a factor that allows for the effect of non-uniform self-equilibrating stresses (see Eq. 3.5.3(1))
k_4	a factor that takes account of the stress distribution immediately prior to cracking (see Eq. 3.5.3(1))
l_{tr}	transfer length
L_{nx}	shorter clear span of a slab supported on four sides
L_x	shorter effective span of a slab supported on four sides
L_{ny}	longer clear span of a slab supported on four sides
L_y	longer effective span of a slab supported on four sides
M_{cr}	cracking moment, calculated ignoring effects of concrete shrinkage as per AS 3600-1994 (In AS 3600-2000, it is proposed that this term will be redefined to include the effects of concrete shrinkage, which leads to a reduced, more realistic value of flexural stiffness for deflection calculations – see DR 99193 CP)

M^*	design bending moment at strength limit state
M_x^{*+}	positive design bending moment at mid-span, at strength limit state, in x-direction
M_y^{*+}	positive design bending moment at mid-span, at strength limit state, in y-direction
M_x^{*-}	negative design bending moment at continuous slab edge, at strength limit state, in x-direction
M_y^{*-}	negative design bending moment at continuous slab edge, at strength limit state, in y-direction
M_{xs}^{*+}	positive design bending moment at mid-span, at serviceability limit state, in x-direction
M_{ys}^{*+}	positive design bending moment at mid-span, at serviceability limit state, in y-direction
M_{xs}^{*-}	negative design bending moment at continuous slab edge, at serviceability limit state, in x-direction
M_{ys}^{*-}	negative design bending moment at continuous slab edge, at serviceability limit state, in y-direction
$M_{xs.1}^{*+}$	positive design bending moment at mid-span, at serviceability limit state, in x-direction, calculated with $\psi_s=1.0$
$M_{ys.1}^{*+}$	positive design bending moment at mid-span, at serviceability limit state, in y-direction, calculated with $\psi_s=1.0$
$M_{xs.1}^{*-}$	negative design bending moment at continuous slab edge, at serviceability limit state, in x-direction, calculated with $\psi_s=1.0$
$M_{ys.1}^{*-}$	negative design bending moment at continuous slab edge, at serviceability limit state, in y-direction, calculated with $\psi_s=1.0$
M_s^*	design bending moment at serviceability limit state
$M_{s.1}^*$	design bending moment at serviceability limit state, calculated with $\psi_s=1.0$
M_{sy}	moment capacity at a cracked section of a reinforced-concrete slab assuming the steel has yielded, i.e. under-reinforced
M_{uo}	nominal or ultimate strength in bending
$(M_{uo})_{\min}$	minimum nominal strength in bending permitted at critical sections (see Eq. 5.3(1))
n	modular ratio ($=E_s/E_c$)
ρ	reinforcement ratio for bending of a cracked section ($=A_{st}/bd$)
ρ_{\min}	minimum reinforcement ratio (see Fig. 3.6)
$\bar{\rho}$	reinforcement ratio for bending of an uncracked section ($=A_{st}/bD$)
ρ_s	reinforcement ratio for tension ($=A_{st}/A_c$)
ρ_r	effective reinforcement ratio ($=A_{st}/A_{c,\text{eff}}$)
P	prestressing force (see Eq. 5.3(1))
Q	live load
S_b	bar spacing (see Eq. 3.6.2(1))
S_{cr}	crack spacing
S_{crm}	average final crack spacing
$S_{\text{cr},\min}$	minimum crack spacing
$S_{\text{cr},\max}$	maximum crack spacing

T^*	design tensile force at strength limit state (Note: In AS 3600, this symbol is used for the design torsional moment.)
T_b	tensile force in steel bar at a crack
T'_b	tensile force in steel bar at end of transition length, where full bond exists
T_{cr}	tensile force at first cracking (i.e. immediately prior to cracking), equal to the sum of the tensile forces in the steel reinforcement and the concrete
T_s^*	design tensile force at serviceability limit state, assumed to be located at the centroid of the uncracked section
$T_{s.1}^*$	design tensile force at serviceability limit state, assumed to be located at the centroid of the uncracked section, and calculated with $\psi_s=1.0$
w	crack width
w_k	design crack width (characteristic value)
w_{max}	maximum crack width
x	depth of elastic neutral axis below top face of beam at an uncracked section (see Figs 5.3 and 5.4)
X	parameter used to calculate elastic section properties
\bar{X}	parameter used to calculate elastic section properties
Y	parameter used to calculate elastic section properties
\bar{Y}	parameter used to calculate elastic section properties
z	lever arm of internal force couple (see Eq. 3.6.1(1) and Fig. 5.7)
Z	section modulus of uncracked section, referred to the extreme fibre at which flexural cracking occurs (see Eq. 5.3(1))
Z_c	section modulus of uncracked section, referred to the extreme compression fibre at which flexural cracking occurs (see Eq. 3.6.1(2))
Z_t	section modulus of uncracked section, referred to the extreme tension fibre at which flexural cracking occurs (see Eq. 3.6.1(2)) (=Z)

Greek Letters

β	a factor that relates the mean crack width in tests to the design value (see Eq. 3.5.2(1))
β_1	a factor that accounts for the bond properties of the reinforcement (see Eq. 3.5.2(5))
β_2	a factor that accounts for repeated stressing of the reinforcement (see Eq. 3.5.2(5))
β_x	short span bending moment coefficient for a slab supported on four sides
β_y	long span bending moment coefficient for a slab supported on four sides
χ	ratio of lever arm to internal force couple, to effective depth, d
Δ	support movement
ϵ_c	concrete strain
ϵ_{cm}	average or mean concrete strain over transition length l_r
ϵ_{cs}	free shrinkage strain of concrete
ϵ_s	steel strain

ϵ_{sm}	average or mean steel strain over transition length l_{tr} , or average difference in strain between steel and concrete (see Eqs 3.5.2(1) and 3.5.2(5))
ϕ	capacity factor (see Table 2.3 of AS 3600-1994)
η	degree of moment redistribution (see Eq. 4.1(2))
κ	parameter used to calculate second moment of area
Σ_o	bar perimeter
ρ_c	density of concrete (excluding allowance for steel reinforcement)
τ	bond stress
τ_m	mean bond stress
ξ	parameter that accounts for moment gradient effects (see Eq. 3.3.3.2(2))
ψ_l	long-term load factor (see AS 1170.1)
ψ_s	short-term load factor (see AS 1170.1)