The curvilinear stress block – A design model to decarbonize reinforced concrete columns using 600 MPa steels

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ABSTRACT

AS 3600 - Concrete structures, provides clear and prescriptive guidance on how to implement the rectangular stress block method to produce an axial load-moment interaction curve for the design of columns in combined compression and bending. When this method is applied to the design of reinforced columns with 600 MPa steels, the reduction in steel cross sectional area compared to 500 MPa steels can be minimal. Intuitively, it would be expected that the higher grade of steel would result in a lower cross-sectional area of steel required to resist the same design load.

Alternative methods using a curvilinear stress-strain relationship for the concrete is permitted by AS 3600, but the Standard provides significantly less guidance on how this method should be implemented. A method referencing the fib Model Code and the Euronorms, to utilise a curvilinear stress block which arguably better models a reinforced concrete column, is proposed. This model recognises the additional strength available in 600 MPa reinforcing steels and requires a reduced cross-sectional area of steel compared to designs using 500 MPa reinforcing steel.

Design examples presented show how 600 MPa steels in reinforced concrete columns require a lower cross-sectional area of steel to provide the same design capacity using the curvilinear stress block model compared to the rectangular stress block model. The examples also quantify the decarbonization resulting from using 600 MPa steels compared to 500 MPa steels when designing reinforced concrete columns using this method.

1. INTRODUCTION

The Australian construction industry is burdened with generating almost 20% of Australia's carbon footprint (Yu, et.al. 2016). These statistics and the tangible effects of climate change are driving the construction industry to become increasingly focused on how to design and construct more sustainable buildings through decarbonization of materials.

Initially, the construction industry's push for decarbonization focused on reducing the carbon emission from the operation of the building because this was proportionally the largest contributor. Therefore, owners, developers and builders looked to the architects, mechanical and electrical engineers to design systems that reduced carbon emissions during the building's operation. The significant improvements in the operation area have now shifted the focus for decarbonization to the embodied carbon within a structure. With that shift, there is an increasing demand for structural engineers to play their part by reducing the structures embodied carbon (Arup 2022).

Manufacturers of construction materials, understanding the market's desire to decarbonize, are continually looking for opportunities to manufacture products which have less embodied carbon or allow a structure to be constructed with less embodied carbon. InfraBuild, the only Australian manufacturer of reinforcing steels has conducted extensive research and development to produce higher strength grades of steel which are designed to reduce the mass of steel used in a structure and hence the embodied carbon.

Changes to Australian Standards, which now recognise these higher strength grades, will facilitate adoption in design and construction using these steels. Intennis elbow 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 (RC) 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 most other elements.

Intuitively, a RC element that is governed in design by strength rather than serviceability is likely to benefit the most from a higher strength steel. Given the design of a reinforced column is generally governed by strength it was expected that an increase in the strength from 500 MPa by 20% to 600 MPa would result in a reduction in the cross-sectional area of steel of approximately 16.7% and hence a comparative decarbonization of the steel in the structure. However, this may not always be the case and may be dependent on the design method chosen by the engineer.

Australian design standards usually offer engineers 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 lower tier, and the curvilinear stress block method a higher tier.

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 or curvilinear stress block method. This paper examines the two tiers of design and its impact on the decarbonization. It proposes a method adopted from the *fib* Model Code and the Euronorms, to utilise a curvilinear stress block which arguably better models a RC column, particularly if is not rectangular in section (Jenkins 2011). A building designed and constructed using 500 MPa reinforcing steel was re-designed with 600 MPa steels using the proposed curvilinear stress block method as an example to quantify the decarbonization when using the higher grade of steel.

2. AS 3600 DESIGN PROVISIONS

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

(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 curvilinear stress-strain relationship required by the Standard is "defined by recognised 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

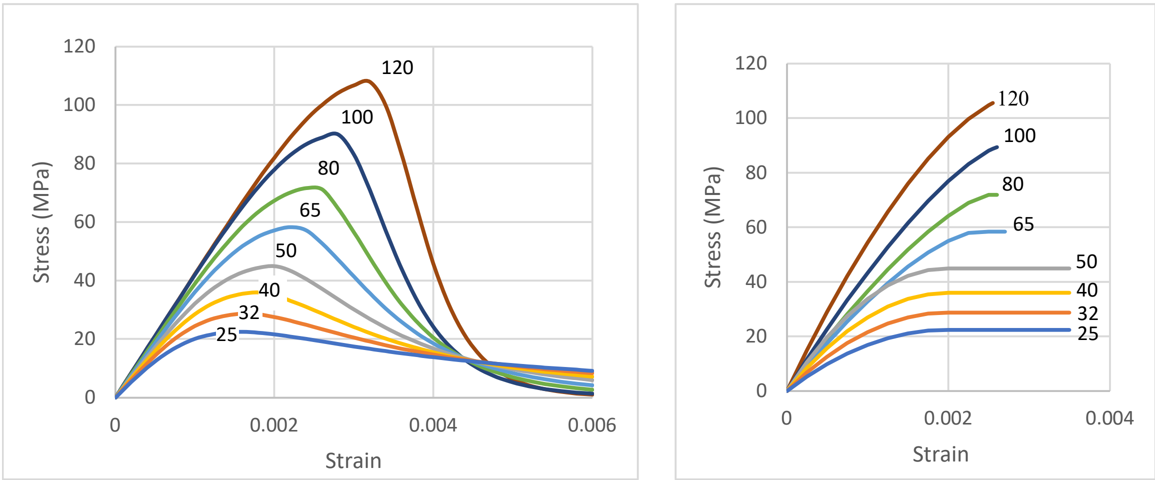
The curvilinear stress-strain relationship is a higher tier model than that of the rectangular stress block, and arguably offers a higher degree of accuracy, and better utilisation of the additional capacity offered by 600 MPa steels [Ng and McGregor 2021]. This approach offers better utilisation because 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 "recognised simplified equations" or the properties "determined from test data". The AS 3600 – *Concrete structures - Commentary (Supplement 1)* [Standards Australia 2022], (Clause C8.1.2), suggests two options:

1. A curvilinear model for both the ascending and descending branches of the compressive stress-strain relationship, then 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 (refer AS 3600 Commentary C3.1.4).
2. A curvilinear stress block model with a curvilinear ascending branch followed by a constant value of stress equal to the peak stress (refer to Figure 2.1b) with the extreme fibre compressive strain dependant on the concrete strength given by the formula-

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

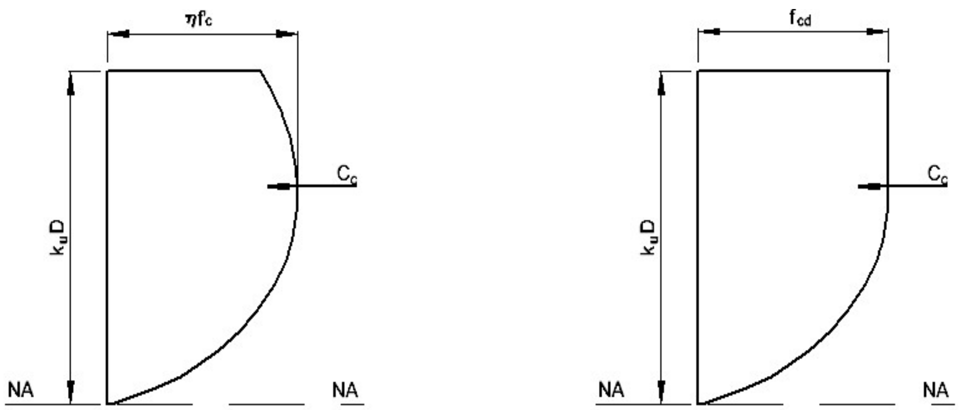
Figure 2.1 shows a plot of the stress-strain relationship for the curvilinear model (curvilinear ascending and descending branches) and the curvilinear stress block model (curvilinear ascending branch only).



(a) Curvilinear Model (b) Curvilinear Stress Block Model

Figure 2.1 Concrete stress-strain relationships

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



(a) Curvilinear Stress Distribution (b) Curvilinear Stress Block Model

Figure 2.2 The stress distribution in the cross section of a column subjected to a moment and an axial force using simplified recognised concrete stress strain relationships

2.2 Curvilinear stress block model

The curvilinear stress distribution 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 curvilinear stress block model (or parabolic-rectangular stress block model) is the model commonly adopted by designers because it matches very closely the curvilinear stress distribution and is simple to use because the resultant force is easily calculated as the sum of the forces from the rectangular and parabolic stress blocks acting through their combined centroid.

2.2.1 Design stress-strain curves

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

$\varepsilon_{c2} = 0.002$ for $f_{ck} \leq 50$ MPa (Eq. 2.2.1.1a)

$\varepsilon_{c2} = 0.002 + 0.000085(f_{ck} - 50)^{0.53} \leq 0.0035$ for $f_{ck} > 50$ MPa (Eq. 2.2.1.1b)

$\varepsilon_{cu} = 0.0026 + 0.035\left(\frac{90-f_{ck}}{100}\right)^4 \leq 0.0035$ for $f_{ck} \leq 90$ MPa (Eq. 2.2.1.2a)

$\varepsilon_{cu} = 0.0026$ for $f_{ck} > 90$ MPa (Eq. 2.2.1.2b)

$n = 2$ for $f_{ck} \leq 50$ MPa (Eq. 2.2.1.3a)

$n = 1.4 + 2.34\left(\frac{90-f_{ck}}{100}\right)^4$ for $50 < f_{ck} \leq 90$ MPa (Eq. 2.2.1.3b)

$n = 1.4$ for $f_{ck} > 90$ MPa (Eq. 2.2.1.3c)

$f_{cd} = \propto_{cc} f_{ck}/\gamma_c$ (Eq. 2.2.1.4)

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

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

Where:

f_{ck} – is the characteristic compressive strength of concrete, which is referred to as f'_c in AS 3600.

f_{cd} – is the design compressive strength

$\propto_{cc}$ – is a coefficient taking account of the long-term effects on the compressive strength and the unfavourable effects from the way the load is applied. The *fib* Model Code suggests that for normal design situations it may be assumed that the increase in the compressive strength after 28 days compensates for the effect of sustained loading, so that $\propto_{cc}$ = 1.0 for new structures. However, AS 3600 Clause 3.1.4 requires that, for design purposes, the shape of the in situ uniaxial compressive stress-strain curve shall be modified so that the maximum stress is $0.9f'_c$. For consistency with AS 3600 it is recommended that $\propto_{cc}$ = 0.9 be adopted.

γ_c – is the partial safety factor for concrete, being 1.5 for transient and persistent situations and 1.2 for incidental situations. Australian Standards, including AS 3600, use a capacity reduction factor (∅) rather than this partial safety factor. For consistency with AS 3600, f_{cd} be calculated using a γ_c equal to 1.0 and the final calculated capacity reduced by ∅.

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

Concrete Strength (MPa)	$f'_c = f_{ck}$ (MPa)							
	25	32	40	50	65	80	100	120
ε_{c2}	0.002	0.002	0.002	0.002	0.0024	0.0025	0.0027	0.0028
ε_{cu}	0.0035	0.0035	0.0035	0.0035	0.0027	0.0026	0.0026	0.0026
n	2.0	2.0	2.0	2.0	1.49	1.40	1.40	1.40
f_{dc}	23	29	36	45	59	72	90	108
ε_c	σ_c							
0.00000	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.00050	9.8	12.6	15.8	19.7	17.5	19.2	22.7	25.9
0.00100	16.9	21.6	27.0	33.8	32.8	36.6	43.3	49.7
0.00150	21.1	27.0	33.8	42.2	45.6	51.8	61.6	70.9
0.00200	22.5	28.8	36.0	45.0	55.0	64.2	76.9	89.1
0.00250	22.5	28.8	36.0	45.0	58.5	71.9	88.0	103.1
0.00260	22.5	28.8	36.0	45.0	58.5	72.0	89.4	105.2
0.00280	22.5	28.8	36.0	45.0				
0.00300	22.5	28.8	36.0	45.0				
0.00320	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				

Table 2.1 above was generated using the formulae from the *fib* Model Code for the common concrete grades available in Australia.

3. STRENGTH OF CROSS-SECTION MODELLED BY A CURVILINEAR 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 axial load-moment interaction curve. A similar approach can be used with the proposed curvilinear stress block.

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

1. Squash load point (SLP)
2. Decompression point (DP)
3. Balanced point (BP)
4. Pure bending point (PBP)

Arguably, the simplest way to apply the curvilinear 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 in AS 3600 and substitute the curvilinear stress block for the rectangular stress block.

3.1 Squash load

Confinement of the concrete provided by fitments modifies the stress-strain relationship by increasing the allowable strain. However, for simplicity this is conservatively omitted and the methodology AS 3600 prescribes for the rectangular stress block is also applied to the curvilinear stress block.

$$N_{\mathrm{u0}} = (\propto_1 \times f'_{\mathrm{c}} \times A_{\mathrm{c}}) + (A_s \times \sigma_s) \qquad \text{(Eq. 3.1 (a))}$$

Where:

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

σ_s = steel stress with a maximum strain in the reinforcement (ε_s) of 0.0025

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

$$E = {}^{\sigma_s}\!/_{\varepsilon_s} \qquad \text{(Eq. 3.1(c))}$$

Rearranging gives $\sigma_s = E \times \varepsilon_s$ (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, the maximum strain limit of 0.0025 for the reinforcing steel limits the capacity of a 600 MPa bar to 500 MPa.

Thus

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

3.2 Decompression point

The decompression point is calculated taking the strain in the extreme compression fibre equal to the ultimate concrete strain ($\mathcal{E}_{\mathrm{cu}}$) 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 $\varepsilon_{\mathrm{s}} = 0.003$.

The neutral axis depth at the balanced point is $k_{\mathrm{ub}}d$ 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 $\varepsilon_s = E/f_{sy}$ (Eq. 3.3(b))

3.4 Pure bending point

The pure bending point occurs when the sum of the compressive forces in the steel and concrete is equal to the sum of the tensile forces in the steel. To find this point an iterative process varying the $k_u$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 not overly conservative to adopt the same approach for the curvilinear stress block, however, if the calculations are produced using software or a spreadsheet then it is just as easy to set k_u 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

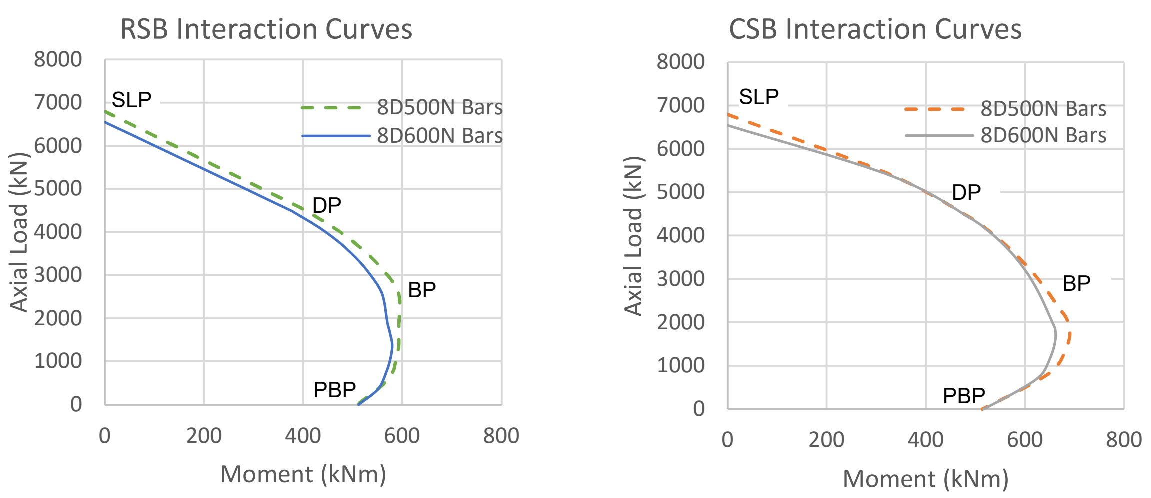
Where the neutral axis lies within the cross section the maximum strain shall be taken as that given in Table 2.1. The values along this part of the curve can be determined by varying the value of k_u from the value of k_u determined at the pure bending point up to the decompression point where $k_u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

4. RECTANGULAR STRESS BLOCK vs CURVILINEAR STRESS BLOCK

Comparing the rectangular stress block (RSB) method with the proposed curvilinear stress block (CSB) method the primary difference is the ultimate allowable strain. While the RSB method limits the ultimate concrete stain under compression and bending to just 0.003, the CSB method allows up to 0.0035. Depending on the geometry of the column, the additional 16.7% in concrete strain permitted by the CSB method allows the additional strength available in a 600 MPa bar (compared with a 500 MPa bar) to be utilised.

4.1 Example of RSB Method vs CSB Method

Consider a 400 x 600 mm column reinforced with 8 D500N28 bars. Each bar has a cross-sectional area of 616 mm^2 and a load capacity of 308 kN, In the same column reinforced with 8 D600N25.6 bars, each bar has a cross-sectional area of 515 mm^2, but the same load capacity of 308 kN. Even though the bars have the same load capacity, when the axial load-moment interaction curves are constructed, the diagrams are not identical. Figure 4.1a shows the results comparing the RSB Method and Figure 4.2b shows the results comparing the CSB method.



(a) (b)

Figure 4.1 Rectangular Stress Block (RSB) vs Curvilinear Stress Block Interaction Curves (CSB)

The comparison in Figure 4.1 shows that the CSB method results in a closer correspondence of the interaction curves for 600 MPa and 500 MPa steels of the same load capacity used to reinforce a concrete column. While it can be demonstrated (Ng and McGregor 2021) that the difference in the interaction diagram between the two steel grades are greater for small columns compared with large columns regardless of either method, the difference is more pronounced using the RSB method.

5.0 REDESIGN EXAMPLE

A design example is presented to illustrate the potential decarbonization by utilising 600 MPa steels in RC columns. The proposed curvilinear method was used to redesign all the columns in the building with the resulting quantities of steel and embodied carbon compared with those from the original 500 MPa design. The 19-storey building which was the subject of the example is located in Parramatta, NSW and was originally designed by Hyder Consulting.

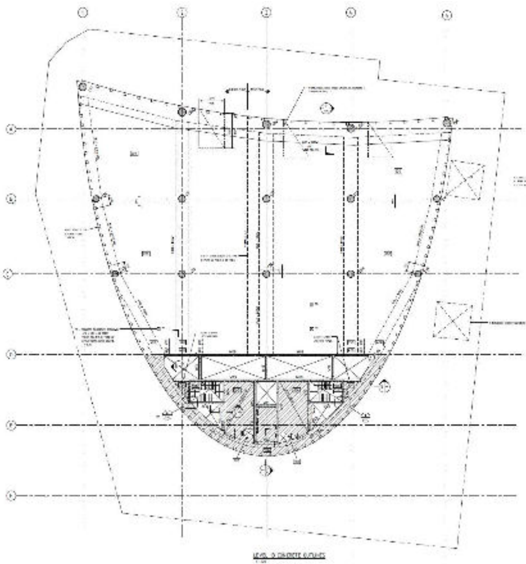


Figure 5.1 Design example to determine the decarbonization impact on the columns

Software developers, Skyciv were commissioned by InfraBuild to produce software using the proposed curvilinear method described in this paper. The software provides engineers with a design tool that generates the axial load-moment interaction curves for a 500 MPa RC column and the corresponding RC column using 600 MPa bars and fitments with the same load capacity. No other changes were made to the design inputs for the columns. This software was used to generate the interaction curves for all 84 columns in this building.

5.1 Redesign results using 600 MPa bars and fitments

The design load combinations for the original column design with 500 MPa standard diameters bars were checked against newly generated 600 MPa design curves and were all found to lie within these interaction curves. This was expected given that the 600 MPa bars have the same load capacity as the 500 MPa bars. and therefore, the axial force / moment interaction curves are closely aligned. Figure 5.2. shows the interaction curves for an 800 mm circular column with 60 MPa concrete and 15D500N24 bars and the equivalent load capacity 600 MPa bars (15D600N21.9); the close alignment of the two curves is typical of all the columns that were redesigned using the corresponding 600 MPa bar.

Three of the column sections in the initial design used the minimum 1% of gross column cross-sectional area of reinforcing steel required by AS 3600 Clause 10.7.1; when the 500 MPa steel was substituted with 600 MPa reinforcing bars the reinforcing percentage fell to 0.97% for column ID 8, 0.94% for column ID 14 and 0.89% for column ID 1. The AS 3600 Commentary notes that 1% minimum steel requirement was "*included when the standard grade of reinforcing steel was 400 MPa, to provide resistance to accidental bending …… It also provides some restraint to creep and shrinkage* *deformations and is usually sufficient to avoid yielding of the longitudinal reinforcement due to shrinkage and to creep under sustained service loading.* Based on this Commentary note, some designers may use their engineering judgement and deem that the three columns with 600 MPa steels are structurally adequate given they had the required strength even though they did not meet the minimum 1% of gross column area requirement of reinforcing steel. The alternative is the addition of bars to meet the 1% requirement.

All 500 MPa column fitments could be substituted with 600 MPa in diameters that provide the same load capacity.

If 600 MPa were used in the design example the total savings in steel used in the columns are in the order of 16.4%. Details of the savings are shown in Table 5.1.

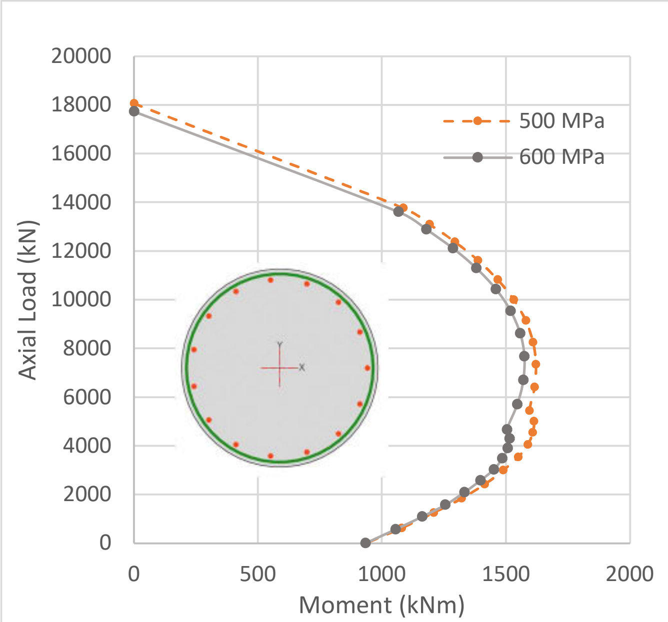


Figure 5.2 Typical column curves with 500 MPa steel and equivalent load capacity 600 MPa steel

Table 5.1 Material savings using 600 MPa reinforcing steels in lieu of 500 MPa reinforcing steels

Element	Tonnes of Reinforcing Steel		Mass Savings (%)	600 MPa Bars Substituted
	500 MPa	600 MPa		
Fitments – All Col	37	31	16.7	All
Main Bars - Column ID 1	1.0	1.0	0	Nil
Main Bars - Column ID 8	6.0	5.4	9.7	All + 1*
Main Bars - Column ID 14	1.5	1.4	1.4	All + 2 *
Main Bars – All Other Columns	236	197	16.6	All
Total	**282**	**237**	**16.4**	

* Denotes bars added to meet minimum steel requirement of 1% gross column cross-sectional area

5.2 Decarbonization

The embodied carbon per tonne in InfraBuild's 600 MPa reinforcing steels is essentially the same as it is for 500 MPa reinforcing steels. Therefore a 16.4% mass saving is equivalent to a saving in embodied carbon of the same order and hence a decarbonization in the column of 16.4%. Given the acknowledgment in the AS 3600 Commentary that the 1% minimum steel requirement was based on 400 MPa steels and is a figure related to the strength of the reinforcing bar, it would be proposed that this be revised, such that the minimum reinforcement is a function of the yield strength of the bar. However even without that amendment to the Standard, this design example demonstrates there is a significant reduction in steel consumed if 600 MPa reinforcing steels were utilised.

Furthermore, there is scope in the existing edition of AS 3600 to further decarbonize by utilizing even higher strength fitments. If 800 MPa fitments were used in this example, a 19.1% reduction in embodied carbon in the column steels would be possible.

6. FURTHER WORK

The natural extension of this work on columns is - concrete walls and bored piers. The current edition of AS 3600 specifically excludes the use of steel with yield strengths in excess of 500 MPa in walls and AS 2159 – 1995, *Piling - Design and installation* also limits the steel yield strengths to 500 MPa. Changes to these Standards and the development of design model that reflect their behaviour in service should provide the construction industry with further opportunities to decarbonize.

7. CONCLUSIONS

The curvilinear stress block model based on the curvilinear stress-strain relationship for concrete provides a higher tier design model for designing RC columns. This higher tier enables the potential offered by 600 MPa reinforcing steels to be realised. As a result, smaller diameter 600 MPa reinforcing bars could generally be used in lieu of standard 500 MPa reinforcing bars. In the design example presented, a 16.4% reduction in the embodied carbon was achieved by substituting 600 MPa bars for 500 MPa bars where it was permitted by the design to AS 3600.

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