8.6.2.2 *Crack control without direct calculation of crack widths*

The calculated steel stress (σ_{scr}) shall comply with:

(a) For beams primarily subject to tension, the calculated steel stress (σ_{scr}) on the cracked section shall not exceed the maximum steel stress given in Table 8.6.2.2(A) for the largest nominal diameter (d_b) of the bars in the section, and under direct loading the calculated tensile steel stress ($\sigma_{scr.1}$) shall not exceed $0.8f_{sy}$.

(b) For beams primarily subject to flexure, the calculated tensile steel stress (σ_{scr}) shall not exceed the larger of the maximum steel stresses given in—

(i) Table 8.6.2.2(A) for the largest nominal diameter (d_b) of the bars in the tensile zone; and

(ii) Table 8.6.2.2(B) for the largest centre-to-centre spacing of adjacent parallel bars in the tensile zone.

Under direct loading the calculated tensile steel stress ($\sigma_{scr.1}$) shall not exceed $0.8f_{sy}$. Bars with a diameter less than half the diameter of the largest bar in the section shall be ignored when determining spacing.

NOTE: Design bending moments at the serviceability limit state will normally be estimated using elastic analysis. Substantial errors may result where the actual in-service moments are likely to have redistributed significantly from the elastic distribution.

TABLE 8.6.2.2(A)

MAXIMUM STEEL STRESS FOR TENSION OR FLEXURE IN REINFORCED BEAMS

Nominal bar diameter (d_b) mm	Maximum steel stress MPa		
	w'_{max} = 0.2 mm	w'_{max} = 0.3 mm	w'_{max} = 0.4 mm
10	190	265	335
12	175	245	305
16	155	215	270
20	140	195	240
24	125	175	215
28	115	160	200
32	105	150	185
36	100	140	175
40	90	130	165

TABLE 8.6.2.2(B)

MAXIMUM STEEL STRESS FOR FLEXURE IN REINFORCED BEAMS

Centre-to-centre spacing mm	Maximum steel stress MPa		
	w'_{max} = 0.2 mm	w'_{max} = 0.3 mm	w'_{max} = 0.4 mm
50	200	300	400
100	170	270	360
150	155	245	330
200	145	225	300
250	135	210	280
300	125	200	260

8.6.2.3 *Crack control by calculation of crack widths*

The calculated maximum crack width in a reinforced concrete member at any time after cracking is given by:

$$w = s_{r,\max}\left(\varepsilon_{sm} - \varepsilon_{cm}\right) \le w'_{\max} \qquad \ldots 8.6.2.3(1)$$

where

$s_{r,\max}$ = the maximum crack spacing;

ε_{sm} = the mean strain in the reinforcement at the design loads for the serviceability limit states, including the effects of tension stiffening and any imposed deformations

ε_{cm} = the mean strain in the concrete between the cracks.

The difference between the mean strain in the reinforcement and the mean strain in the concrete is taken as:

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_{scr}}{E_s} - \frac{0.6 f_{ct}}{E_s p_{eff}}\left(1 + n_e p_{eff}\right) + \varepsilon_{cs} \ge 0.6\,\frac{\sigma_{scr}}{E_s} \qquad \ldots 8.6.2.3(2)$$

where

σ_{scr} = the stress in the tensile reinforcement assuming a cracked section

ε_{cs} = the absolute value of the final long-term shrinkage strain determined in accordance with Clause 3.1.7

n_e = the effective modular ratio $(1 + \varphi_{cc})E_s/E_c$

φ_{cc} = the creep coefficient associated with the time interval after cracking determined in accordance with Clause 3.1.8

f_{ct} = the mean value of the axial tensile strength of concrete at the time cracking is expected

p_{eff} = the reinforcement ratio given by $A_{st}/A_{c,eff}$

$A_{c,eff}$ = the effective area of concrete in tension surrounding the bars with depth $h_{c,ef}$ equal to the lesser of $2.5(D\text{-}d)$, $(D\text{-}kd)/3$ or $D/2$

D = the overall depth of the cross-section

d = the effective depth to the centroid of the tensile steel

kd = the depth of the neutral axis on the cracked section

For cross- sections with bonded reinforcement fixed at reasonably close centres, i.e. bar spacing $\le 5(c + 0.5d_b)$, the maximum final crack spacing may be calculated from:

$$s_{r,\max} = 3.4c + 0.3 k_1 k_2 d_b / p_{eff} \qquad \ldots 8.6.2.3(3)$$

where

d_b = the tensile bar diameter

c = the clear cover to the longitudinal reinforcement

k_1 = a coefficient that accounts for the bond properties of the bonded reinforcement, with $k_1 = 0.8$ for deformed and $k_1 = 1.6$ for plain bars

k_2 = a coefficient that accounts for the longitudinal strain distribution, with $k_2 = 0.5$ for bending and $k_2 = 1.0$ for pure tension. For cases in combined tension and bending, $k_2 = (\varepsilon_1 + \varepsilon_2)/(2\varepsilon_1)$ and ε_1 is the greater and ε_2 is the lesser of the tensile strains at the boundaries of the cross-section (assessed on the basis of a cracked section)

8.6.3 Crack control for flexure in prestressed beams

Flexural cracking in a prestressed beam shall be deemed to be controlled if, under the short-term service loads, the resulting maximum tensile stress in the concrete does not exceed $0.25\sqrt{f'_c}$ or, if this stress is exceeded, by providing reinforcement or bonded tendons, or both, near the tensile face with a centre-to-centre spacing not exceeding 300 mm and by either of the following:

(a) Limiting the calculated maximum flexural tensile stress in the concrete under short-term service loads to $0.6\sqrt{f'_c}$.

or

(b) Limiting the increment in steel stress near the tension face to that given in Table 8.6.3, as the load increases from its value when the extreme concrete tensile fibre is at zero stress to the short-term service load value.

or

(c) Limiting the calculated maximum crack width in accordance with Clause 8.6.2.3.

TABLE 8.6.3

MAXIMUM INCREMENT OF STEEL STRESS FOR FLEXURE IN PRESTRESSED BEAMS

Nominal reinforcement bar diameter (d_b) mm	Maximum increment of steel stress MPa
≤12	320
16	280
20	240
24	210
≥28	200
All bonded tendons	200

8.6.4 Crack control in the side face of beams

For crack control in the side face of beams where the overall depth exceeds 750 mm, longitudinal reinforcement, consisting of 12 mm bars at 200 mm centres or 16 mm bars at 300 mm centres, shall be placed in each side face.

8.6.5 Crack control at openings and discontinuities

Reinforcement shall be provided for crack control at openings and discontinuities in a beam.

8.7 VIBRATION OF BEAMS

Vibration of beams shall be considered and action taken, where necessary, to ensure that vibrations induced by machinery, or vehicular or pedestrian traffic, will not adversely affect the serviceability of the structure.

8.8 T-BEAMS AND L-BEAMS

8.8.1 General

Where a slab is assumed to provide the flange of a T-beam or L-beam, the longitudinal shear capacity of the flange-web connection shall be checked in accordance with Clause 8.4.

For isolated T-beams or L-beams, the shear strength of the slab flange on vertical sections parallel to the beam shall also be checked in accordance with Clause 8.2.

8.8.2 Effective width of flange for strength and serviceability

In the absence of a more accurate determination, the effective width of the flange of a T beam or L-beam for strength and serviceability shall be taken as—

(a) T-beams.. $b_{ef} = b_w + 0.2a$; and

(b) L-beams.. $b_{ef} = b_w + 0.1a$,

where a is the distance between points of zero bending moment, which, for continuous beams, may be taken as $0.7L$.

In both Items (a) and (b) above, the overhanging part of the flange considered effective shall not exceed half the clear distance to the next member. The effective width so determined may be taken as constant over the entire span.

8.9 SLENDERNESS LIMITS FOR BEAMS

8.9.1 General

Unless a stability analysis is carried out, beams shall conform with the limits specified in Clauses 8.9.2 to 8.9.4.

8.9.2 Simply supported and continuous beams

For a simply supported or continuous beam, the distance L_l between points at which lateral restraint is provided shall be such that L_l/b_{ef} does not exceed the lesser of 180 b_{ef}/D and 60.

8.9.3 Cantilever beams

For a cantilever beam having lateral restraint only at the support, the ratio of the clear projection (L_n) to the width (b_{ef}) at the support shall be such that L_n/b_{ef} does not exceed the lesser of 100 b_{ef}/D and 25.

8.9.4 Reinforcement for slender prestressed beams

For a prestressed beam in which L_l/b_{ef} exceeds 30, or for a prestressed cantilever beam in which L_n/b_{ef} exceeds 12, the following reinforcement shall be provided:

(a) Stirrups providing a steel area, $A_{sv.min}$ in accordance with Clause 8.2.1.7.

(b) Additional longitudinal reinforcement, consisting of at least one bar in each corner of the compression face, such that—

$$A_{sc} \geq 0.35 A_{pt} f_{pb}/f_{sy} \quad \text{. . . 8.9.4}$$

SECTION 9 DESIGN OF SLABS FOR STRENGTH AND SERVICEABILITY

9.1 STRENGTH OF SLABS IN BENDING

9.1.1 General

The strength of a slab in bending shall be determined in accordance with Clauses 8.1.1 to 8.1.8, except that for two-way reinforced slabs, the minimum strength requirements of Clause 8.1.6.1 shall be deemed to be satisfied by providing tensile reinforcement such that A_{st}/bd is not less than the following in each direction:

(a) Slabs supported by columns at their corners .... $0.24(D/d)^2 f'_{ct.f}/f_{sy}$.

(b) Slabs supported by beams or walls on four sides .... $0.19(D/d)^2 f'_{ct.f}/f_{sy}$.

9.1.2 Reinforcement and tendon distribution in two-way flat slabs

In two-way flat slabs, at least 25% of the total of the design negative moment in a column-strip and adjacent half middle-strips shall be resisted by reinforcement or tendons or both, located in a cross-section of slab centred on the column and of a width equal to twice the overall depth of the slab or drop panel plus the width of the column.

9.1.3 Detailing of tensile reinforcement in slabs

9.1.3.1 *General procedure for arrangement*

Tensile reinforcement shall be arranged in accordance with the following:

(a) Where the bending moment envelope has been calculated, the termination and anchorage of flexural reinforcement shall be based on a hypothetical bending-moment diagram formed by displacing the calculated positive and negative bending-moment envelopes a distance D along the slab from each side of the relevant sections of maximum moment. Additionally, the following shall apply:

   (i) Not less than one third of the total negative moment reinforcement required at a support shall be extended a distance $12d_b$ or D, whichever is greater, beyond the point of contraflexure.

   (ii) At a simply supported discontinuous end of a slab, not less than one half of the total positive moment reinforcement required at midspan shall be anchored by extension past the face of the support for a distance of $12d_b$ or D, whichever is greater, or by an equivalent anchorage.

   Where no shear reinforcement is required in accordance with Clause 8.2.5 or Clause 9.2, the extension of the midspan positive moment reinforcement past the face of the support may be reduced to $8d_b$ if at least one half of the reinforcement is so extended, or to $4d_b$ if all the reinforcement is so extended.

   (iii) At a support where the slab is continuous or flexurally restrained, not less than one quarter of the total positive moment reinforcement required at midspan shall continue past the near face of the support.

   (iv) Where frames incorporating slabs are designed to resist lateral loading, the effects of such loading on the arrangement of the slab reinforcement shall be taken into account but in no case shall the lengths of reinforcement be made less than those shown in Figures 9.1.3.2 and 9.1.3.4.

(b) Where the bending moment envelope has not been calculated, the requirements of Clauses 9.1.3.2, 9.1.3.3 or 9.1.3.4, as appropriate to the type of slab, shall be satisfied.

9.1.3.2 *Deemed-to-conform arrangement for one-way slabs*

For one-way slabs continuous over two or more spans analysed using simplified elastic analysis, as detailed in Clause 6.10.2, where—

(a) the ratio of the longer to the shorter of any two adjacent spans does not exceed 1.2; and

(b) the imposed actions (live loads) may be assumed to be uniformly distributed and the imposed action (live load) (q) is not greater than twice the permanent action (dead load) (g),

the arrangement of tensile reinforcement shown in Figure 9.1.3.2 shall be deemed to conform with Clause 9.1.3.1(a).

Where adjacent spans are unequal, the extension of negative moment reinforcement beyond each face of the common support shall be based on the longer span.

For one-way slabs of single span, the arrangement of tensile reinforcement shown in Figure 9.1.3.2, for the appropriate end support conditions, shall be deemed to conform with Clause 9.1.3.1(a).

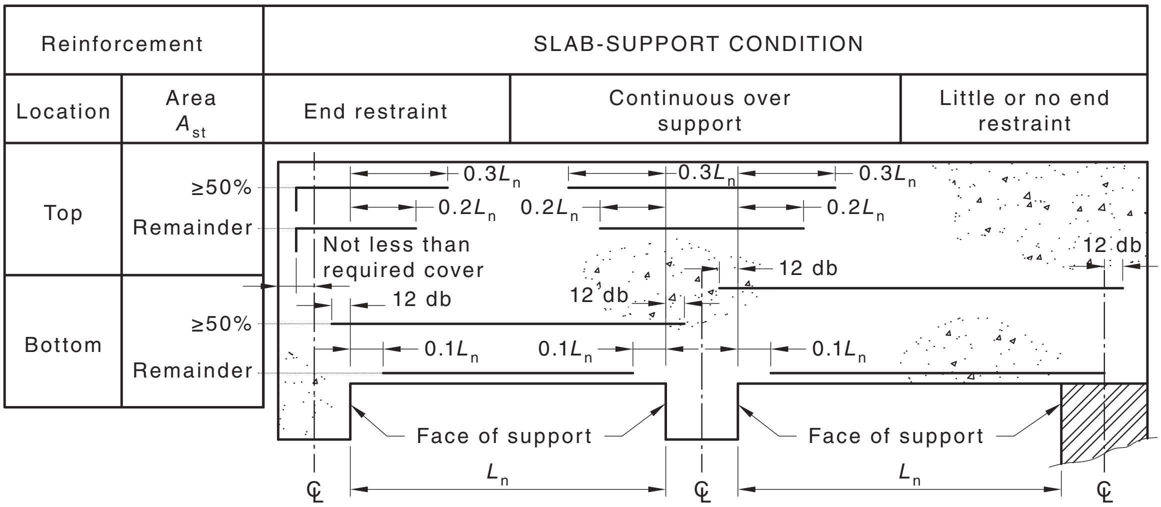

FIGURE 9.1.3.2 ARRANGEMENT OF REINFORCEMENT

9.1.3.3 *Deemed-to-conform arrangement for two-way slabs supported on beams or walls*

For two-way simply supported or continuous rectangular slabs supported by walls or beams on four sides analysed using simplified elastic analysis, as detailed in Clause 6.10.3, the following deemed-to-conform arrangement shall be used and the arrangement of tensile reinforcement, shown in Figure 9.1.3.2 and further prescribed herein, shall be deemed to conform with Clause 9.1.3.1(a):

(a) The arrangement shall apply to each direction.

(b) Where a simply supported or continuous slab is not square, the arrangement shall be based on the span (L_n) taken as the shorter span.

(c) Where adjacent continuous rectangular slabs have unequal shorter spans, the extension of negative moment reinforcement beyond each face of a common support shall be based on the span (L_n) taken as the longer of the shorter spans.

(d) Negative moment reinforcement provided at a discontinuous edge shall extend from the face of the support into the span for a distance of 0.15 times the shorter span.

(e) At an exterior corner of a two-way rectangular slab supported on four sides and restrained against uplift, reinforcement shall be provided in both the top and the bottom of the slab. This reinforcement shall consist of two layers perpendicular to the edges of the slab and extend from each edge for a distance not less than 0.2 times the shorter span. The area of the reinforcement in each of the four layers shall be not less than—

(i) for corners where neither edge is continuous .... $0.75A_{st}$; and

(ii) for corners where one edge is continuous .... $0.5A_{st}$,

where A_{st} is the area of the maximum positive moment reinforcement required at midspan.

Any reinforcement provided may be considered as part of this reinforcement.

9.1.3.4 *Deemed-to-conform arrangement for two-way flat slabs*

For multispan, reinforced, two-way flat slabs analysed using simplified elastic analysis, as detailed in Clause 6.10.4, the following deemed to conform arrangement shall be used and the arrangement of tensile reinforcement, shown in Figure 9.1.3.4 and further prescribed herein, shall be deemed-to-conform with Clause 9.1.3.1(a).

Where adjacent spans are unequal, the extension of negative moment reinforcement beyond each face of the common support shall be based on the longer span.

All slab reinforcement perpendicular to a discontinuous edge shall be extended (straight, bent or otherwise) past the internal face of the spandrel, wall or column for a length—

(a) *for positive moment reinforcement*, not less than 150 mm except that it shall extend as close as permitted to the edge of the slab if there is no spandrel beam or wall; and

(b) *for negative moment reinforcement*, such that the calculated force is developed at the internal face in accordance with Clause 13.1.

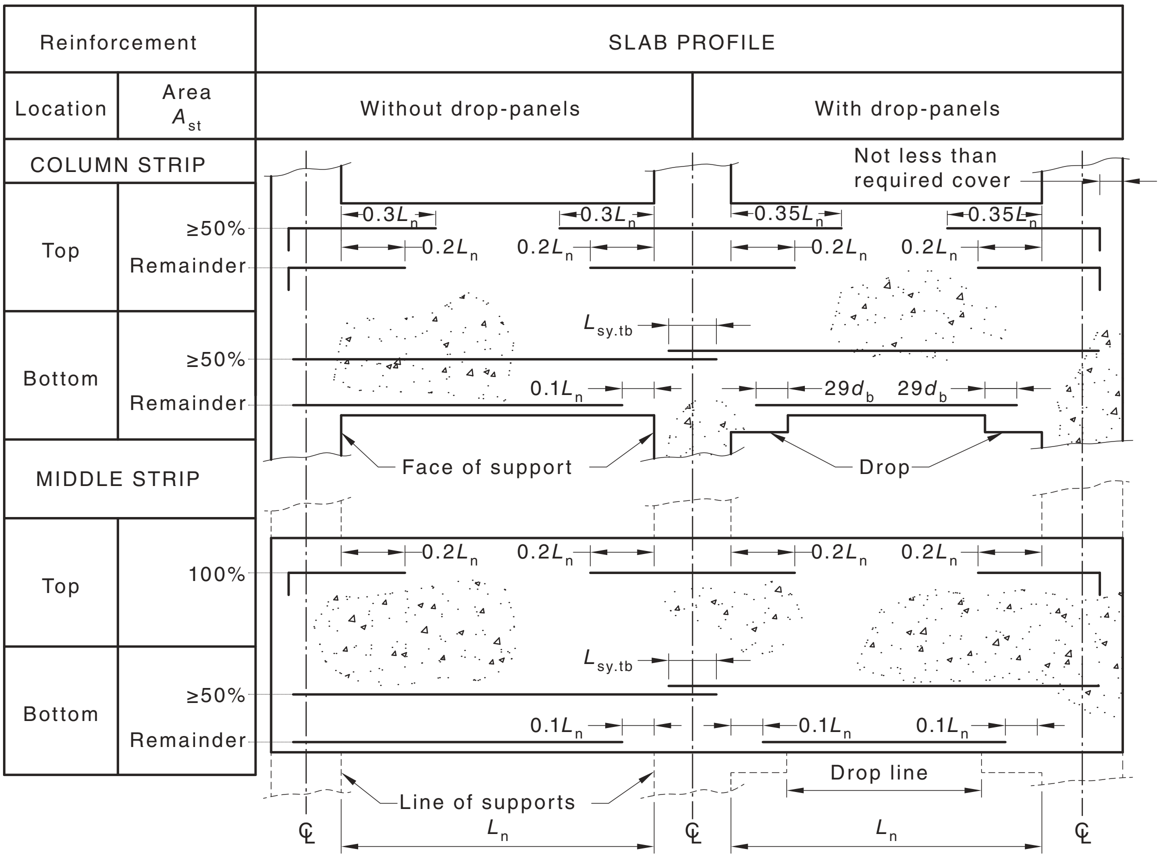

FIGURE 9.1.3.4 ARRANGEMENT OF REINFORCEMENT

9.2 STRUCTURAL INTEGRITY REINFORCEMENT

9.2.1 General

Reinforcement at connections shall be provided to increase the resistance of the structural system to progressive collapse at walls and columns.

At least two of the column strip bottom bars or strands in each direction shall pass within the region bounded by the longitudinal reinforcement of the column and shall be continuous through the interior support and fully anchored beyond the face of the exterior support.

9.2.2 Minimum structural integrity reinforcement

The summation of the area of bottom reinforcement connecting the slab, drop panel, or slab band to the column or column capital on all faces of the periphery of a column or column capital shall be not less than,

$$A_{s.min} = \frac{2N^*}{\phi f_{sy}} \qquad \ldots 9.2.2$$

in which N^* is the column reaction from the floor slab at the ultimate limit state.

Integrity reinforcement shall not be required if there are beams containing shear reinforcement and with at least two bottom bars continuous through the joint in all spans framing into the column.

This reinforcement shall be placed within the column core and may consist of the following:

(a) Bottom reinforcement extended to lap over the column in accordance with Clause 13.2.

(b) Extra bars over the column extending a minimum distance of 2 $L_{sy.tb}$ from the face of the column or column capital and lapping with any existing bottom reinforcement. These bars should have hooked or cogged ends where terminating at discontinuous edges.

9.2.3 Minimum reinforcement for distributing loads

Minimum reinforcement in a secondary direction shall be provided for the purpose of distributing loads.

NOTE: For shrinkage and temperature effects see Clause 9.5.3.

9.2.4 Spacing of reinforcement and tendons

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 17.1.3.

The maximum spacing of reinforcement and tendons for crack control shall be determined in accordance with Clause 9.5.

Where the plain concrete between tendons is unable to safely distribute the applied loads to the tendons, reinforcement for this purpose shall be supplied transverse in a prestressed slab. Unless calculations show that a wider spacing is possible, the maximum spacing of tendons in an unreinforced slab subjected to uniformly distributed loads shall be the lesser of 10 times the slab thickness and 1500 mm.

NOTE: The tendon spacing adjacent to a supporting column will be controlled by Clause 9.1.2 and may need to be significantly less than the above limits.

9.3 STRENGTH OF SLABS IN SHEAR

9.3.1 Definitions and symbols

For the purpose of this clause, the definitions and symbols below apply to flat slabs.

9.3.1.1 *Effective area of a support or concentrated load*

The area totally enclosing the actual support or load and for which the perimeter is a minimum [see Figure 9.3(A)].

9.3.1.2 *Critical opening*

Any opening through the thickness of a slab where an edge, or part of the edge, of the opening is located at a clear distance of less than $2.5b_o$ from the critical shear perimeter [see Figure 9.3(A)].

9.3.1.3 *Critical shear perimeter*

The perimeter defined by a line geometrically similar to the boundary of the effective area of a support or concentrated load and located at a distance of $d_{om}/2$ therefrom [see Figure 9.3(A)].

9.3.1.4 *Symbols*

The following symbols apply:

a = dimension of the critical shear perimeter measured parallel to the direction of M_v^* [see Figure 9.3(B)]

b_o = dimension of an opening [see Figure 9.3(A)]

b_w = width of the web of a spandrel beam [see Figure 9.3(B)]

D_b = overall depth of a spandrel beam [see Figure 9.3.6]

D_s = overall depth of a slab or drop panel

d_{om} = mean value of d_o, averaged around the critical shear perimeter

M_v^* = bending moment transferred from the slab to a support in the direction being considered [see Figure 9.3(B)]

u = length of the critical shear perimeter [see Figure 9.3(A)]

y_1 = larger overall dimension of a closed fitment (see Figure 9.3.6)

β_h = ratio of the longest overall dimension of the effective loaded area, Y, to the overall dimension, X, measured perpendicular to Y [see Figure 9.3(A)]

9.3.1.5 *Torsion strip*

A strip of slab of width a, whose longitudinal axis is perpendicular to the direction of M_v^* [see Figure 9.3(B)].

9.3.2 Strength

The strength of a slab in shear shall be determined in accordance with the following:

(a) Where shear failure can occur across the width of the slab, the design shear strength of the slab shall be calculated in accordance with Clause 8.2.

(b) Where shear failure can occur locally around a support or concentrated load, the design shear strength of the slab shall be taken as ϕV_u, where V_u is calculated in accordance with one of the following:

  (i) Where M_v^* is zero, V_u is taken as equal to V_{uo} calculated in accordance with Clause 9.3.3.

  (ii) Where M_v^* is not zero, V_u is calculated in accordance with Clause 9.3.4.

NOTE: For types of shear reinforcement other than those covered in Clauses 9.3.3 and 9.3.4 strength may be determined by tests, in accordance with Appendix B.

9.3.3 Ultimate shear strength where M_v^* is zero

The ultimate shear strength of a slab where M_v^* is zero, V_{uo} is given by either—

(a) where there is no shear head—

$$V_{uo} = ud_{om}(f_{cv} + 0.3\sigma_{cp}) \qquad \ldots 9.3.3(1)$$

where

$$f_{cv} = 0.17\left(1+\frac{2}{\beta_h}\right)\sqrt{f'_c} \le 0.34\sqrt{f'_c}\text{; or}$$

NOTE: The value of σ_{cp} should be evaluated separately for the case of corner, edge and internal columns.

(b) where there is a shear head—

$$V_{uo} = ud_{om}(0.5\sqrt{f'_c} + 0.3\sigma_{cp}) \le 0.2ud_{om}f'_c \qquad \ldots 9.3.3(2)$$