SENSE 600®

SENSE 600®
Design Guide for Reinforced Concrete Columns

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Document: T02 V0.1

Preface

The demand for sustainable construction has driven product manufacturers in Australia and around the world to conduct research and development into innovative products that offer improved environmental credentials. Given steel is already one of the most recycled materials in construction, and certainly the highest in terms of value, the logical progression is to look further up the waste hierarchy of - Reduce, Reuse and Recycle. The top of the hierarchy is obviously the best and this is where InfraBuild Steel has a significant focus. InfraBuild Steel has conducted extensive research and development in our own facilities and in collaboration with leading Universities in Australia, to produce higher grades of steel designed to reduce the mass of steel consumed in the search for more sustainable construction solutions.

Changes to Australian Standards, that now recognise these higher grades, will facilitate adoption in design and construction using these steels. In 2018 changes to AS 3600 - Concrete structures (Standards Australia 2018) and in 2019 changes to AS/NZS 4671 - Steel for the reinforcement of concrete (Standards Australia 2019), have provided the reinforced concrete industry the opportunity to explore the benefits offered by higher strength, ductile reinforcing steels. The design models in AS 3600:2018 now apply for reinforcing steels with yield strengths up to 800 MPa for column fitments and up to 600 MPa for all other elements.

Significant sustainability benefits can be achieved using higher strength steels particularly in reinforced concrete elements that are governed predominately by strength rather than serviceability. Australian Standards usually offer designers options for different tiers of design. A lower tier will generally result in a simpler model with a more conservative result while a higher tier will require a more complex model but will result in a design which better utilises the capacity of the design element.

Options with different tiers of complexity are offered in AS 3600 in relation to column design. The rectangular stress block method is the lowest tier and the preferred method if calculations are to be completed by hand. However, given most, if not all designers will have access to column design software on the computer sitting on their desk and some even on the phone in their pocket on site, there is little need to resort to this lowest tier. Commercial design software typically utilises a curvilinear method rather than the rectangular stress block method because of the improved results. Furthermore, the lower tier of design offered by the rectangular stress block method currently is not able to efficiently utilise the additional strength offered by 600 MPa steels over 500 MPa steels.

While AS 3600 provides clear and prescriptive guidance to designers on how to implement the rectangular stress block method it only provides a series of guiding principles on how to implement the curvilinear method. This publication provides guidance on how to implement the curvilinear method in accordance with the provision of AS 3600: 2018.

Contents

1. Scope

This Design Guide is concerned with the strength design of the cross-section of reinforced concrete columns subjected to combined axial force and bending. The rectangular stress block method is well covered in the Australian Standard AS 3600 – Concrete structures and it is not intended that the method will be covered in this document. A curvilinear method is also permitted by AS 3600, however in comparison, guidance on how to apply the method is not as well detailed as it is for the rectangular stress block method. The curvilinear method is important for designs with 600 MPa reinforcing bars as it better utilises the additional capacity of these bars when compared to 500 MPa bars.

The Design Guide outlines detailed steps and provides worked examples on an option that satisfies the requirements of the curvilinear method permitted by AS 3600.

Also covered in this Design Guide are other aspects to improve the efficiency of the design which are not excluded by AS 3600 or is an extension to the Standard.

The effects of slenderness on column design is outside the scope of this Design Guide as it is adequately covered by other texts on designing reinforced concrete to AS 3600.

2. AS 3600 Design Provisions

AS 3600:2018, Clause 10.6.1, (Standards Australia 2018) Basis of strength calculations, provides an outline for determining the longitudinal reinforcing bars required in a column section–

(a) *Plane sections normal to the axis remain plane after bending.*

(b) *The concrete has no tensile strength.*

(c) *The distribution of stress in the concrete and the steel is determined using a stress-strain relationship determined from recognized simplified equations or determined from test data*

(d) *The strain in compressive reinforcement does not exceed 0.003.*

(e) *Where the neutral axis lies outside of the cross-section, consideration shall be given to the effect on strength of spalling of the cover concrete.*

Two options are offered to calculate the strength of the cross section and develop an Axial Load/ Moment interaction curve - a simplified method using the rectangular stress block or the option to use a curvilinear stress-strain relationship.

The rectangular stress block method limits the concrete strain under concentric load to 0.0025 and the ultimate concrete strain at the extreme fibre is limited to 0.003 when the section is subjected to bending. These limits in the concrete strain and the compatibility of the composite section, limit the steel strain under squash load to 0.0025 and to less than 0.003 when the column section is resisting an axial force and moment.

If a curvilinear stress block is adopted, the stress-strain relationship required by the Standard is “defined by recognized simplified equations” or “determined by test data”. The Note to Clause 10.6.1 reads:

1) *If a curvilinear stress-strain relationship is used, then –*
    a) *Clause 3.1.4 places a limit on the value of the maximum concrete stress; and*
    b) *the strain in the extreme fibre may be adjusted to obtain the maximum bending strength for a given axial load.*
2) *The effect of the confinement on the strength of a section may be taken into account, provided secondary effects such as concrete spalling, for example, are also considered.*

2.1 Curvilinear Stress-Strain Relationship

The curvilinear stress-strain relationship is a higher tier model than that of the rectangular stress block, which arguably offers a higher degree of accuracy, and better utilisation of the additional capacity offered by 600 MPa steels (Ng and McGregor 2021). This approach offers better utilisation because the ultimate concrete strain is not limited to 0.003.

AS 3600 Clause 10.6.1 (c) refers to Clause 3.1.4 for the concrete stress-strain relationship. This acknowledges that the relationship is curvilinear and allows users to utilise "recognized simplified equations" or the properties "determined from test data". The AS 3600 Supplement (Standards Australia 2022), commonly referred to as the Commentary (Clause C8.1.2), offers two options:

1) A curvilinear model for both the ascending and descending branches of the compressive stress-strain relationship (refer to Figure 2.1a and AS 3600 Commentary C3.1.4), requires that the strain at the extreme compressive fibre at ultimate load should be selected such that the moment on the section is maximized when the rules of equilibrium and strain compatibility are applied. This is a complex calculation.

2) A curvilinear stress block model with a curvilinear ascending branch followed by a constant value of stress equal to the peak stress with the extreme fibre compressive strain, dependant on the concrete strength. One such representation is provided in the *fib* Model Code (2013) and given by:

$$\varepsilon_{cu} = 0.0026 + 0.035 \left(\frac{90 - f'_c}{100}\right)^4 \leq 0.0035 \qquad \text{Eq. (2.1)}$$

Figure 2.1 show plots of the stress-strain relationship for the curvilinear model and the curvilinear stress block model.

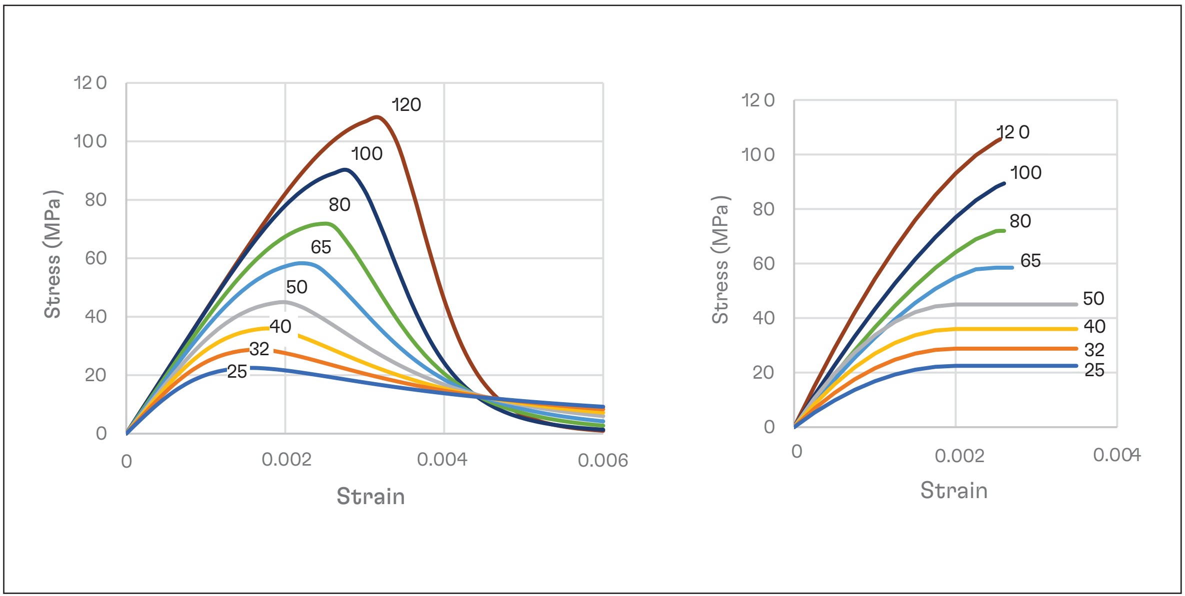


Figure 2.1 – Concrete stress strain relationships

Figure 2.2 shows diagrammatically the difference between the stress distribution on the cross section of a column subjected to a moment and an axial force, for both models.

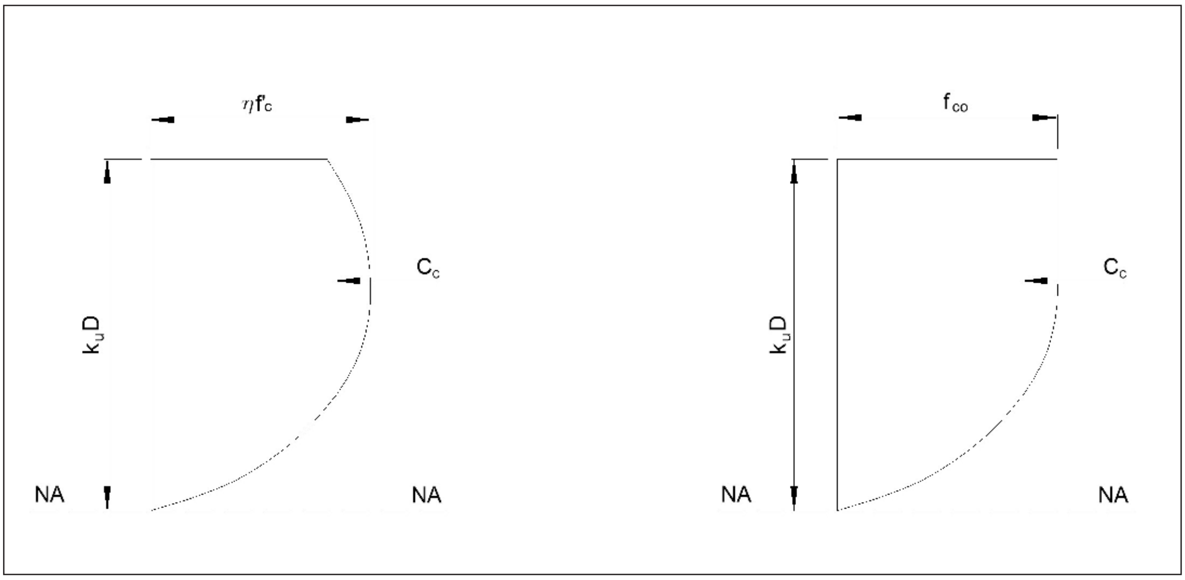


Figure 2.2 – The stress distribution on the cross setion of a column subjected to a moment and an axial force using simplified recgonized concrete stress strain relationships

2.2 Rectangular – Parabolic stress block

The curvilinear model with the stress block as shown in Figure 2.2 (a) is arguably the option that most closely models the actual behaviour of a column in combined bending and compression. However, the rectangular – parabolic stress block model is the model commonly adopted by designers because it matches very closely the curvilinear model and is simple to use because the stress distribution is easily calculated as is the centroid of the compression block. The centroid is simply the resultant of the rectangular and parabolic stress blocks.

2.2.1 Design stress-strain curves

AS 3600, Clause 3.1.4 requires designers to use a stress-strain relationship defined by recognized simplified equations. Given that AS 3600 is modelled on the *fib* Model Code (2013), the obvious choice is to use the rectangular-parabolic stress-strain relationship given in that publication. The specific equations are (adjusted for use with the phi-factor approach of AS3600):

$$\varepsilon_{c2} = 0.002 \qquad \text{for } f'_c \leq 50\text{MPa} \qquad \text{(Eq. 2.2.1.1a)}$$

$$\varepsilon_{c2} = 0.002 + 0.000085\left(f'_c - 50\right)^{0.53} \leq 0.0035 \qquad \text{for } f'_c > 50\text{MPa} \qquad \text{(Eq. 2.2.1.1b)}$$

$$\varepsilon_{cu2} = 0.0026 + 0.035\left(\frac{90 - f'_c}{100}\right)^4 \leq 0.0035 \qquad \text{for } 50\text{MPa} < f'_c \leq 90\text{MPa} \qquad \text{(Eq. 2.2.1.2)}$$

$$\varepsilon_{cu2} = 0.0026 - 0.035\left(\frac{90 - f'_c}{100}\right)^4 \leq 0.0035 \qquad \text{for } f'_c > 90\text{MPa} \qquad \text{(Eq. 2.2.1.2b)}$$

$$n = 2 \qquad \text{for } f'_c \leq 50\text{MPa} \qquad \text{(Eq. 2.2.1.3a)}$$

$$n = 1.4 + 0.0234\left(\frac{90 - f'_c}{100}\right)^4 \qquad \text{for } 50\text{MPa} < f'_c \leq 90\text{MPa} \qquad \text{(Eq. 2.2.1.3b)}$$

$$n = 1.4 - 0.0234\left(\frac{90 - f'_c}{100}\right)^4 \qquad \text{for } f'_c > 90\text{MPa} \qquad \text{(Eq. 2.2.1.3c)}$$

$$f'_{co} = 0.9 f'_c \qquad \text{(Eq. 2.2.1.4)}$$

$$\sigma_c = f_{co}\left[1 - \left(1 - \frac{\varepsilon_c}{\varepsilon_{c2}}\right)^n\right] \qquad \text{for } 0 \leq \varepsilon_c \leq \varepsilon_{c2} \qquad \text{(Eq. 2.2.1.5a)}$$

$$\sigma_c = f_{co} \qquad \text{for } 0 \leq \varepsilon_c \leq \varepsilon_{c2} \qquad \text{(Eq. 2.2.1.5b)}$$

Where:

f'_c is the characteristic compressive strength of concrete

The preceding formulae were used to calculate the values for the common concrete strengths available in Australia and are shown in Table 2.1 and the plot of stress-strain relationships are shown in Figure 2.2b.

Table 2.1 Stress - Strain relationship for concretes referenced in AS 3600

Concrete Strength (MPa)	f'_c (MPa)						
	25	**32**	**40**	**50**	**65**	**80**	**100**
ε_{c2}	0.002	0.002	0.002	0.002	0.0024	0.0025	0.0027
ε_{cu2}	0.0035	0.0035	0.0035	0.0035	0.0027	0.0026	0.0027
n	2.0	2.0	2.0	2.0	1.49	1.40	1.40
f_{co}	23	29	36	45	59	72	90
ε_c				σ_c			
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.00025	5.3	6.8	8.4	10.5	9.0	9.8	11.5
0.0005	9.8	12.6	15.8	19.7	17.5	19.2	22.6
0.00075	13.7	17.6	21.9	27.4	25.5	28.2	33.2
0.001	16.9	21.6	27.0	33.8	32.8	36.6	43.2
0.00125	19.3	24.8	30.9	38.7	39.5	44.5	52.7
0.0015	21.1	27.0	33.8	42.2	45.6	51.8	61.5
0.00175	22.1	28.4	35.4	44.3	50.8	58.4	69.6
0.002	22.5	28.8	36.0	45.0	55.0	64.2	76.9
0.00225	22.5	28.8	36.0	45.0	57.9	68.9	83.1
0.00250	22.5	28.8	36.0	45.0	58.5	71.9	88.0
0.00255	22.5	28.8	36.0	45.0	58.5	72.0	88.7
0.00260	22.5	28.8	36.0	45.0	58.5	72.0	89.4
0.00270	22.5	28.8	36.0	45.0	58.5		
0.00280	22.5	28.8	36.0	45.0			
0.00290	22.5	28.8	36.0	45.0			
0.00300	22.5	28.8	36.0	45.0			
0.00310	22.5	28.8	36.0	45.0			
0.00320	22.5	28.8	36.0	45.0			
0.00330	22.5	28.8	36.0	45.0			
0.00340	22.5	28.8	36.0	45.0			
0.00350	22.5	28.8	36.0	45.0			

3. Strength of cross section calculated using a rectangular-parabolic stress block

AS 3600 Clause 10.6.2 details how the rectangular stress block can be used to calculate the strength of a column cross section for various bending and compression combinations prior to the application of a capacity reduction factor, ∅. The strength of a particular cross section can then be represented by a strength interaction diagram. A similar approach can be used with the rectangular-parabolic stress block.

There are four key points in a strength interaction diagram, they are:

1. Squash load
2. Decompression point
3. Balanced point
4. Pure bending point

Arguably, the simplest way to apply the rectangular-parabolic stress block to AS 3600 and ensure conformity is to use the same method as outlined for the rectangular stress block. That is, use each of the Clauses 10.6.2.2 to 10.6.2.5 and just substitute the rectangular-parabolic stress block for the rectangular stress block.

3.1 Squash load

Confinement of the concrete provided by fitments modifies the stress-strain relationship. However, in this section of the Guide the same methodology AS 3600 prescribes for the rectangular stress block is used for the rectangular-parabolic stress block. Section 5.1. of this guide examines the options permitted by AS 3600 to model confinement and the impact it has on the squash load.

$$N_{u0} = (\propto_1 \times f'_C \times A_c) + (A_s \times \sigma_s) \qquad \text{(Eq. 3.1 (a))}$$

Where

$$\propto_1 = 1.0 - 0.003\, f'_c \text{ with } 0.72 \leq \propto_1 \leq 0.85 \qquad \text{(Eq. 3.1 (b))}$$

σ_s = steel stress with a maximum strain in the reinforcement ($\mathcal{E}_s$) of 0.0025

600 MPa reinforcing steel has a modulus of elasticity of 200 MPa, the same as a 500 MPa reinforcing steel. The relationship between the stress and strain of a reinforcing bar is:

$$E_s = {}^{\sigma_s}/_{\mathcal{E}_s} \qquad \text{(Eq. 3.1(c))}$$

Rearranging gives $\sigma_s = E_s \times \mathcal{E}_s$ (Eq. 3.1(d))

Substituting gives $= 200 \times 10^3 \times 0.0025$ (Eq. 3.1(e))

Evaluating gives $= 500\ MPa$ (Eq. 3.1 (f))

This indicates that at the squash load point, the maximum strain limit set at 0.0025 for the reinforcing steel limits the capacity of a 600 MPa bar to 500 MPa.

Thus

$$N_{u0} = (\propto_1 \times f'_C \times A_c) + (A_s \times \sigma_s) \text{ where } \sigma_s \leq 500 \text{ MPa} \qquad \text{(Eq. 3.1 (g))}$$

3.2 Decompression point

The decompression point is calculated taking the strain in the extreme compression fibre equal to the ultimate concrete strain (ε_{cu}) and the extreme tensile fibre strain equal to zero. The neutral axis is at the full depth of section.

3.3 Balanced point

The balanced point is where the outermost layer of reinforcing steel has reached its yield stress and the extreme concrete fibre has just reached its ultimate value. For 600 MPa reinforcing steel ε_S= 0.003. The neutral axis depth at the balanced point is $k_{ub}d$ where the value of k_{ub} is given by the formula –

$$k_{ub} = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_S} \qquad \text{(Eq. 3.3(a))}$$

Where

$$\varepsilon_S = E_S / f_{sy} \qquad \text{(Eq. 3.3(b))}$$

3.4 Pure bending point

The pure bending point occurs when the sum of the compressive forces is equal to the sum of the tensile forces. To find this point an iterative process varying the value is adopted until the sum of the forces is equal to zero.

3.5 Transition from the squash load to the decompression point

AS 3600 states that where the neutral axis lies outside the section, the section strength may be calculated using a linear relationship between the decompression point and the squash load. It is reasonable (and conservative) to adopt the same approach for the rectangular-parabolic stress block; however, if the calculations are produced using software or a spreadsheet then it is just as easy to set for an appropriate range of values greater than 1 up to 100 and calculate the corresponding moments and axial force capacities.

3.6 Transition from the decompression point to the pure bending point

Where the neutral axis lies within the cross section the maximum strain shall be taken as that given in Table 2.1. The values along this part of the curve can be determined by varying the value of k_u from the value of k_u determined at the pure bending point up to the decompression point where k_u = D/d_0

3.7 Capacity reduction factor

The design capacity of the section is determined by multiplying the axial force capacity and its corresponding moment capacity by the capacity reduction factor, phi (∅). Table 2.2.2 of AS 3600 provides the phi factors and is summarised in Table 3.7.1.

Table 3.7.1 Capacity reduction factors

Squash Load	Capacity reduction factors (∅)
$N_u \geq N_{ub}$	$\varnothing = \varnothing_0$ where $\varnothing_0$ = 0.65 for Short Columns and Q/G≥ 0.25 $\varnothing_0$ = 0.60 otherwise Q = Live load & G = Dead load
$N_u < N_{ub}$	$\varnothing = \varnothing_0 + [(\varnothing' - \varnothing_0)(1 - N_u/N_{ub})]$ Where ∅' is $0.65 \leq 1.24 - 13k_{u0}/12 \leq 0.85$

It is evident from Table 3.7.1 that $\varnothing_0$ is dependent on the slenderness (column effective length) and the dead to live load ratio of the column and not the method for calculating the cross-sectional strength.

4. AS 3600 worked example

Two worked examples are presented to illustrate how the rectangular-parabolic stress method can be applied to: - a) Rectangular columns and b) Circular columns

4.1 Rectangular column

Consider a rectangular column 600 x 400 mm shown in Figure 4.1.1 reinforced with 8 S26 bars.

b = 400mm

D= 600mm

Reinforcing steel

Longitudinal - 8S26 (600MPa)

Fitments - S11@300 (600MPa)

f_c = 40MPa;

Cover = 40mm

d= 600 – 40 – 11 – 26/2 = 536mm

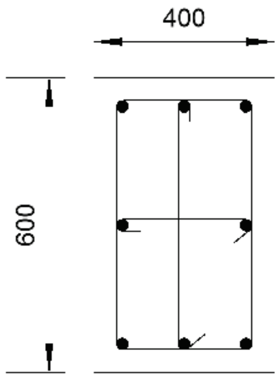


Figure 4.1.1 – Design Example 1: Rectangular column cross section

The corresponding stress-strain curve for 40 MPa concrete is shown in Figure 4.1.2

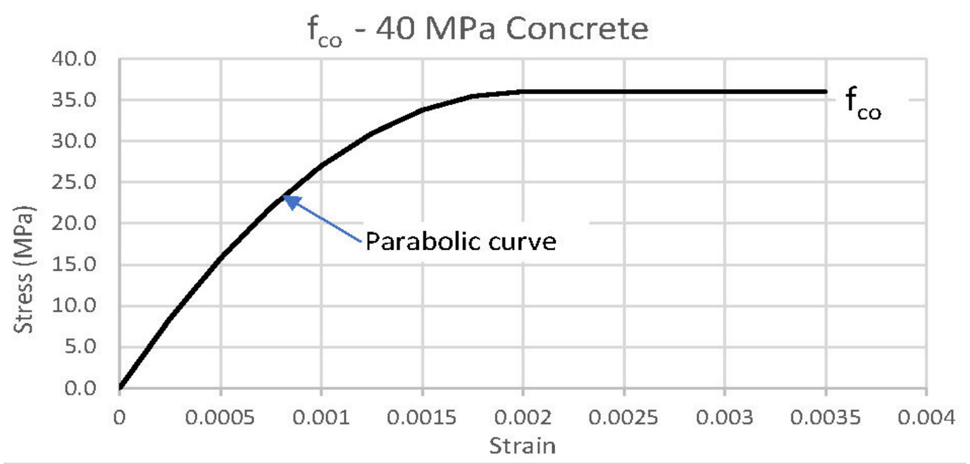


Figure 4.1.2 – Stress strain curve for 40MPa concrete

Calculate the 4 points as shown on the curve in Figure 4.1.3

1) Squash Load N_{uo}

2) Decompression point M_{uD} , N_{uD}

3) Balanced point M_{ub}, N_{ub}

4) Pure Bending M_{uo}

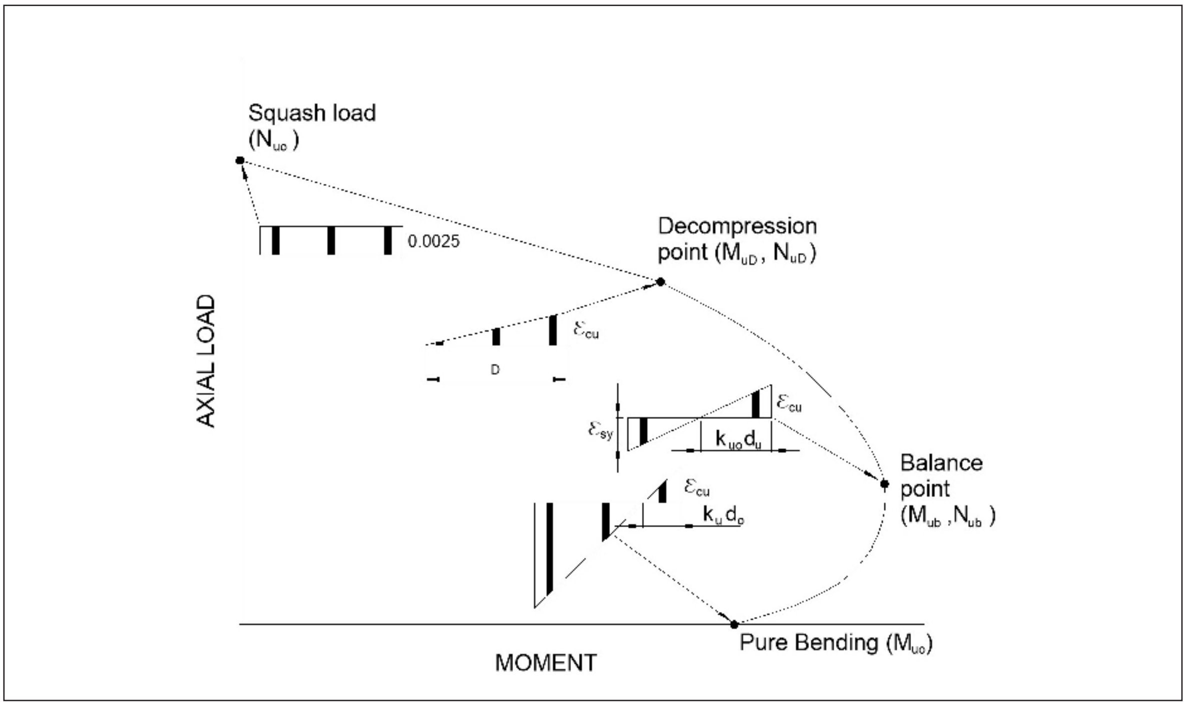


Figure 4.1.3 – Key points on the Axial Load: Moment interaction curve

4.1.1 Axial load

Use Eq. 3.1(g) to calculate the squash load.

$$N_{uo} = (\alpha_1 \text{ x } f'_c \text{ x } A_c) + (A_s \text{ x } \sigma_s) \textit{ where}$$

f'_c = 40 MPa

Where α_1 using Eq 3.1(b) is:

α_1 : 0.72 ≤ 1.0 - 0.003 x 40 ≤ 0.85

α_1 = 0.85

$A_s = 8 \text{ x } \dfrac{\pi \text{ x } 25.6^2}{4} = 4{,}118 \text{mm}^2$

A_c = 400 x 600 – 4118 = 235,882mm^2

σ_s = 500 *MPa*

Substituting gives:

N_{uo} = (0.85 x 40 x 235,882) + (4,118 x 500)

= 10.078988 N

= 10080 kN

Applying the capacity reduction factor (∅) as required by AS3600 and table 3.7.1 gives the following design capacities depending on the column effective length and the dead (G) to live load ratio (Q).

Table 4.1.1.1 Design Capacity for each of the two reduction factors

Capacity Reduction Factor	∅ N_{uo} (kN)	∅*Mu* (kNm)
∅ = 0.65	6551	0
∅ = 0.6	6047	0

4.1.2 Decompression point

At the decompression point the distance to the neutral axis D = 600 and the value of k_u = *D/d*$_0$.

For a concrete with f'c = 40 MPa, from Table 2.1, ε_{c2} = 0.002 and ε_{cu} = 0.0035.

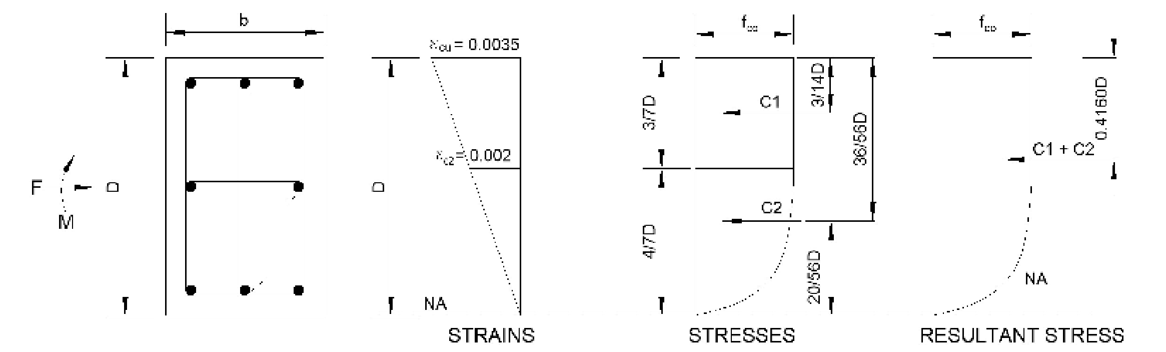


Figure 4.1.2.1 – Strains and resultant concrete stresses at the decompression point

Referring to Figure 4.1.2.1 above, the following can be deduced

1. The strain varies linearly from 0, at the neural axis to ε_{cu} (= 0.0035) at the extreme compressive fibre.
2. Applying similar triangles, an expression for where strain reaches ε_{c2} (= 0.002) is determined to be a distance of 4/7D from the neutral axis.
3. The compressive force in the concrete increases parabolically from zero, at the neutral axis to, f_{co} (36 MPa) and then has a constant value of f_{co} for the remaining 3/7D to the extreme compression face.
4. C1 represents the compressive force equal to f_{co} x 3/7D x b (stress x area)
5. C1 acts at 3/14D (3/7D/2) below the extreme compression face. Refer to Appendix B for geometrical properties of a parabola.
6. C2 represents the compressive force equal to 2/3 x f_{cd} x 4/7D x B (stress x area). The stress being equivalent to half a parabola (2/3 f_{oo}) and the area being 4/7d x B.
7. C2 acts through the centroid of the parabola 20/56d ($^5/_8$ X $^4/_7$D) above the neutral axis
8. The two forces C1 and C2 can be summed to give a resultant force C_c, where

   C_c = 0.8095 x f_{co} x b x D (Eq. 4.1.2)

   acting 0.4160D below the compressive fibre.

The strain distribution and the resultant forces are shown in Figure 4.1.2.2.

The resultant forces are summed to determine the axial capacity and moments are taken about the plastic centroid of the section to determine the flexural capacity. The results are presented in Table 4.1.2.1 with the convention that compressive forces are positive.

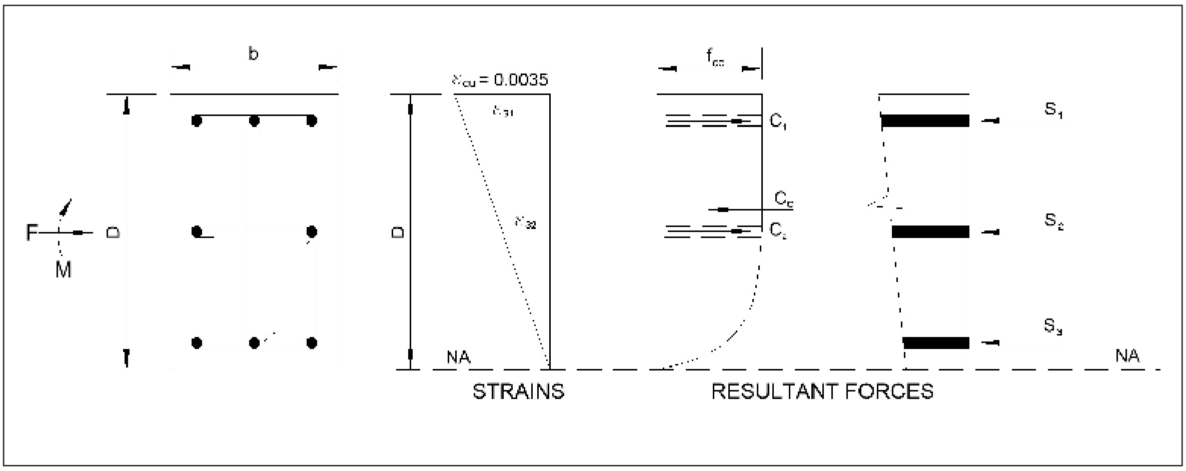


Figure 4.1.2.2 – Strains and resultant forces at the decompression point

Table 4.1.2.1 Resultant forces at the decompression point

Force	x_i (mm)	ε_i	σ_i or $f_{co.i}$ (MPa)	A_i (mm²)	F_i (kN)	$D/2-x_i$ (mm²)	M_i (kNm)
S_1	64	0.0031	600	1544	926	236	219
S_2	300	0.0018	350	1029	360	0	0
S_3	536	0.0004	74	1544	115	-236	-27
C_c	250				6994	50	353
C_1	64	0.0031	-36	1544	-56	236	-13
C_2	300	0.0018	-35	1029	-36	0	0
C_3	536	0.0004	-8.4	1544	-19	-236	3
Total					**8285**		**535**

Note that the values in Table 4.1.2.1 considers the double counting that would otherwise occur because the reinforcing bars in reinforcing layers 1 and 2 have displaced concrete and thus reduced the resultant concrete compressive force, C_c. An adjustment can be made by subtracting C_1 and C_2 corresponding to the displaced concrete and appropriate f'_c at each layer.

At the decompression point the calculated capacities are:

N_{uD} = 8285 kN; M_{uD} = 535 kNm

Applying the capacity reduction factor (∅) as required by AS 3600 and Table 3.7.1 gives the following design capacities depending on the column effective length and the dead to live load ratio.

Table 4.1.2.2 Design Capacities at the decompression point for each of the two reduction factors

Design Value	$\varnothing N_{uD}$ (kN)	∅MuD(kNm)
∅ = 0.65	5385	348
∅ = 0.6	4971	321

4.1.3 Balanced point

At the balanced point Eq. 3.3(a) provides the value of $k_u = k_{ub}$ where

$$k_{ub} = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_s}$$

And Eq. 3.3 (b) gives $\qquad \varepsilon_s = E_s/f_{sy}$

For a 600 MPa Steel

$$\varepsilon_s = \frac{2 \times 10^5}{600} = 0.003$$

$$k_{ub} = \frac{0.0035}{0.0035 + 0.0035} = 0.5385$$

The depth to the neutral axis is

$$d_b = k_{ub} \times d$$
$$= 0.5385 \times 536$$
$$= 289\text{mm}$$

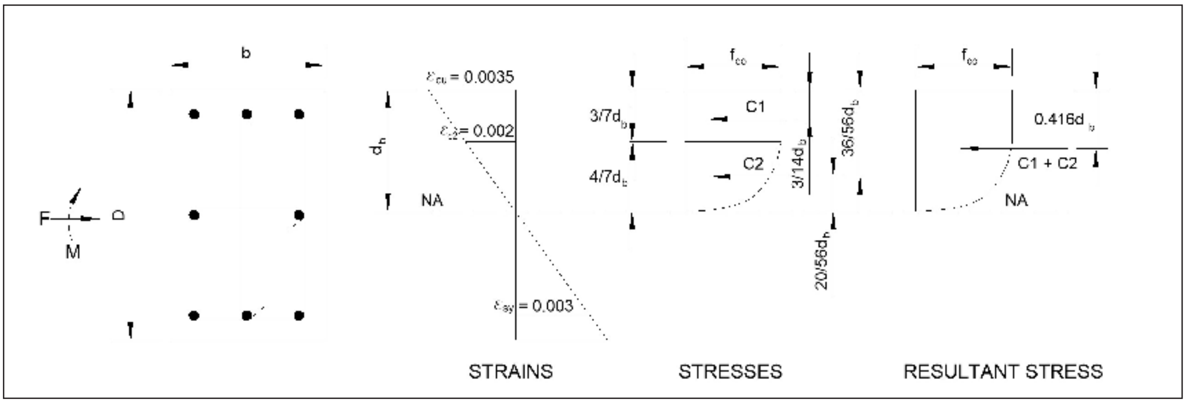


Figure 4.1.3.1 – Strains and resultant concrete stresses at the balanced point

An examination of the stress-strain distribution and resultant stresses for the balanced point shown in Figure 4.1.3.1 compared with those for the decompression point shown in Figure 4.1.2.1 reveals a degree of similarity. Since the expressions for the resultant force and where it acts in Eq. 4.1.1 are expressed in terms a dimension D the same derived expressions can be used for the balanced point by replacing D with d_b. Hence at the balanced point

$C_c = 0.8095 \times f_{co} \times B \times d_b$ (Eq. 4.1.3 (a))

acting at $0.4160d_b$ below the compression face as shown in Figure 4.1.3.2.

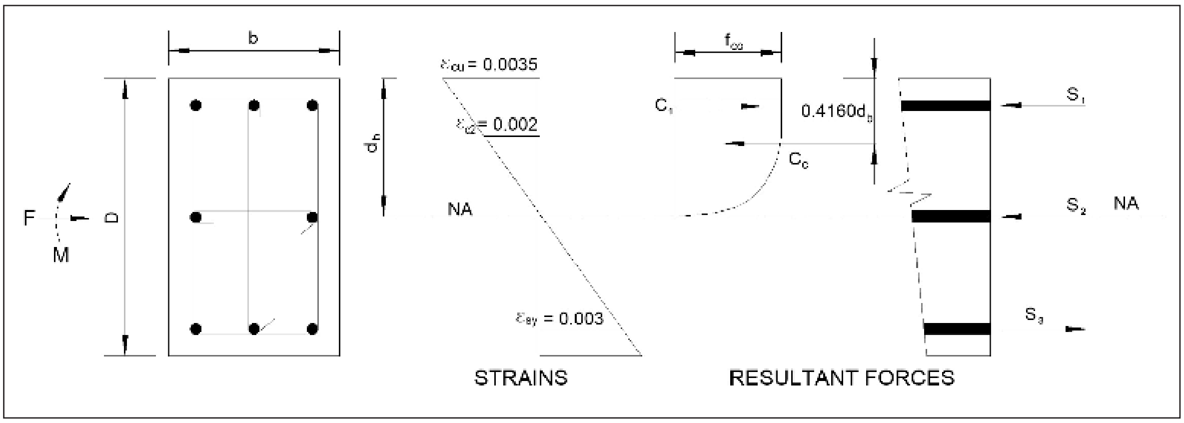


Figure 4.1.3.2 – Strains and resultant forces at the balanced point

The resultant forces are summed to determine the axial capacity and moments are taken about the plastic centroid to determine the moment capacity for the balanced point. The results are presented in Table 4.1.3.1. Note that these values considers the double counting that would otherwise occur because the reinforcing bars in layer 1 have displaced concrete and thus reduces the calculated resultant concrete compressive force, C_c. An adjustment can be made by subtracting C_1 corresponding to the displaced concrete and appropriate $f_{co.i}$ at that layer.

Table 4.1.3.1 Resultant forces at the balanced point

Force	x_i (mm)	ε_i	σ_i or $f_{co.i}$ (MPa)	A_i (mm^2)	F_i (kN)	D/2-x_i (mm^2)	M_i (kNm)
S_1	64	0.0027	545	1544	842	236	199
S_2	300	-0.0001	-27	1029	-28	0	0
S_3	536	-0.0030	-600	1544	-926	-236	219
C_c	120				3366	180	605
C_1	64	0.0027	-36	1544	-56	236	-13
Total					**3197**		**1010**

At the balanced point the calculated capacities are:

N_{ub} = 3197 kN $\quad\quad$ M_{ub} = 1010 kNm

Applying the capacity reduction factor (∅) as required by AS 3600 and Table 3.7.1 gives the following design capacities depending on the column effective length and the dead to live load ratio.

Table 4.1.3.2 Design Capacities at the balanced point for each of the two factors

Design Value	∅ N_{ub} (kN)	∅ M_{ub} (kNm)
∅ = 0.65	2078	657
∅ = 0.6	1918	606

4.1.4 Pure bending point

At the pure bending point, as the name suggests, the column has no applied axial load. To find this point the strains and the resultant forces are calculated for various values of k_u. An iterative process is undertaken to find the value of k_u such that the resultant force is zero. This is most efficiently achieved using a simple piece of software code or spreadsheet. For this example an Excel spreadsheet using the "Goal Seek" option was utilised to determine the k_u value to be 0.1902 giving a neutral axis depth of 102 mm.

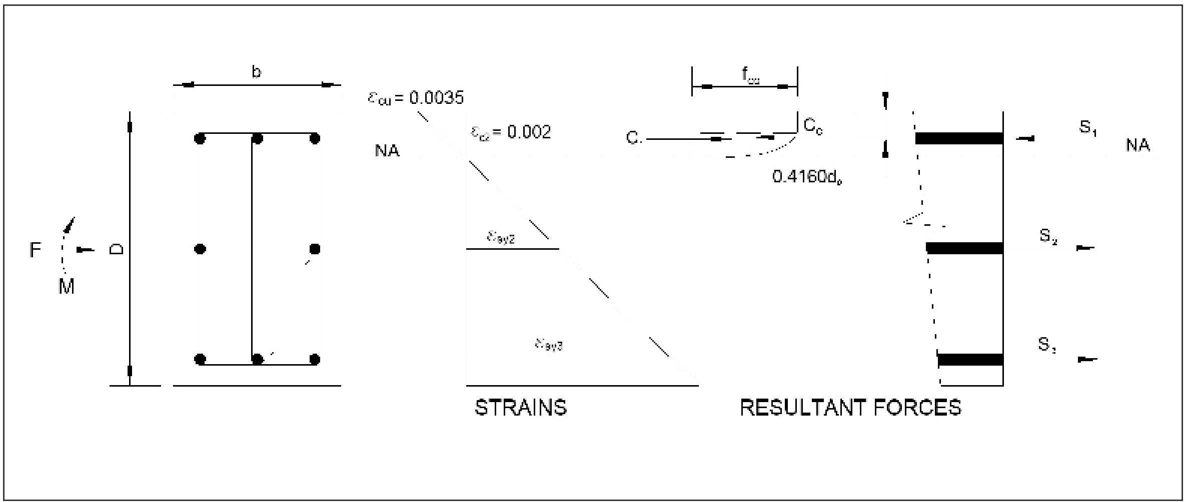


Figure 4.1.4.1 – Strains and resultant concrete stresses at the pure bending point

Table 4.1.4.1 provides the details of the stresses, strains and forces and confirms that when the resultant forces are summed the net value is zero and the moment is 609 kNm. Note that the values in Table 4.1.4.1 take into account the double counting that would otherwise occur because the reinforcing bars in reinforcing layer 1 have displaced concrete and thus reduces the resultant concrete compressive force, C_c. An adjustment can be made by subtracting C1 corresponding to the displaced concrete and appropriate f'_c at that layer.

Table 4.1.4.1 Resultant forces at the Pure Bending point

Force	x_i (mm)	ε_i	σ_i or $f_{co.i}$ (MPa)	Ai (mm^2)	Fi (kN)	D/2-x_i (mm^2)	M_i (kNm)
S_1	64	0.0013	262	1544	405	236	96
S_2	300	-0.0068	-600	1029	-618	0	0
S_3	536	-0.0149	-600	1544	-926	-236	219
C_c	43				1189	258	306
C_1	64	0.0013	-31	1544	-49	236	-12
				Total	**0**		**609**

At the pure bending point the calculated capacities are:

N_{ub} = 0 kN; M_{ub} = 609 kNm

Applying the ∅ value as required by AS 3600 and Table 3.7.1 it is noted that $N_u < N_{ub}$ so

$$\varnothing = \varnothing_0 + [(\varnothing' - \varnothing_0)(1 - N_u / N_{ub})]$$

Where

$\varnothing' = 1.24 - 13k_{u0}/12$ where $0.65 \le \varnothing' \le 0.85$ and k_{uo} is for Pure bending

$= 1.24 - 13 * 0.1902 / 12$

$= 1.03$

$\therefore \varnothing' = 0.85$ since $0.65 \le \varnothing' \le 0.85$

Therefore

$$\varnothing = \varnothing_0 + [(\varnothing' - \varnothing_0)(1 - N_u / N_{ub})]$$

$$= \varnothing_0 + [(0.85 - \varnothing_0)(1 - 0/3197)]$$

$$= 0.85$$

The ∅ value of 0.85 is the expected result, however the calculation steps have been included to demonstrate how the ∅ value varies linearly from 0.85 to $\varnothing_0$ relative to the axial load from the pure bending point to the balanced point.

At the pure bending point the design moment is –

$$\varnothing M_{uo} = 0.85 \times 609$$

$$= 518\text{kNm}$$

4.1.5 Axial Load – Moment interaction curve

The four key points calculated in the preceding sections are plotted and labelled on the Axial Load – Moment Interaction curve shown in Figure 4.1.5.1. The points between the decompression point and the pure bending point have been generated using the same spreadsheet used to calculate the four key points by including varying values of k_u from D/d_0 down to 0.1902, the value that gave the pure bending point.

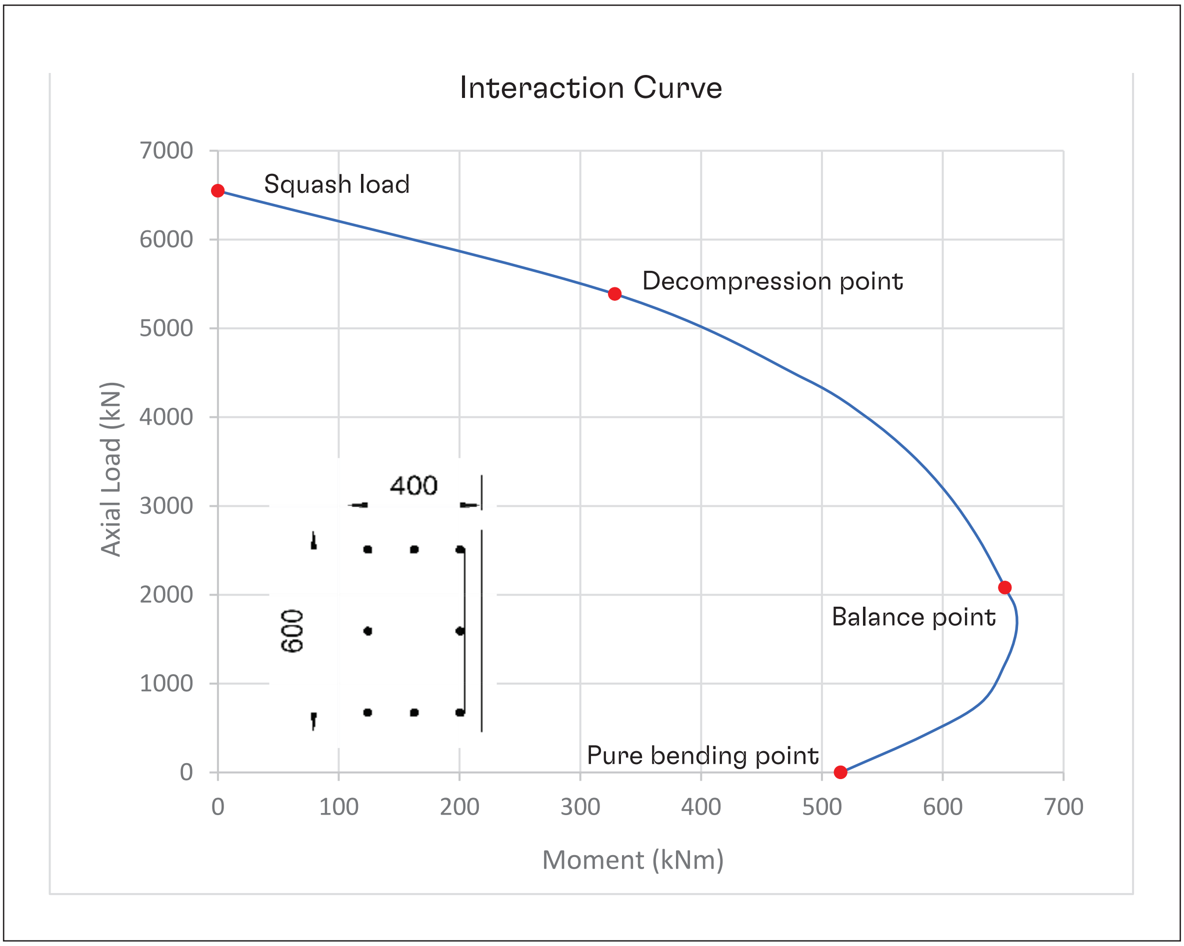


Figure 4.1.5.1 – Axial force - Moment Interaction curve using a parabolic rectangular strain for a section with a capacity reduction factor of 0.65

4.2 Circular Column

The second example considers a circular column 800 mm in diameter shown in Figure 4.2.1.

D = *800 mm*

Reinforcing steel

Longitudinal - 8 S29 (600 MPa)

Fitments - S 11@300 (600 MPa)

$f'c$ = 65 MPa;

Cover = 40 mm

d_0 = 800 – 40 – 11 – 29/2 = 536mm

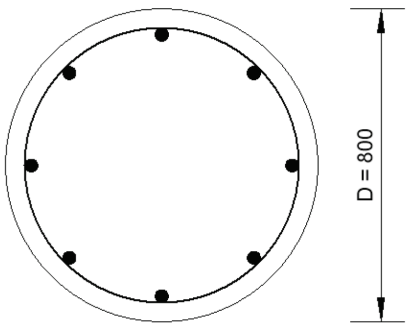


Figure 4.2.1 – Design example 1 – Circular column cross section

This example differs from the first in a significant way. The circular shape means the cross-section width varies so a simple expression for the concrete compressive force and the location of the result is more difficult to derive. It should also be noted for concrete strengths over 50 MPa the stress-strain relationship while still rectangular-parabolic is not of degree 2. For a 65 MPa concrete equation– Eq. 2.2.1.4 and Eq 2.2.1.5 gives

$$\sigma_c = 58.5 \left[1 - \left(1 - \frac{\varepsilon_c}{.0024} \right)^{1.49} \right] \qquad \text{(Eq. 4.2.1)}$$

The expression for the resultant concrete stresses derived in Example 1 only applies to parabolas of degree 2 and therefore not applicable for 65 MPa concrete where n = 1.49 and not 2. Appendix B provides the general formula for each of the standard concrete strengths in AS 3600.

The corresponding stress-strain curve for 65 MPa concrete is shown in Figure 4.2.2.

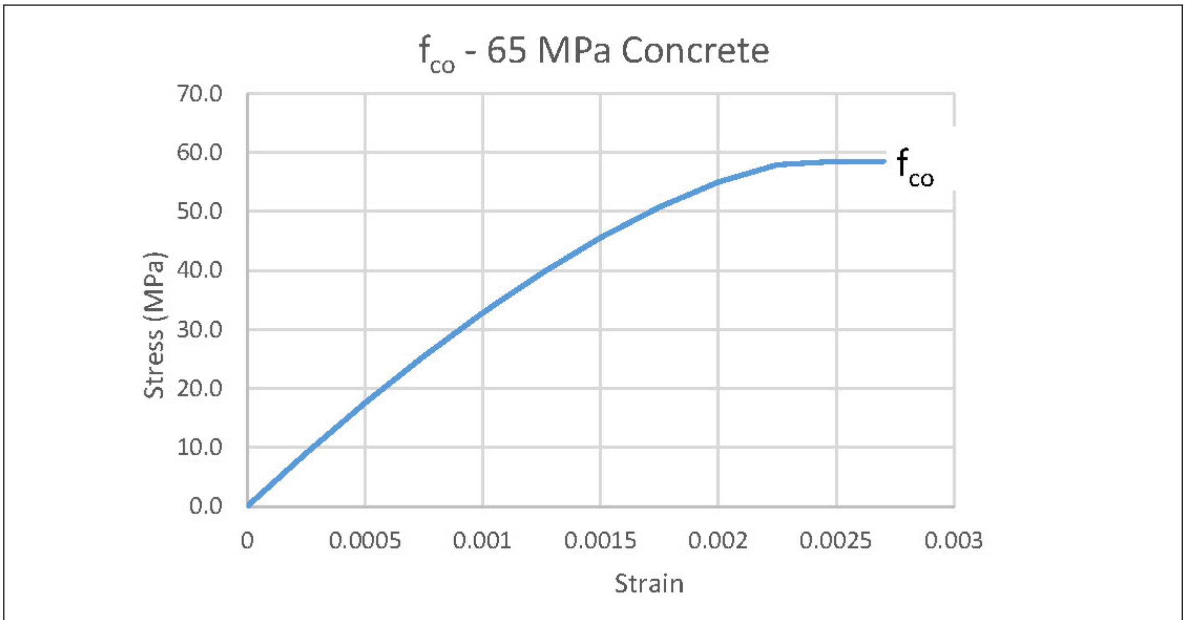


Figure 4.2.2 – Stress-strain curve for 65 MPa concrete

For this example, 4 points which form the interaction curve are calculated:

1) Squash load N_{uo}

2) Decompression point M_{uD}, N_{uD}

3) Balanced point M_{ub}, N_{ub}

4) Pure Bending point M_{uo}

4.2.1 Squash load

Use Eq 3.1(g) to calculate the squash load.

$$N_{u0} = \alpha_1 \times f'_C \times A_C + A_S \times \sigma_S \quad \textit{where}$$

$f'_C = 65$

Where α_1 using Eq 3.1(b) is:

$\alpha_1 : 0.72 \leq 1.0 - 0.003 \times 65 \leq 0.85$

$\alpha_1 = 0.8050$

$A_S = 8 \times \frac{\pi \times 29.2^2}{4} = 5357 mm^2$

$A_C = \pi \times \frac{800^2}{4} - 5357 = 497{,}298\ mm^2$

$\sigma_S = 500$ MPa

Substituting gives:

$$\begin{aligned} N_{u0} &= 0.8050 \times 65 \times 497{,}298 + 5357 \times 500 \\ &= 28{,}699{,}741\ N \\ &= 28{,}700\ kN \end{aligned}$$

Applying the capacity reduction factor (∅) as required by AS 3600 and Table 3.7.1 gives the following design capacities dependaant on the column effective length and the dead to live load ratio.

Table 4.2.1 Design capacities for each of the two phi factors

Design Value	∅ N_{ub} (kN)	∅ M_{ub} (kNm)
∅ = 0.65	18,660	0
∅ = 0.6	17,220	0

4.2.2 Decompression point

At the decompression point the distance to the neutral axis is 800 mm. For a concrete with f'_c = 65 MPa, from Table 2.2.1 and the equation from 2.2.1, ε_{c2} = 0.0024, ε_{cu} = 0.0027 and f_{dc}= 58.5 MPa. Using this information, the strains and concrete stresses at the decompression point can be represented as shown in Figure 4.2.2.1

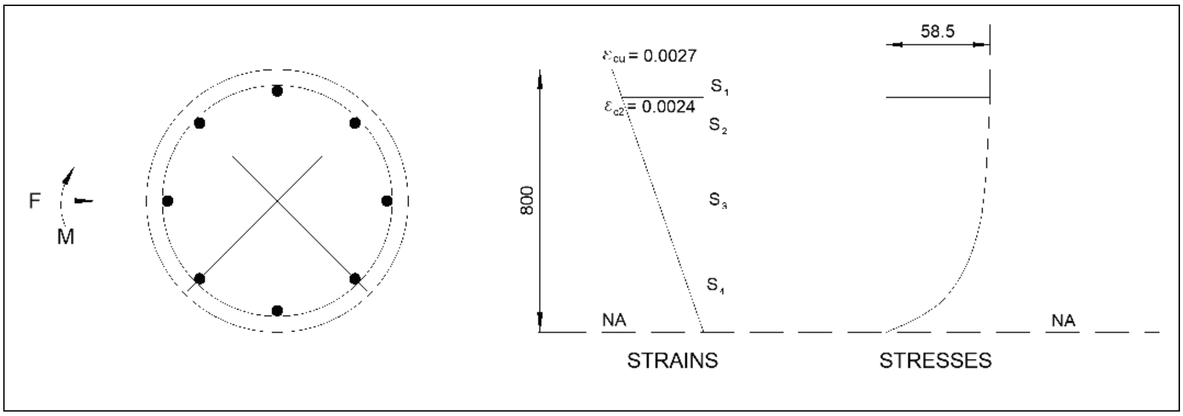


Figure 4.2.2.1 – Strains and concrete stresses at the decompression point

In Example 1, it was relatively simple to derive a general expression resolving the rectangular-parabolic stress distribution and multiplying by the width to determine the resulting force acting on the rectangular section. However, for a circular section where there is not a constant width, it is easier to determine the stress x area by numerical integration. In this Example, the area to be integrated, is divided into 10 strips and the midpoint rule using the concrete stress value at the midpoint of the strip as shown in Figure 4.2.2.2 is used. This midpoint method of numerical integration was chosen for its simplicity in demonstrating the calculation process in this example. Other methods such as Simpson's rule with more strips will give more accurate values.

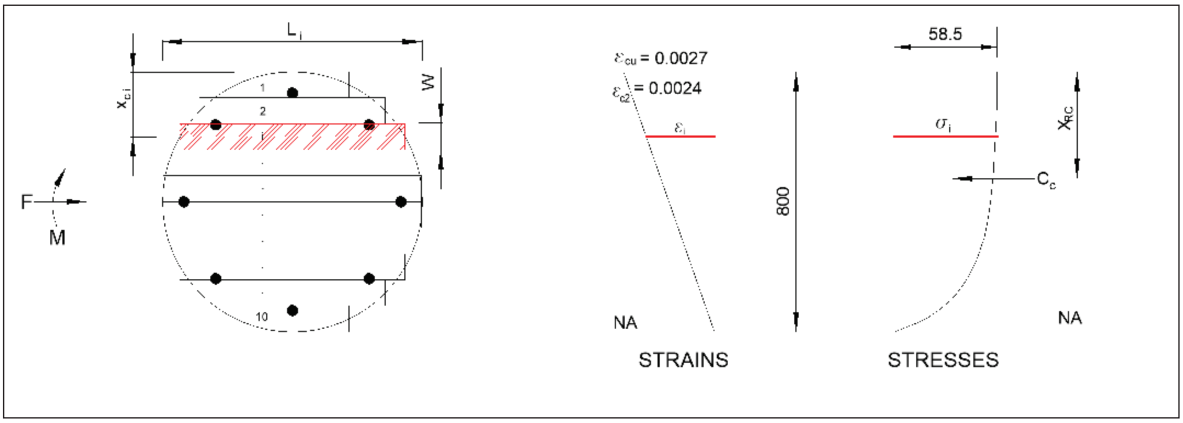


Figure 4.2.2.2 – Numerical integration to determine resultant concrete compression force

The resultant concrete compression force is given by

$$Cc = \sum_{i=1}^{10} L_i \times W \times \sigma_i$$

Where $L_i = \sqrt{4 \times X_{ci} \times (D - X_{ci})}$

At the decompression point:

$$k_u = 1.089 \rightarrow D = 800\text{mm}$$

Using 10 intervals

$$W = 800 \div 10$$

$$= 80\text{mm}$$

The numerical integration presented in Table 4.2.2.1 is from a spreadsheet using the formulae from Section 2.2.1 to determine the concrete properties, rather than reading and extrapolating Table 2.1. This was considered easier and more accurate.

The concrete strain is determined using similar triangles as shown in Figure 4.2.2.1 and the stress at each point, , is calculated using Eq. 2.2.1.5a

$$\sigma_c = f_{co}\,[1 - (1 - \frac{\varepsilon_c}{\varepsilon_{c2}})^n]$$

From Table 2.1: $f_{co} = 58.5$

$\varepsilon_{c2} = 0.0024$

$n = 1.49$

σ_c could also be interpolated from Table 2, however it is generally easier and more accurate to use Eq. 2.2.1.5a.

Table 4.2.2.1 Numerical integration to determine the concrete compression force at the decompression point							
i	x_{ci} (mm)	L_i (mm)	Area (mm^2)	Strain -	Stress (MPa)	C_i (kN)	C_i*x_{ci} (kNmm)
1	40	348.7	27897	0.002600	58.5	1632	65279
2	120	571.3	45705	0.002326	58.4	2670	320351
3	200	692.8	55426	0.002053	55.7	3089	617831
4	280	763.2	61052	0.001779	51.3	3132	877059
5	360	796.0	63679	0.001505	45.7	2909	1047145
6	440	796.0	63679	0.001232	39.1	2488	1094788
7	520	763.2	61052	0.000958	31.6	1931	1003958
8	600	692.8	55426	0.000684	23.4	1298	778793
9	680	571.3	45705	0.000411	14.5	664	451406
10	760	348.7	27897	0.000137	5.0	139	105840
					Totals	**19952**	**6362449**

From the numerical integration table, the resultant force (C) is 19952 kN

The location of the resultant force is given by

$$X_{RC} = \frac{\sum_{n=1}^{10} C_i x_{ci}}{\sum_{n=1}^{10} C_i}$$

$$= \frac{6362449}{19952}$$

$$= 319\text{mm}$$

The resultant concrete compression force and steel forces are shown in Figure 4.2.2.3

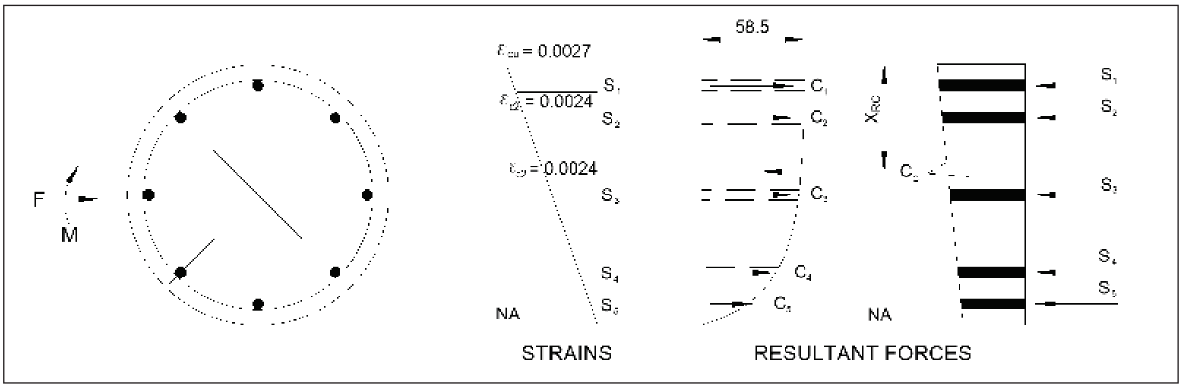


Figure 4.2.2.3 – Resultant forces at the decompression point

The resultant forces shown in Figure 4.2.2.3 are summed to determine the axial capacity and moments are taken about the plastic centroid to determine the moment capacity. The results are presented in Table 4.2.2.2 with the convention that compressive forces are positive. The Table entries C_1, C_2, C_3 C_4 and C_5 are included to account for the void in the concrete where the reinforcing bars are located and thus reduces the calculated concrete compressive force, C_c, to better model the section.

Table 4.2.2.2 Resultant forces and moments at decompression point							
Force	x_i (mm)	ε_i	σ_i or $f_{co.i}$ (MPa)	A_i (mm²)	F_i (kN)	$D/2-x_i$ (mm²)	M_i (kNm)
S1	66	0.0025	502	670	336	334	113
S2	164	0.0022	435	1339	583	236	138
S3	400	0.0014	274	1339	367	0	0
S4	637	0.0006	112	1339	149	-237	-35
S5	734	0.0002	45	670	30	-334	-10
Cc	319	0.0016			19952	81	1618
C1	66	0.0025	59	670	-39	334	-13
C2	164	0.0022	57	1339	-77	236	-18
C3	400	0.0014	42	1339	-57	0	0
C4	637	0.0006	19	1339	-26	-237	6
C5	734	0.0002	8	670	-5	-334	2
				Total	**21213**		**1800**

At the Decompression point the calculated capacities are:

N_{uD} = 21213 kN; M_{uD} = 1800 kNm

Applying the capacity reduction factor, as required by AS 3600 and Table 3.7.1 gives the following design capacities depending on the column effective length and the dead to live load ratio.

Table 4.2.2.3 Design Capacities for each of the two factors		
Design Value	$\varnothing N_{uD}$ (kN)	$\varnothing M_{uD}$ (kNm)
$\varnothing$ = 0.65	13790	1170
$\varnothing$ = 0.6	12730	1080

4.2.3 Balanced point

At the balanced point Eq. 3.3(a) provides the value of $k_u = k_{ub}$, where

$$\frac{k_{ub} = \varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_s}$$

And 3.3(b) gives $\varepsilon_s = E_s / f_{sy}$

For a 600 MPa Steel

$$\varepsilon_s = \frac{2 \times 10^5}{600} = 0.003$$

For concrete with f'_c = 65 MPa, ε_{cu} = 0.0027 (from Table 2.1)

$$k_{ub} = \frac{0.0027}{0.0027 + 0.003} = 0.4737$$

The depth to the neutral axis is

$$d_b = k_{ub} \times d$$

$$= 0.4737 \times 734$$

$$= 348\text{mm}$$

At the balanced point the value of, and the distance to the neutral axis is 348 mm. For a concrete with f = 65 MPa, from equations in 2.2.1, ε_{c2} = 0.0024, ε_{cu} = 0.0027 and 58.5 MPa. Using this information, the strains and concrete stresses at the decompression point can be represented as shown in Figure 4.2.3.1.

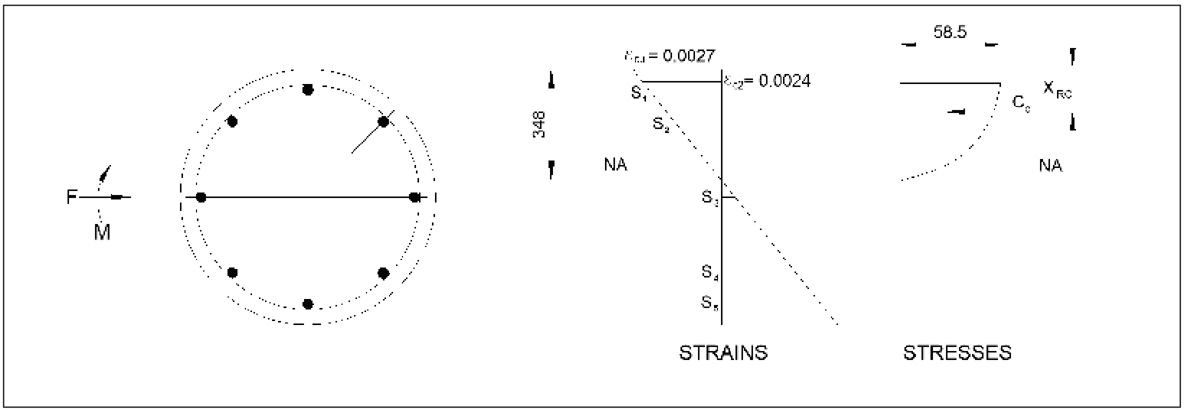


Figure 4.2.3.1 – Strains and concrete stresses at the balanced point

The concrete compression force, C_c can be found for the balanced point using the same method of numerical integration used for the decompression point. Table 4.2.3.1 contains the data for the numerical integration to determine C_c at the balanced point.

Table 4.2.3.1 Numerical integration to determine the concrete compression force at the balanced point

i	x_{ci} (mm)	L_i (mm)	Area (mm^2)	Strain -	Stress (MPa)	C_i (kN)	C_i*xci (kNmm)
1	17.4	233.3	8118	0.002600	58.5	475	8260
2	52.2	395.1	13744	0.002326	58.4	803	41892
3	87.0	498.0	17326	0.002053	55.7	966	83985
4	121.8	574.7	19994	0.001779	51.3	1026	124904
5	156.5	634.8	22082	0.001505	45.7	1009	157905
6	191.3	682.5	23744	0.001232	39.1	928	177511
7	226.1	720.5	25064	0.000958	31.6	793	179226
8	260.9	750.1	26094	0.000684	23.4	611	159439
9	295.7	772.3	26868	0.000411	14.5	390	115392
10	330.5	787.8	27407	0.000137	5.0	137	45217
Totals						**7136**	**1093730**

From the numerical integration table, the resultant force (C) is 7136 kN

The location of the resultant force is given by

$$X_{RC} = \frac{\sum_{n=1}^{10} C_i x_{ci}}{\sum_{n=1}^{10} C_i}$$

$$= \frac{1093730}{7136}$$

$$= 153 \text{ mm}$$

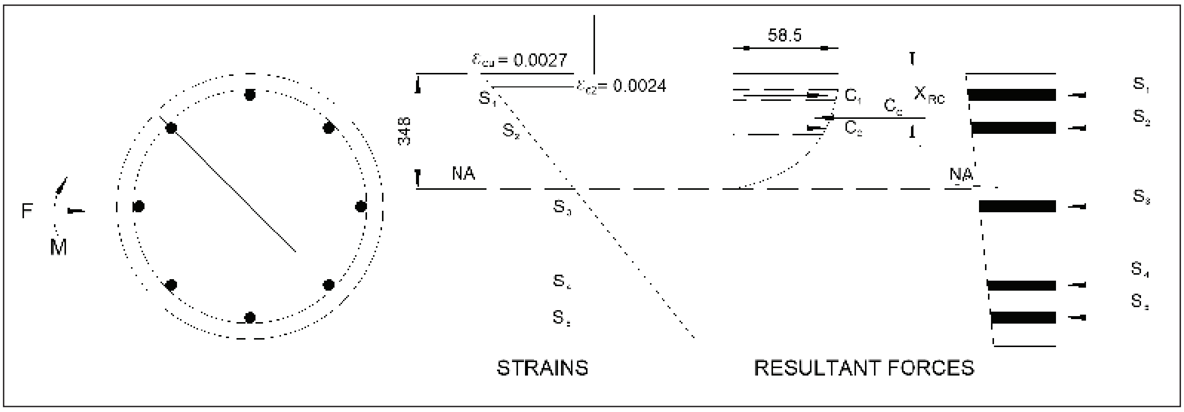


Figure 4.2.3.2 – Resultant forces at the balanced point

The resultant forces shown in Figure 4.2.3.2 are summed to determine the axial capacity and moments are taken about the plastic centroid of the section to determine the corresponding flexual capacity. The results are presented in Table 4.2.3.2

Table 4.2.3.2 Resultant forces and moments at balanced point

Force	x_i (mm)	ε_i	σ_i or $f_{co.i}$ (MPa)	A_i (mm²)	F_i (kN)	$D/2-x_i$ (mm²)	M_i (kNm)
S1	66	0.0022	444	670	297	334	99
S2	164	0.0015	290	1339	388	236	92
S3	400	-0.0004	-82	1339	-110	0	0
S4	637	-0.0023	-455	1339	-609	-237	144
S5	734	-0.0030	-600	670	-402	-334	134
Cc	153				7136	247	1761
C1	66	0.0022	58	670	-39	334	-13
C2	164	0.0015	44	1339	-59	236	-14
Total					**6604**		**2204**

At the balanced point the calculated capacities are:

N_{ub} = 6604 kN; M_{ub} = 2204 kNm

Applying the capacity reduction factor (∅) as required by AS 3600 and Table 3.7.1 gives the following design capacities depending on the column effective length and the dead to live load ratio.

Table 4.2.3.3 Design Capacities for each of the two phi factors

Design Value	∅ N_{ul} (kN)	∅ M_{ul} (kNm)
∅ = 0.65	4292	1432
∅ = 0.6	3962	1322

4.2.4 Pure bending

At the pure bending point, the applied axial load is zero. An iterative process is undertaken to find the value of k_u such that the resultant force of the concrete and steel is zero. This is most efficiently achieved using a simple piece of software code or spreadsheet. For this particular example an Excel spreadsheet using the "Goal Seek" option was utilised to determine the k_u value to be 0.1913 giving a neutral axis depth of 140.5 mm as shown in Figure 4.2.3.1.

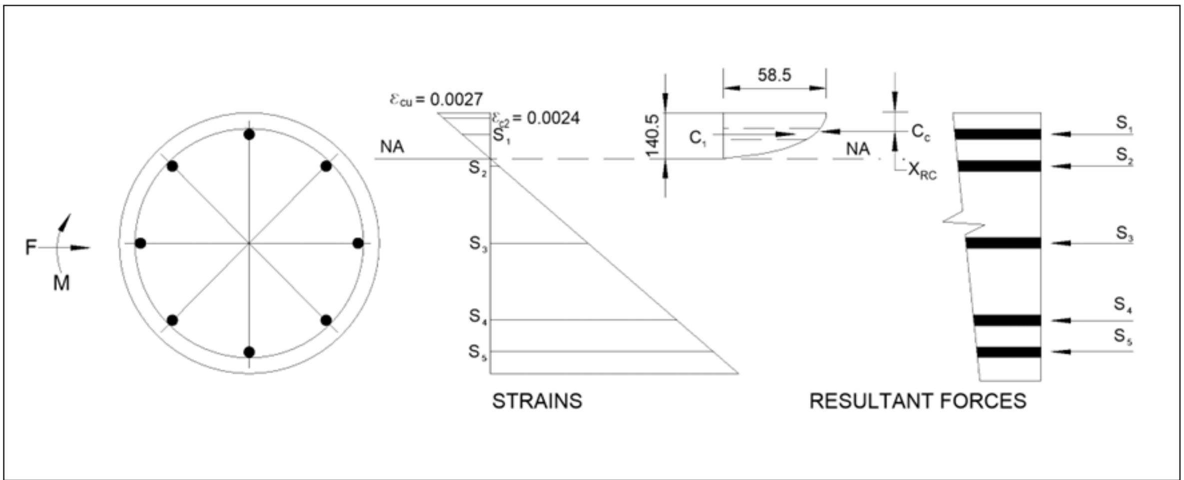


Figure 4.2.3.1 – Resultant forces at the pure bending point

The numerical integration for a neutral axis at 140.5 mm is presented in Table 4.2.3.1 and the sum of the resultant forces and moments about the plastic centroid arc is presented in Table 4.2.4.2.

Table 4.2.3.1 Numerical integration to determine the concrete compression force at the pure bending point

i	x_{ci} (mm)	L_i (mm)	Area (mm^2)	Strain	Stress (MPa)	C_i (kN)	$C_i{*}\,X_{ci}$ (kNm)
1	7.0	149.2	2097	0.002600	58.5	123	861
2	21.1	256.2	3600	0.002326	58.4	210	4427
3	35.1	327.7	4605	0.002053	55.7	257	9006
4	49.2	384.2	5399	0.001779	51.3	277	13607
5	63.2	431.6	6064	0.001505	45.7	277	17495
6	77.2	472.5	6640	0.001232	39.1	259	20028
7	91.3	508.7	7148	0.000958	31.6	226	20622
8	105.3	541.0	7602	0.000684	23.4	178	18739
9	119.4	570.1	8010	0.000411	14.5	116	13880
10	133.4	596.4	8380	0.000137	5.0	42	5578
Total						**1964**	**124243**

From the numerical integration table, the resultant force (C) is 1965 kN

The location of the resultant force is given by

$$X_{RC} = \frac{\sum_{n=1}^{10} C_i x_{ci}}{\sum_{n=1}^{10} C_i}$$

$$= \frac{124243}{1964}$$

$$= 63.3 \text{ mm}$$

Table 4.2.4.2 Resultant forces and moments at pure bending point

Force	x_i (mm)	ε_i	σ_i or $f_{co.i}$ (MPa)	A_i (mm^2)	F_i (kN)	$D/2-x_i$ (mm^2)	M_i (kNm)
S1	66	0.0015	292	670	195	334	65
S2	164	-0.0005	-90	1339	-121	236	-29
S3	400	-0.0051	-600	1339	-804	0	0
S4	637	-0.0097	-600	1339	-804	-237	190
S5	734	-0.0116	-600	670	-402	-334	134
Cc	63				1964	337	661
C1	66	0.0015	45	670	-30	334	-10
Total					**0**		**1013**

At the pure bending point the calculated capacities are:

$N_{uo} = 0$ kN; $M_{uo} = 1013$ kNm

Applying the capacity reduction factor 0.85 required by AS 3600 and Table 3.7.1.

$$\varnothing M_{uo} = 0.85 \times 1013$$

$$= 861 \text{ kNm}$$

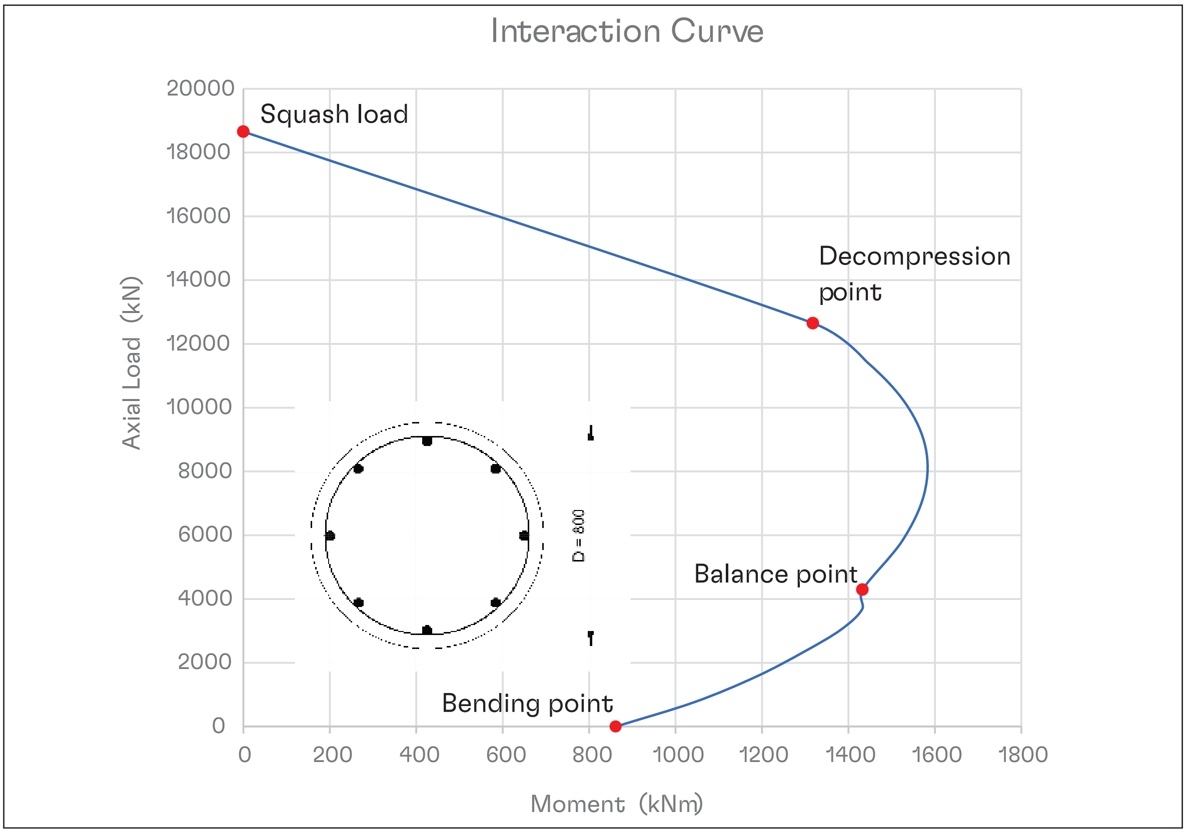


Figure 4.2.4.2 – Axial Force-Moment Interaction curve using parabolic rectangular strain for a section with a capacity reduction factor of 0.65

4.2.5 Axial Load – Moment interaction curve

The four key points calculated in the preceding sections are plotted and labelled on the Axial Force – Moment Interaction curve shown in Figure 4.2.4.2. A straight line joins the squash load and the decompression point. The points between the decompression point and the pure bending point have been generated using the same spreadsheet used to calculate the four key points by including varying values of k_u from 1 down to 0.1896, the k_u value corresponding to the pure bending point.

5. Extension to AS 3600

The previous sections of this guide calculated the squash load for a column using the same process outlined in AS 3600 for the rectangular stress block method. However, Note 2 of Clause 10.6.1 confirms that designers can consider the effect of confinement which potentially offers higher values for the squash load of a column. This section of the design guide provides commentary on how concrete confinement can be modelled in a column to derive a more accurate and potentially higher, squash load value. Two worked examples are included to demonstrate how the model is applied and when it is effective to apply.

The other aspect covered in this section is the concept of minimum reinforcement in a column. The minimum longitudinal reinforcement rule in AS3600: 2018 Clause 10.7.1 (a), requiring a minimum area of 1 per cent of the concrete cross section ($0.01A_g$) can be traced back to at least AS 1480: 1982 when the yield strength of reinforcing steel was just 410 MPa. Given yield strengths for columns can be as high as 600 MPa consideration should be given to adjusting this value to be in line with the higher strengths of reinforcing bars available.

5.1 Effect of Confinement

Concrete confinement modifies the concrete stress-strain relationship allowing higher critical strains to be achieved. In Section 3.1 it was noted that the AS 3600 rectangular stress block model, under concentric loading of the column, set the maximum strain limit for the steel to be 0.0025. This limit is set on the basis of ensuring compatibility of the steel strain to the concrete strain limit of 0.0025. This strain limit on the steel in turn limits the stress in the 600 MPa steel to below its specified characteristic yield strength and hence the full potential of the 600 MPa steel is not realised. If the limiting strain in the concrete is increased by confinement, a higher proportion of the potential 600 MPa of the steel can be realised.

Application of AS 3600 Clause 3.1.4 permits the use of recognized simplified equations to determine the stress-strain relationship. Application of the stress-strain equations in the *fib* Model Code, including those related to confining of concrete would meet the requirements of this clause and is confirmed by its reference in the AS 3600 Commentary (Standards Australia 2022). The *fib* Model Code states that if the confined concrete properties are exploited in terms of calculations, consideration shall be given to the spalling of the concrete cover and premature buckling of the longitudinal reinforcement.

If sufficient fitments were provided to confine the concrete such that the concrete strain could reach the squash load could be determined using the following equation

$$N_{u0.c} = \alpha_{1.c} \times f'_{c.c} \times A_{c.c} + A_s \times \min(E_s \times \varepsilon_{c2.c}, f_{sy}) \qquad \text{(Eq 5.1(a))}$$

Where

$\alpha_{1.c}$ = in situ strength factor for concrete (0.9).

$f'_{c.c}$ = confined characteristic concrete strength.

$A_{c.c}$ = area of concrete core; that is, it excludes the cover concrete area.

A_s = area of the steel reinforcement.

E_s = Elastic modulus of steel.

$\varepsilon_{c2.c}$ = ultimate strain in the confined concrete.

f_{sy} = yield stress of longitudinal bars.

The AS 3600 Commentary explains that the in the equation Eq. 3.1(g) includes two components: "the in situ strength factor for the concrete (taken as 0.9) and a factor to account for spalling of the cover concrete as the applied axial load nears ultimate". However, in Eq. 5.1(a) it is assumed that all the cover concrete has spalled leaving just the core concrete area as denoted by $A_{c.c}$ and therefore $\alpha_{1.c}$ can take the value of 0.9.

The values of $f'_{c.c}$ and $\varepsilon_{cu.c}$ can be determined using guidance provided by the *fib* Model Code.

$$f'_{c.c} = f'_c \left[1 + 3.5 \left(\frac{\sigma_2}{f'_c}\right)^{3/4}\right] \quad \text{(Eq. 5.1(b))}$$

$$\varepsilon_{c2.c} = \varepsilon_{cu} \left[1 + 5 \left(\frac{f'_{c.c}}{f'_c} - 1\right)\right] \quad \text{(Eq. 5.1(c))}$$

$$\varepsilon_{cu.c} = \varepsilon_{cu2} + 0.2\, \sigma_2 / f'_c \quad \text{(Eq. 5.1(d))}$$

Where

σ_2 (= σ_3) is the effective lateral compressive stress at the ultimate limit state due to confinement and

ε_{c2} and $\varepsilon_{cu.2}$ are from Table 2.1

AS 3600 Clause 10.7.3.3 provides guidance to calculate the core confinement by a simplified method which is analogous to those in the *fib* Model Code. The confining pressure (referred to as in the *fib* Model Code) can be calculated using the following expression.

$$f_{r.eff} = k_e f_r \quad \text{(Eq. 5.1 (e))}$$

Where

k_e = an effectiveness factor accounting for the arrangement of fitments - refer to Figure 5.1.1.

f_r = average confining pressure on the core cross section taken at the level of the fitments

The average confining pressure on the core at the level of the fitments shall be calculated as follows:

$$\frac{\sum^m A_{b.fit}\, f_{sy} \sin\theta}{d_s s} \quad \text{(Eq 5.1 (f))}$$

Where

$A_{b.fit}$	= cross sectional area of one leg of fitment
f_{sy}	= yield stress of the fitment
θ	= angle between the fitment leg and the confinement plane
m	= number of fitment legs crossing the fitment plane
d_s	= overall dimension measured between centre-lines of the outermost fitments
s	= centre to centre spacing of fitments along the column

AS 3600: 2018 Figure 10.7.3.3 provides details on calculating confining pressures on circular, square and rectangular cross sections.

The effectiveness factor is calculated as follows

(a) For rectangular sections

$$k_e = \left(1 - \frac{s}{2b_c}\right)\left(1 - \frac{s}{2d_c}\right)\left(1 - \frac{nw^2}{6b_c d_c}\right) \qquad \text{(Eq 5.1 (g))}$$

Where

n = number of laterally restrained longitudinal bars

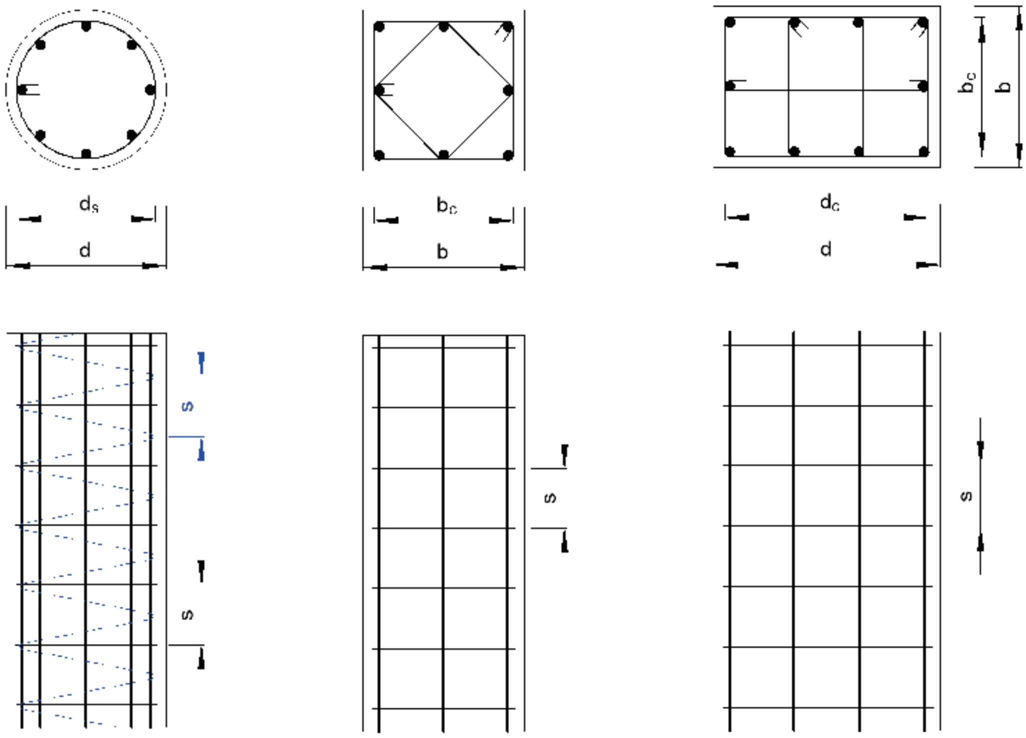
w = average clear spacing between adjacent longitudinal bars

b_c = core dimension measured between the centrelines of the outermost fitments across the width of the section (refer figure 5.1.1)

d_c = core dimension measured between the centrelines of the outermost fitments across the depth of the section (refer figure 5.1.1)

(b) For circular sections

$$k_e = \left(1 - \frac{s}{d_s}\right)^2 \qquad \text{(Eq.5.1(h))}$$



$$k_e = \left(1 - \frac{s}{2d_s}\right)^2$$

$$f_r = \frac{2A_{b.fit} f_{sy.f}}{d_s s}$$

$$k_e = \left(1 - \frac{s}{2a_c}\right)^2 \left(1 - \frac{nw^2}{6a_c^2}\right)$$

$$f_r = \frac{2A_{b.fit} f_{sy.f}(1 + \sin\theta)}{b_c s}$$

$$k_e = \left(1 - \frac{s}{2b_c}\right)\left(1 - \frac{s}{2d_c}\right)\left(1 - \frac{nw^2}{6b_c d_c}\right)$$

$$f_r = min\left(\frac{\sum A_{b.fit} f_{sy.f}}{b_c s}, \frac{\sum A_{b.fit} f_{sy.f}}{d_c s}\right)$$

(a) Circular section (spiral fitment shown dotted)

(c) Square section (with 2 square fitments)

(d) Rectangular sections

Figure 5.1.1 f_r and k_e expressions for circular, square and rectangular column sections

AS 3600 Clause 10.7.3.3 offers and alternative calculation for calculating the confining pressure for rectangular and circular columns.

$$f_{r.eff} = 0.5k_e \rho_s f_{sy.f} \qquad \text{(Eq. 5.1 (h))}$$

Where

$$\rho_s = \frac{A_{b.fit}\ x\ total\ perimeter\ of\ fitments\ crossing\ the\ section}{A_c\ x\ s} \qquad \text{(Eq. 5.1 (i))}$$

$A_c = b_c\ x\ d_s$

5.2 Example of rectangular column

This example examines the rectangular column in Section 4.1 and compares the two options offered in AS 3600 Clause 10.7.3.3 to calculate the confining pressure provided by the fitments. These values are then utilised to assess the squash load of the column considering the effects of confinement.

5.2.1 Calculate effective confining pressure using method 1

B = 400 mm
D = 600 mm
Reinforcing steel
Longitudinal - 8S26 (600 MPa)
Fitments - S11@300 (600 MPa)
f'_c = 40 MPa;
Cover = 40 mm

$$d_s = 600 - 2 \times 40 - 11 = 509 \text{ mm}$$

$$b_c = 400 - 2 \times 40 - 11 = 309 \text{ mm}$$

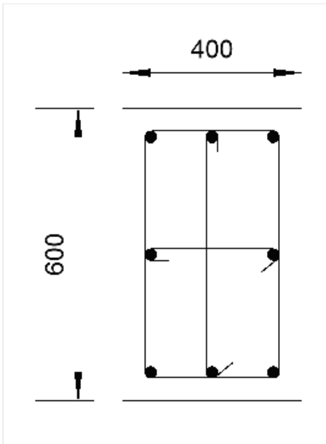


Figure 5.2.1 – Design Example 3: Rectangular column cross-section

Calculate confining pressure in each direction to find minimum value using Eq. 5.1(e)

(a) Across the depth of the column

$$f_{r.d} = \frac{\sum^{n} A_{b.fit}\, f_{sy} \sin\theta}{d_s s}$$

$$= 3 \times \frac{\frac{\pi \times 11^2}{4} \times 600\sin 90}{509 \times 300}$$

$$= 1.120$$

(b) Across the width of the column

$$f_{r.b} = \frac{\sum_1^m A_{b.fit} f_{sy} \sin\theta}{b_s s}$$

$$= 3 \times \frac{\frac{\pi \times 11^2}{4} \times 600\sin90}{309 \times 300}$$

$$= 1.845$$

min (fr.d ,fr.b) $= \min(1.120, 1.845)$

$= 1.120$

Calculate the effectiveness factor of the fitments using Eq. 5.1 (g)

$$k_e = \left(1 - \frac{s}{2b_c}\right)\left(1 - \frac{s}{2d_c}\right)\left(1 - \frac{nw^2}{6b_c d_c}\right)$$

s = 300mm

n = 8

w_x = [400 - (2 x 40) - (2 x 11) - (3 x 25.6)] /2

= 110.6mm

w_y = [600 - (2 x 40) - (2 x 11) - (3 x 25.6)] / 2

= 210.6mm

w = [(4 x 110.6) + (4 x 210.6)] / 8

= 160.6mm

$$k_e = \left(1 - \frac{300}{2 \times 309}\right)\left(1 - \frac{300}{2 \times 509}\right)\left(1 - \frac{8 \times 160.6^2}{6 \times 309 \times 509}\right)$$

$$= 0.2836$$

Calculate the effective confining pressure using Eq. 5.1 (d)

$$f_{r.eff} = k_e f_r$$

$= 0.2836 \times 1.120$

$= 0.318$

5.2.2 Calculate effective confining pressure using method 2 for rectangular column sections

Calculate the volumetric ratio of fitments to concrete using Eq. 5.1 (i)

$$\rho_s = \frac{A_{b.fit} \text{ x } \textit{total perimeter of fitments crossing the section}}{A_c \text{ x } s}$$

$$= \frac{\frac{\pi \text{ x } 11^2}{4} \text{X } (509 + 309 + 509 + 309 + 509 + 309)}{509 \text{ x } 309 \text{ x } 300}$$

$$= 0.0049$$

Calculate the effective confining pressure using Eq. 5.1 (h)

$$f_{r.eff} = 0.5 k_e \rho_s f_{sy.f}$$

Where

$$k_e = 0.2836 \qquad \text{(from section 5.2.1)}$$

$$f_{r.eff} = 0.5 \text{ x } 0.2826 \text{ x } 0.0049 \text{ x } 600$$

$$= 0.417$$

This 0.417 value compares with the lower value of 0.318 determined using method 1. In this case, the lower value can be attributed to the lower confining pressure across the depth of the column compared with that across the width of the column. Method 1 takes the lower of the two values while Method 2 effectively takes the average of the two values. For a symmetrical column the two values should be similar. For consistency with the *fib* model method the more conservative of value 0.3175 derived using the simplified calculation from Section 5.2.1 will be adopted to calculate the parameters for design utilising confined concrete.

5.2.3 Confined characteristic concrete strength properties

Calculate confined characteristic concrete strength using Eq. 5.1(b)

$$f'_{c.c} = f'_c \left[1 + 3.5 \left(\frac{\sigma_2}{f'_c}\right)^{3/4}\right]$$

Where $\sigma_2 = f_{r.eff}$

$$f'_{c.c} = 40 \left[1 + 3.5 \left(\frac{0.3175}{40}\right)^{3/4}\right]$$

$$= 43.7 \text{ MPa}$$

Calculate the strain at peak stress of the confined concrete using Eq. 5.1(c)

$$\varepsilon_{c2.c} = \varepsilon_{cu} \left[1 + 5 \left(\frac{f'_{c.c}}{f'_c} - 1\right)\right]$$

Where

$$\varepsilon_{cu} = 0.002 \text{ from Table 2.1 for 40 MPa concrete}$$

$$\varepsilon_{c2.c} = 0.002 \left[1 + 5 \left(\frac{43.7}{40} - 1\right)\right]$$

$$= 0.002925$$

5.2.4 Squash load considering confined concrete

Calculate the squash load using Eq. 5.1(a)

$$N_{u0.c} = \alpha_{1.c} \times f'_{c.c} \times A_{c.c} + A_s \times \min(E_s \times \varepsilon_{c2.c}, f_{sy})$$

$$As = 8 \times \frac{(\pi \times 25.6^2)}{4}$$

$$= 4118\text{mm}^2$$

$$N_{u0.c} = 0.9 \times 43.7 \times (309 \times 509 - 4118) + 4118 \times (2 \times 10^5 \times 0.002925)$$

$$= 8434\ kN$$

$$\varnothing N_{u0.c} = 0.65 \times 8434$$

$$= 5482\ kN$$

The 5482 kN considering the impact of concrete confinement compares with 6551 kN for the squash load calculation to AS 3600. The explanation for the lower value is that when the load in the concrete causes it to exceed the concrete's peak stress capacity, the concrete cover spalls. While the fitments are able to confine the concrete core, they are not able to confine the concrete cover. For larger columns the area of the concrete core is proportionally larger than the concrete cover so the increase in strength of the concrete core may exceed the reduction in strength due to the loss of the cover. It should also be noted that in this example the concrete confinement allowed the concrete strain to reach 0.0029 which in turn allowed the steel to reach a stress of 580 MPa. Closer fitment spacing does allow a higher concrete stress, but the maximum squash load is still less than the AS 3600 value in this case.

5.3 Example of large circular column

This example considers a large circular column to illustrate how the additional capacity of the confined concrete and the corresponding ultimate strain at peak load allowing the longitudinal reinforcement to reach it yield strength will more than compensate the reduction in capacity due to the loss of the concrete cover.

D = 1200 mm

Reinforcing steel

Longitudinal -20S33 (600 MPa)

Fitments- S18@300 (600 MPa)

Concrete

f'_c = 65 MPa;

Cover = 40 mm

$$d_s = 1200 - 2 \times 40 - 18 = 1102 \text{ mm}$$

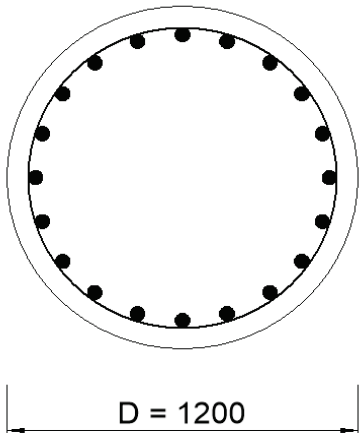


Figure 5.2.2 – Design Example 4: Circular column

5.3.1 Calculate squash load using AS 3600

Calculate the squash load to AS 3600 using Eq. 3.1(g).

$$Nuo = (\alpha_1 \times f'_c \times A_c) + (A_s \times \sigma_s)$$

Where

$$f'_c = 65 \text{ MPa}$$

α_1 using Eq 3.1(b) is:

$$\alpha_1 : 0.72 \leq 1.0 - 0.003 \times 65 \leq 0.85$$

$$\alpha_1 = 0.805$$

$$As = 20 \times \frac{\pi \times 32.9^2}{4} = 17{,}002 \ mm^2$$

$$Ac = \frac{\pi \times 1200^2}{4} - 17{,}002 = 1{,}113{,}971 \ mm^2$$

σ_s = 500 MPa (limited by concrete strain)

Substituting gives:

$$N_{u0} = (0.805 \times 65 \times 1{,}113{,}971) + (17{,}002 \times 500)$$

$$= 66{,}789{,}533 \text{ N}$$

$$= 66{,}790 \text{ kN}$$

$$\emptyset N_{u0} = 0.65 \times 66{,}790 \text{ kN}$$

$$= 43{,}410 \text{kN}$$

5.3.2 Calculate effective confining pressure method 1

Calculate confining pressure using the equation from Fig 5.1.1 (a)

$$f_r = \frac{2A_{b\text{-}fit} f_{sy}}{d_s s}$$

$$f_r = \frac{2 \times \frac{\pi \times 18.3^2}{4} \times 600}{1102 \times 300}$$

$$= 0.9550$$

Calculate the effectiveness factor of the fitments using Eq. 5.1.1 (g)

$$k_e = \left(1 - \frac{s}{2d_s}\right)^2$$

$$k_e = \left(1 - \frac{300}{2 \times 1102}\right)^2$$

$$= 0.7463$$

Calculate the effective confining pressure using Eq. 5.1 (d)

$$f_{r.eff} = k_e f_r$$

$$= 0.7463 \times 0.9550$$

$$= 0.713$$

5.3.3 Confined characteristic concrete strength properties

Calculate confined characteristic concrete strength using Eq. 5.1(a)

$$f'_{c.c} = f'_c \left[1 + 3.5 \left(\frac{\sigma_2}{f'_c}\right)^{3/4}\right]$$

Where $\sigma_2 = f_{r.eff}$

$$f'_{c.c} = f'_c\ 65 \left[1 + 3.5 \left(\frac{0.7086}{65}\right)^{3/4}\right]$$

$$= 72.7 \text{ MPa}$$

Calculate the strain at peak stress of the confined concrete using Eq. 5.1(b)

$$\varepsilon_{c2.c} = f'_c \left[1 + 5 \left(\frac{72.7}{65} - 1\right)\right]$$

ε_{cu} = 0.0024 from Table 2.1 for 40 MPa concrete

$$\varepsilon_{c2.c} = 0.0024 \left[1 + 5 \left(\frac{72.7}{65} - 1\right)\right]$$

$$= 0.0038$$

5.3.4 Squash load considering confined concrete

Calculate the squash load using Eq. 5.1(a)

$N_{u0.c} = \alpha_{1.c} \times f'_{c.c} \times A_{c.c} + A_s \times min\,(E_s \times \varepsilon_{c2.c'} f_{sy})$

Where

$As = 20 \times \dfrac{\pi \times 32.9^2}{4} = 17002\ mm^2$

$Ac.c = \dfrac{\pi \times 1102^2}{4} - 17002 = 936269\ mm^2$

$N_{u0.c} = 0.9 \times 72.7 \times 936269 + 17002 \times 600$

$= 71468314\ N$

$= 71468\ kN$

$\varnothing N_{u0} = 0.65 \times 71468$

$= 46550$ kN

The squash load considering confined concrete of 46 550 kN compares with the lower value of 43 410 when confinement of the concrete is ignored. This example demonstrates for large columns, the reduction in capacity with the loss of cover associated with the confined concrete model is more than compensated by the additional concrete stress in the core and the additional strain allowing the full utilisation of the 600 MPa steel.

5.4 Minimum reinforcement requirements

AS 3600: 2018 Clause 10.7.1 relates to the minimum cross-sectional area of longitudinal reinforcing steel which limits the full utilisation of 600 MPa steels. This clause requires 1 per cent reinforcing steel based on the gross cross-sectional area of the column except it may be reduced, if the column has a larger area than required for strength to 0.1 $\varnothing N/f_{sy}$. The minimum value of 1 per cent has been in the reinforced concrete Standard as far back as AS 1480: 1982 when the typical grades of steel were 410 MPa. The AS 3600 Commentary (Standards Australia 2022) states that this is to provide resistance to accidental bending and to provide some restraint to creep and shrinkage to avoid yielding of the reinforcement. Higher strength steels typically allow lower quantities to be used, so minimum requirements need to be examined to ensure they do not unnecessarily limit the full utilisation of the higher strength steel.

Both moment capacity in the column to resist bending and yielding of the reinforcement due to creep and shrinkage are directly related to the yield strength of the steel. If the yield strength of the steel is increased from 410 MPa to 600 MPa it is reasonable to argue that the minimum area of reinforcing steel can be reduce by the same proportion such that the force capacity of the reinforcing bars remains unchanged; that is, from 0.01Ag to 0.0068Ag.

A proposal to reduce the minimum reinforcement requirement by over 30% may seem extreme but it is reasonable if examined rationally. It is likely that this clause in the Standard has simply not been reviewed and updated in the last 40 years as the AS 3600 Commentary indicates the figure should be related to the yield strength of the steel. The standard yield strengths since AS 1480: 1982 has been raised twice, firstly to 500 MPa and now to 600 MPa with no changes to this minimum value or to make it a function of the yield strength. The *fib* Model Code (2013) Clause 7.13.5.4 requires the minimum longitudinal reinforcement to be 0.2%. Furthermore, for large columns it permits designers to just consider the column as a concrete tube having 200 mm thick walls for the purpose of calculating the concrete area.

Designers may wish to exercise their engineering judgement based on the preceding information and adopt a value for the minimum longitudinal reinforcement in a concrete column of $A_{cs \cdot min}$ as the larger of

(a) $0.01 \times 500 / f_{sy}$

(b) $0.15N^* / f_{sy}$

This value strikes the balanced as it is less conservative than the current requirements of AS3600: 2018 and more conservative than the *fib* Model Code. The proposed minimum value is also a function of the yield strength, so it is consistent with the rationale to provide bending capacity for unintended loads and yielding capacity for creep and shrinkage. Designs using 600 MPa steels can initially adopt 0.83% as the minimum value for determining the axial force – moment diagram and then do a simple check to ensure $0.15N^* < A_{st} f_{sy}$

6. References

- *fib* 2013, *fib Model Code for Concrete Structures 2010*, The International Federation for Structural Concrete, Ernst & Sohn, Lausanne, Switzerland.
- Ng, A. and McGregor, G. 2021, 'Changes to Australian Standards to Better Utilise Higher Strength Reinforcing Steels'. *Proceedings of Concrete Institute of Australia's Biennial National Conference 2021,* Virtual Conference, September 2021.
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7. Appendix A – SENSE 600® properties

SENSE 600® reinforcing steels are produced to AS/NZS 4671 in diameters that provide an equivalent design load capacity to standard diameter 500 MPa reinforcing steel bars.

Table A7.1 provides details of the diameters and areas of SENSE 600® reinforcing bars.

Table A7.1 SENSE 600® Equivalent Force capacity bars

600 MPa			500 MPa	Load Capacity
Designation	Dia (mm)	Area (mm²)	Dia (mm)	kN
S11	11.0	94	12	57
S15	14.6	168	16	101
S18	18.3	262	20	158
S22	21.9	377	24	227
S26	25.6	513	28	308
S30	29.2	670	32	403
S33	32.9	848	36	509
S37	36.5	1050	40	629

8. Appendix B – Closed-form solutions for parabolic stress distributions

For designers undertaking manual calculations with a calculator for a rectangular column, closed form solutions such as those used in Example 1 are more convenient to use than numerical integration. This Appendix provides the closed form solutions for each of the characteristic concrete strengths in Table 2.1 to calculate the concrete compressive force and its location per unit width.

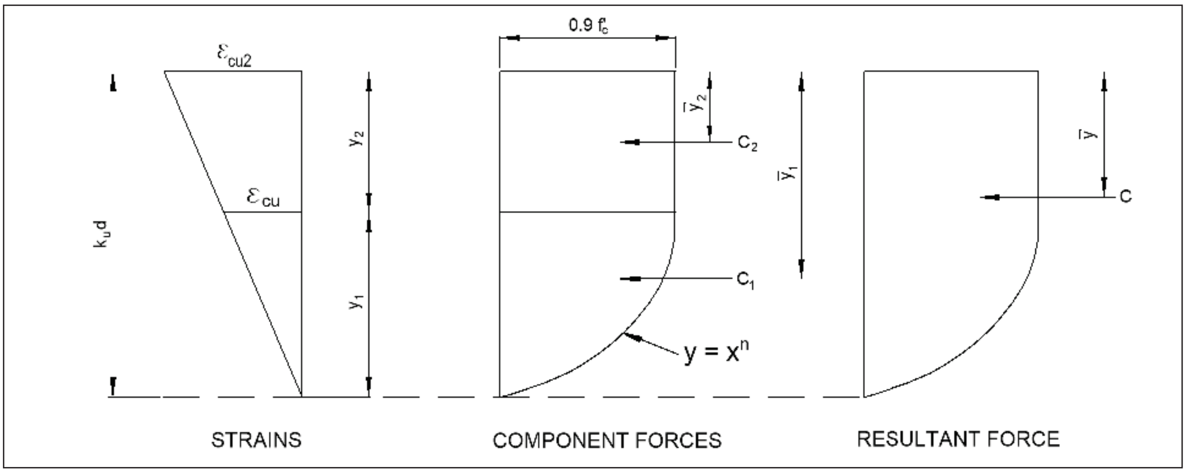

Figure B1 – Resultant forces per unit width for rectangular parabolic stress distributions

Table B1 – Closed form solutions for the concrete compressive force per unit width

n	2	1.49	1.40
$y_1 = \frac{\varepsilon_{cu}}{\varepsilon_{cu2}} k_u d$	$0.5714 k_u d$	$0.8613 k_u d$	$0.9662 k_u d$
$y_2 = \left(1 - \frac{\varepsilon_{cu}}{\varepsilon_{cu2}}\right) k_u d$	$0.4286 k_u d$	$0.1387 k_u d$	$0.0338 k_u d$
$C_1 = 0.9 f'_c \frac{\varepsilon_{cu}}{\varepsilon_{cu2}} \left(1 - \frac{1}{n+1}\right) k_u d$	$0.3810 k_u d 0.9 f'_c$	$0.5156 k_u d 0.9 f'_c$	$0.5640 k_u d 0.9 f'_c$
$C_2 = 0.9 f'_c \left(1 - \frac{\varepsilon_{cu}}{\varepsilon_{cu2}}\right) k_u d$	$0.4286 k_u d 0.9 f'_c$	$0.1387 k_u d 0.9 f'_c$	$0.0338 k_u d 0.9 f'_c$
$\bar{y}_1 = \left[1 - \frac{1}{2} \frac{\varepsilon_{cu}}{\varepsilon_{cu2}} \left(\frac{1 - \frac{1}{2n+1}}{1 - \frac{1}{n+1}} \right)\right] k_u d$	$0.6429 k_u d$	$0.4460\ k_u d$	$0.3749\ k_u d$
$\bar{y}_2 = \frac{1}{2}\left(1 - \frac{\varepsilon_{cu}}{\varepsilon_{cu2}}\right) k_u d$	$0.2143 k_u d$	$0.0694 k_u d$	$0.0169 k_u d$
$C = C_1 + C_2$	$0.8095 k_u d 0.9 f'_c$	$0.6543 k_u d 0.9 f'_c$	$0.5978 k_u d 0.9 f'_c$
$\bar{y} = \frac{C_1 \bar{y}_1 + C_2 \bar{y}_2}{C_1 + C_2}$	$0.4160\ k_u d$	$0.3622\ k_u d$	$0.3547\ k_u d$

It just makes SENSE