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Autor: Bridge, Russell / Patrick, Mark / Webb, John

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High Strength Materials in Composite Construction

Russell BRIDGE
Professor
Uni Western Sydney, Nepean
Kingswood, NSW, Australia

Mark PATRICK
Senior Princ. Res. Engineer
BHP Research - Melb. Labs
Mulgrave, VIC, Australia

John WEBB
Senior Associate
Connell Wagner
Neutral Bay, NSW, Australia

Summary

High strength steels and high strength concretes have been used successfully in composite construction in Australia for some years. The use of high strength materials has developed through both practical considerations and careful research. High strength materials have been used in beams, columns and slabs to improve economy. It has been found that in addition to strength and serviceability, stability, local buckling and ductility are also important effects in the design of composite members incorporating high strength materials.

1. Buildings utilising High Strength Materials

1.1 Benefits

High strength materials offer new improvements in economy in composite construction. High strength materials also offer significant benefits in some specific areas of both concrete and steel construction. In composite construction similar benefits emerge, but the opportunity to combine materials creatively gives other opportunities and challenges.

Particular examples of the benefits in the use of high strength materials include:

- Use of high strength concrete in columns and core walls. Studies (eg. Sparrow [1], Rose & Martin [2]) have shown that increase in concrete strength has an economic benefit. Concrete up to 80 MPa (cylinder strength) is now commonly used in Australian building, while 100 MPa concrete can be supplied in some major cities. Using minimum reinforcement (1%) in conjunction with the highest strength concrete available consistently produces the most economic result. For core walls, this is further accentuated by the increase in rentable space produced by the reduction in structural sizes.

- Use of high strength steel in non-serviceability critical applications including radio masts.

Over the past seven or eight years, Connell Wagner has carried out structural designs for numerous major multi-storey buildings around Australia. Many of these are in the 40 storey plus category. In addition, concept and/or advanced designs have been carried out on very tall buildings such as the 72 and 84 storey schemes for Melbourne Central and options for three supertall buildings in Brisbane. Some advantages of the use of high strength concrete in high rise buildings are given in Table 1.

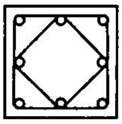
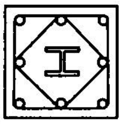
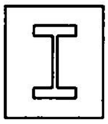
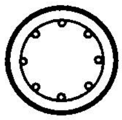
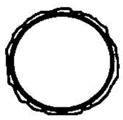
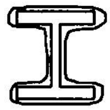
In steel structures, the grades of steel most commonly available have now been raised from 250 MPa and 350 MPa to 300 MPa and 400 MPa respectively providing economies whenever serviceability requirements are not critical. This is also reflected in composite beams for which deflection is not generally critical, provided construction deflection of unpropped beams is controlled by precambering the steel section.

Requirement	Solution
Reduced member sizes, foundation loads	Smaller lighter members
Increased rentable floor space	Reduced core wall thickness and column dimensions
Reduced cost	Less material and easier handling
Wind sway control	Increased column flexural stiffness
Reduced differential shortening	Potential for reduced shrinkage and creep
Early stripping	Strength achieved in shorter time

Table 1 Advantages of high strength concrete in high-rise building construction

1.2 Economics

One of the more interesting aspects of composite construction is in the design of composite columns. Connell Wagner performed many comparisons of column economy during the late 80's and early 90's. These were times of high timber formwork costs when steel systems are particularly attractive. These studies showed the steel tube filled with concrete to be an economical construction medium, which matched the concrete column, and gave other advantages. These columns were used very successfully on the Casselden Place project [3] and several subsequent projects in the early 90's.

No of storeys	**Type 1** Reinforced concrete	**Type 2** Reinforced concrete, steel erection column	**Type 3** Concrete encased steel section	**Type 4** Exposed steel tube filled with reinforced concrete	**Type 5** Fire-sprayed steel tube filled with concrete	**Type 6** Fire-sprayed steel section
						
10 levels	450x450 8Y32	450x450 8Y20 200UC46 Grade 350	410x410 310UC118 Grade 350	500 dia 6Y20 500x6.4 Grade 250	500 dia 500x6.4 Grade 250	310UC240 Grade 350
Relative Cost	1.0	1.22	1.53	1.14	1.10	2.27
30 levels	750x750 20Y36	750x650 12Y36 250UC89 Grade 350	570x570 400x50 flange 360x25 web Grade 350	800 dia. 6Y32 800x10 Grade 250	800 dia. 800x10 Grade 250	Plate girder 500x60 flange 460x40 web Grade 350
Relative Cost	1.0	1.13	1.85	1.11	1.02	2.61

Note: Loaded by 8.4m x 8.4m bays of steel framing

Table 2 Comparison of different column construction options

Costings for a series of alternative column configurations to resist the same load are given in Table 2 (Webb and Peyton [4]). While these costings are not necessarily up to date, they do illustrate some important points eg. a steel circular tube column filled with high strength concrete can be cost effective compared to the all steel column.

Another interesting feature is revealed in Figures 1 and 2. For a constant load carrying capacity for a given concrete strength, the cost of reducing the overall size of a reinforced concrete column using additional reinforcement is very high and becomes prohibitive for higher strength concrete (Figure 1). This phenomenon is not nearly so marked in a tube column (60 MPa concrete) where similar size reductions using increased plate thickness can be achieved with only modest cost penalty (Figure 2).

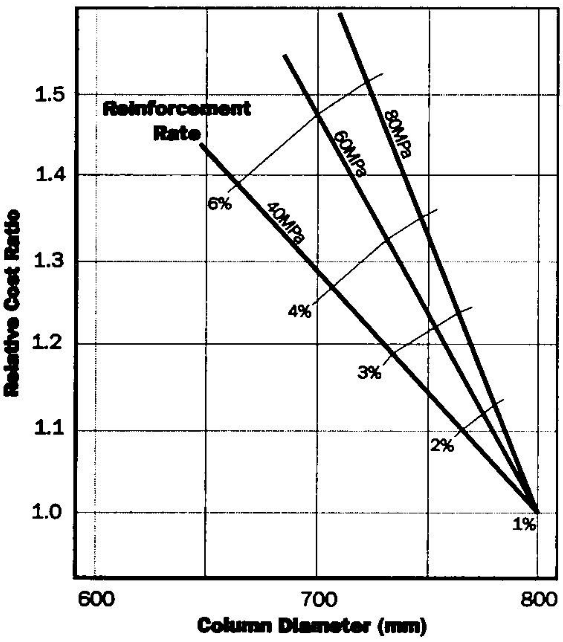


Figure 1 Relative costs for reinforced concrete columns with constant load capacity and varying percentage reinforcement

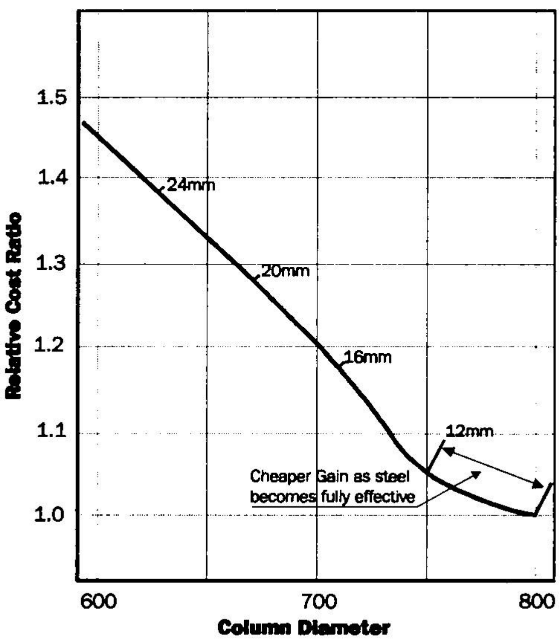


Figure 2 Relative costs for concrete-filled tubes with constant load capacity and varying wall thickness

Tube columns also provide a better environment for high-strength concrete because:

- adequate compaction is possible using pumping techniques alone,
- good curing conditions exist inside the tube,
- minimal creep and shrinkage occur in the effectively sealed conditions with minimal moisture loss, and
- the ductility of the concrete is improved, particularly for the thicker tube walls.

Eurocode 4 [5] also recognises the improved conditions of concrete in a filled tubular section by removing the 0.85 factor on the concrete strength.

1.3 Constructability

The concrete-filled steel tubular column is seen as an attractive viable system because it provides the ability to construct a composite building with the benefits normally associated with traditional steel construction. It is a natural efficient method for using high strength concrete. Research [6] is now underway using these columns with 100 to 120 MPa concrete as outlined in Section 2. The economy of this construction technique is driven by using the minimum amount of reinforcement and steel in the tubes with the highest strength concrete.

Placing concrete for these columns using a pumping technique has been pioneered on the Casselden Place project. The concrete is pumped into the steel tubes through a nozzle connection. In any one lift, concrete is placed from the bottom and pumped up as many as six stories at a time. The placement method, which eliminates the need to vibrate the concrete, was validated by a full-scale prototype test [3].

The tube is erected much like a traditional steel column, with floor reinforcing, concreting, etc. occurring as in conventional steel building practice. The tubes are erected in either two-or three-storey lengths and connected temporarily by turnbuckles. These facilitate plumbing and alignment and allow the crane to quickly release the column. This technique can also achieve excellent tolerances in column plumbing and alignment. The splice is completed using butt-welding, and the turnbuckles are then removed.

Concrete with strength up to 70 MPa (10,000 psi) was used on the project. The superplasticised concrete mix contains silica fume, principally to eliminate bleeding, thereby providing consistency over the height of the placement and eliminating the need to scabble the interface between placements. The method is extremely efficient, and large numbers of columns can be filled at a time by a small workforce and with minimal material waste.

The bare steel tube was capable of supporting six floors of construction, giving the contractor flexibility as to when and where to place the concrete.

2. Steel Tubes filled with High Strength Concrete

2.1 Concrete Stress-Strain Characteristic

High strength concrete is particularly economical when used in circular concrete-filled tubes subjected mainly to axial compression for which the high compressive strength can be fully utilised in design. To obtain this economy, it is essential that the highest strength can be achieved still using conventional materials and manufacturing and placement procedures.

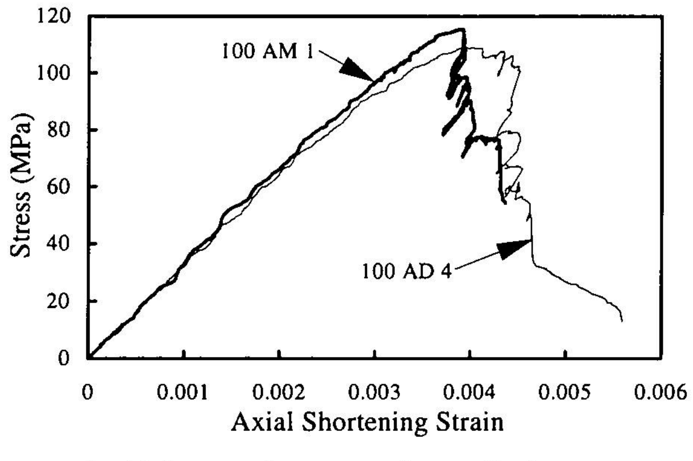


Figure 3 Stress-strain curve for high strength concrete from cylinder tests.

A commercial concrete mix has been developed that meets these objectives and can develop a compressive strength around 120 MPa. However, the post-ultimate stress-strain response of high strength concrete can be characterised by a very rapid unloading, even exhibiting the phenomena of "snap-back". This brittle behaviour cannot be measured using normal testing procedures including displacement control. A number of alternative techniques can be used. A simple method is to use the diametric strain to control the displacement of the testing machine as this was found to increase monotonically with axial displacement. The results of such a test are shown in Figure 3. The concrete cylinder labelled with an "M" was moist-cured in a lime bath at

100% relative humidity and a constant temperature of 20°C. The cylinder labelled with a "D" was dry-cured by sealing the cylinder in a polythene wrap and storing it at ambient temperatures to simulate conditions in the concrete-filled tube specimens.

2.2 Concrete-filled Steel Tubes - Steel and Concrete Loaded

For concrete-filled steel tubes in which the concrete and steel are loaded simultaneously, enhancement of concrete strength due to confinement by the tube can be obtained for low to medium strength concretes and this has been recognised in Eurocode 4 [5]. The cross-sectional strength N_{u0} in axial compression including the beneficial effects of concrete confinement for concretes with strengths up to a maximum of 50 MPa is given by

$$N_{u0} = A_s \eta_2 f_y / \gamma_s + A_c f_c / \gamma_c \left(1 + \eta_1 (t/D)(f_y / f_c)\right) \tag{1}$$

where A_s and A_c are the cross-sectional area of the steel and the concrete, f_y and f_c are the characteristic strengths of the steel and concrete, and γ_c and γ_s are the partial safety factors which may be taken to be unity when the strength of the materials has been accurately measured as in laboratory tests. The increase in concrete strength from confinement (accounted for by the η_1 factor) and the corresponding decrease in steel strength (accounted by the η_2 factor) may be considered if the non-dimensional column slenderness λ is less than 0.5 and the eccentricity of loading does not exceed $D/10$ where D is the external tube diameter.

Recent axial load tests by O'Shea and Bridge [7] on steel tubes filled with high strength concretes with strengths in excess of 100 MPa have revealed that virtually no enhancement can be obtained, the concrete behaving as if unconfined up to the maximum strength. Five tubes designated CS were tested under axial load. Their dimensions and material properties are given in Table 3 together with the maximum load capacity N_{Test} obtained in the test.

Tube	Diameter (mm)	Thickness (mm)	Length (mm)	Average Concrete Strength (MPa)	Steel Yield Stress (MPa)	Max. Load N_{Test} (kN)
S30CS	165	3.00	578	113.5	364	2673
S20CS	190	2.00	660	113.5	272	3360
S16CS	190	1.55	662	113.5	315	3260
S12CS	190	1.15	660	113.5	185	3058
S10CS	190	0.95	662	113.5	211	3070
S20CL	190	2.00	654	113.5	272	3690
S12CL	190	1.15	662	113.5	185	3220

Table 3. Specimen dimensions and properties

In Table 4, code predicted strengths N_{Code} (Equation 1) are compared to the actual test strengths N_{Test}. The code values for η_1 and η_2 are calculated using Eurocode 4 [5]. Using these values, the equivalent factor implied in the code for concrete strength enhancement (a factor of 1.0 being zero enhancement) can be calculated and is shown in Table 4 under "Concrete Enhancement EC4", the thicker tubes having more enhancement. The Grimault and Janss [8] effective steel area was used in the calculations.

An approximate value of the concrete enhancement factor in the tests can be calculated by subtracting the maximum bare steel strength from the maximum concrete-filled steel tube capacity and dividing the remainder by the concrete area and the concrete cylinder strength. This is shown in Table 4 under "Concrete Enhancement Test". The values shown are all slightly less than unity suggesting that there is little concrete enhancement (or alternatively the concrete cylinder strengths are inaccurate). As the actual concrete strength was determined from at least ten cylinder tests, it is more likely that there is little confinement of the concrete in the steel tubes. This is also supported by the fact that the test values are essentially constant and do not increase with tube wall thickness. If no confinement is assumed i.e. letting $\eta_2 = 1.0$ and $\eta_l = 0$, Eurocode 4 [5] provides a good prediction of the section strength as shown in the last column of Table 4. When confinement is considered, Eurocode 4 [5] is slightly unconservative for axially loaded thin-walled steel tubes infilled with high strength concrete. Therefore, confinement for high strength concretes with strengths in the range 100 MPa and above should be ignored.

Specimen	η_2 steel EC4	η_l concrete EC4	Concrete Enhancement EC4	Concrete Enhancement Test	N_{Test}/N_{Code} Confinement EC4	N_{Test}/N_{Code} No Confinement EC4
S30CS	0.866	1.514	1.083	0.949	0.927	0.966
S20CS	0.872	1.390	1.033	0.996	0.982	1.000
S16CS	0.879	1.265	1.025	0.969	0.967	0.980
S12CS	0.883	1.183	1.011	0.939	0.946	0.952
S10CS	0.888	1.084	1.008	0.942	0.957	0.961

Table 4. Comparison of test results with Eurocode 4 [5]

2.3 Concrete-filled Steel Tubes - Concrete Loaded

Two circular tubes designated CL were tested [7], S20CL and S12CL, with only the concrete loaded. Their dimensions are given in Table 3 and were similar to their companion S20CS and S12CS tubes. The concrete was loaded axially with the steel unbonded. This was achieved through greasing the internal tube surface prior to filling with concrete. Strains measured on the steel tubes using rosettes verified the debonding procedure. Special loading disks were manufactured to ensure that the axial load was only applied to the concrete. Therefore the steel only provided lateral confinement to the concrete and did not carry any direct axial load.

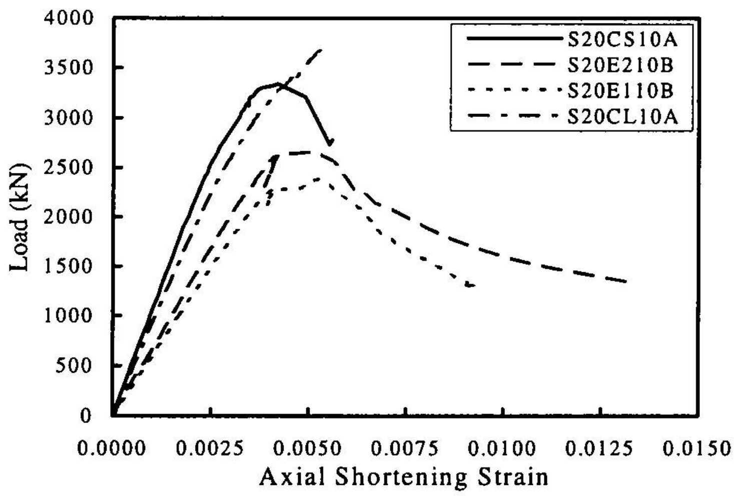


Figure 4 Load - axial strain response for S20 concrete-filled steel tubes

The load-axial strain response for the axially loaded S20CL tube is compared with that for the companion S20CS tube in Figure 4. Also shown is the response for companion eccentrically loaded tubes S20E1 and S20E2 with eccentricities of D/10 and D/20 respectively. These have a more ductile response than the axially loaded tubes.

It can be seen that the maximum load for the concrete loaded only specimen S20CL was higher than that for specimen S20CS for which the concrete and steel were loaded simultaneously. The same behaviour was observed for the S12 tubes as indicated in the last column of Table 3. Therefore, an increased strength can be obtained without the steel being axially loaded. The strength increase was 9.8% for the thicker S20 tubes and 5.3% for the thinner S12 tubes, the thicker tube providing more confinement and hence higher strength increase as expected. Hence, confinement of high strength concrete is possible provided the concrete alone is loaded.

The more efficient use of steel tubes filled with unbonded concrete with only the concrete loaded has been proposed by Orito et. al. [9]. However, detailing, especially at beam to column joints, needs careful consideration to ensure axial load is not transferred to the steel tube.

2.4 High Strength Steels - Local Buckling

With the use of high strength steels, thinner steel sections can be achieved for the same load carrying capacity. However, the effects of local buckling have to be considered. O'Shea and Bridge [7] have examined the local buckling of circular steel tubes with or without concrete infill. The results of axial load tests on bare steel tubes (BS tests) and steel tubes with unbonded concrete infill with only the steel loaded (BSC tests) are shown in Table 5.

Specimen	Test bare steel (kN)	Test concrete infill (kN)	AS4100 [10] (kN)	Grimault & Janss [8] (kN)
S30BS/C	523.3	521.6	510.4	521.6
S20BS/C	284.5	279.9	270.9	304.3
S16BS/C	239.2	283.8	202.0	252.1
S12BS/C	109.1	109.3	100.5	118.2
S10BS/C	92.9	91.0	71.4	91.2

Table 5 Capacity of steel loaded circular tubes

It was found, in general, that the concrete infill did not enhance the local buckling strength as the local buckle was a circumferential "elephant's foot" buckle occurring at one end of the tube. This outwards buckle is not restrained by the concrete infill which therefore has no effect. As shown in Table 5, the local buckling strength for bare steel tubes can also be predicted reasonably accurately using the design rules in current steel codes [10] and the literature [8].

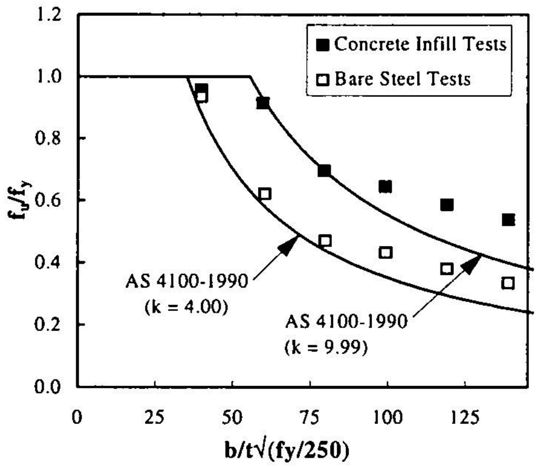


Figure 5 Strength of steel-loaded square tubes

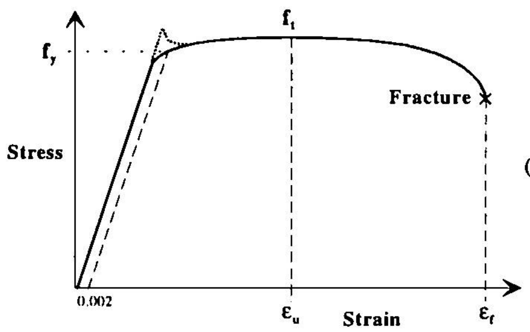
Bridge and O'Shea [11] have also examined the effects of local buckling for square thin-walled steel tubes with or without concrete infill. For the bare steel tubes, the local buckling pattern exhibits both inwards and outwards buckling deformations and the plate elements forming the walls can be considered as having simply supported edges with a buckling coefficient k of 4.0. The concrete infill prevents any inwards buckling and the plate elements can be considered as having clamped edges with a buckling coefficient k of 9.99. This results in an increase in column strength as evidenced in the test results plotted in Figure 5 where b is the width of the tube and t is the plate thickness. Also, current design methods in standards such as AS4100-1990 [10] can be used to predict the plate strength f_u of such tubes provided the correct value of buckling coefficient k can be included in the design method.

3. Composite Slabs utilising Galvanised High Strength Sheet Steels

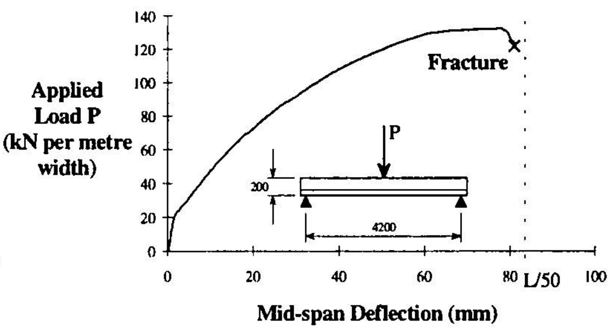
Australian profiled steel sheeting has been consistently manufactured from galvanised, high-strength (f_y = 550 MPa) sheet steels for over 30 years. Research by Australian industry has ensured that the major Australian profiles develop strong mechanical resistance with the concrete. In conventional composite slabs in steel-frame buildings, which typically involve unpropped spans of about three metres and depths of less than 200 mm, the thickness of the sheeting is often determined at the formwork stage when the high yield stress of the steel can be utilised. This can result in the tensile capacity of the sheeting being significantly under-utilised in the design of the composite slab. However, G550 steel can be used most effectively as the overall depth, span and applied load increase. Long-spanning composite slabs in bandbeam construction may have depths up to 400 mm.

3.1 G550 Sheet Steels

The G550 sheet steels used to make profiled steel sheeting range in thickness from 0.6 to 1.0 mm with a total zinc coating mass of 200 to 450 g/m^2. Steels of such high strength are not normally used for this purpose in Europe or America where hot-rolled grades of between about 275 and 350 MPa predominate. Some potentially undesirable properties of G550 steels are their low elongation at fracture and lack of strain-hardening.



(a) Stress-strain curve from coupon testing



(b) Composite slab load-deflection curve

Figure 6 Tensile testing of G550 sheet steels

3.1.1 Material Standards

Cold reduction is used rather than an alloying process to produce the G550 sheet steels from hot-rolled 2.5 mm thick steel which has a minimum specified yield stress of 300 MPa. The material property requirements for G550 sheet steels are specified in AS1397 [12]. The minimum yield stress and tensile strength are both 550 MPa, and the minimum elongation in either a 50 mm or 80 mm gauge length is 2 per cent.

3.1.2 Stress-Strain Curve

Coupons taken in the longitudinal direction of a coil and tested at a low strain-rate exhibit a stress-strain curve of the form shown in Figure 6(a). Material properties are calculated assuming only the base metal is present. The G550 sheet steels may exhibit an upper yield (dashed peak in Figure 6(a)) or else yield gradually (depending on processing after coating), in either case with minimal strain hardening. Yield stress f_y is calculated using the 0.2% strain offset method, and values of the tensile-strength-to-yield-stress ratio (f_t/f_y) equal to unity are consistently obtained. The modulus of elasticity is typically about 200 GPa, and this value can be used for design.

Yield stress values for G550 sheet steels are significantly above the minimum specified 550 MPa. Mean strengths tend to increase as the base metal thickness reduces due to the increased amount of cold reduction. Large variation occurs in the strength and ductility of material taken from different mills. Post-ultimate ductility measured by the fracture-to-ultimate-load ratio is comparable to that for mild sheet steels and can reach as low as 0.7.

3.2 Australian Profiled Steel Sheeting

The major Australian profiles have two important similarities, viz.: (1) the steel ribs are very narrow in width compared with the flat pans, and therefore a transverse section of a slab closely resembles that of a solid slab; and (2) the lapped ribs formed when sheets are joined together on site are specially shaped to develop strong, ductile mechanical resistance with hardened concrete.

3.2.1 Mechanical Resistance developed with Concrete
The Slip-Block™ Test has been specially developed in Australia to measure mechanical resistance τ as a function of slip after the breakdown of adhesion bond [13]. The coefficient of friction μ developed between the sliding surfaces is also measured during the test. Both these parameters are used in partial shear connection theory for designing composite slabs [14].

Tests on the Australian profile Bondek® II have yielded design values $\tau = 115\sqrt{(t_{bm}f'_c)}$ in kPa and $\mu = 0.6$, where t_{bm} is the nominal base metal thickness of the sheeting (mm) and f'_c is the characteristic compressive strength of concrete at 28 days (MPa) [15]. The tops of the sheeting ribs are embossed which contributes significantly to the magnitude and ductility of the mechanical resistance they develop. Embossing must not cause any local fracturing in the G550 sheet steel, since this can lead to premature fracture of sheets when they act as reinforcement at the composite stage.

3.2.2 Quality Control
Profile features critical to the development of mechanical resistance (e.g. lap joint shape, embossments, etc.) must have their geometries adequately maintained by an operational quality-control program. Additional variations that can result because the profile is manufactured at different sites must be considered when establishing minimum production standards.

3.2.3 Residual Stresses
When a profile is roll-formed from coils of G550 sheet steel, large residual stresses typically of ±100 MPa are induced in the sheeting which vary across its width.

3.3 Design of Conventional Composite Slabs

The slabs may be designed as simply-supported for some limit states and continuous for others, subject to certain conditions being satisfied. Reinforcing steel is placed transversely in the slabs for shrinkage and temperature control. Longitudinal reinforcement may be placed in the top face over supports to increase negative moment capacity and control flexural cracking. It may also be placed in the bottom face, where (because the Australian profiles develop strong, ductile mechanical resistance) it can act in conjunction with the sheeting and increase the positive moment capacity. A set of limit-state design rules has recently been prepared to Australian Standards for composite slabs incorporating Bondek II [15] and are briefly discussed below.

3.3.1 Design for Strength
Design for strength is concerned with ensuring that there is sufficient moment and shear capacity in both positive and negative moment regions. Elastic analysis may be used to calculate the design action effects of continuous slabs, and the amount of redistribution permitted depends on the ductility of hinge regions and is influenced by both the amount of the steel reinforcement and its tensile properties. If high-strength, low-elongation reinforcement is used over supports, either no redistribution is permitted [16], or the slabs must be assumed to revert to simple spans at

ultimate load due to possible fracture of the reinforcement (100 per cent redistribution). Premature fracture of low-elongation reinforcement in the bottom-face acting in conjunction with the steel sheeting may also be a problem and may need to be considered in design.

Partial shear connection strength theory can be used to design the positive moment regions of composite slabs exhibiting one-way action. Simplified equations have been formulated to calculate the design positive moment capacity of Bondek II slab cross-sections with either partial or complete shear connection and including the contribution of bottom-face reinforcement. The model used in the theory recognises the existence of support friction and the additional anchorage of the sheeting continuing into adjacent spans. The accuracy of the model has been validated by testing [17, 18].

In one such test, the theory was used to predict the failure load of a Bondek II slab (see Figure 6(b)). The test showed that the full moment capacity of the peak moment cross-section (corresponding to yielding of the steel sheet over its full cross-sectional area) was reached. The stress-strain curve of the steel (t_{bm} = 0.750 mm) in the longitudinal direction took the form shown in Figure 6(a) with an upper yield point. Tensile coupons from the steel gave varied results with yield stress f_y ranging from 630 to 635 MPa, upper yield stress from 640 to 660 MPa, and elongation after fracture (50 mm gauge length) from 5.6 to 8.5 per cent. Interestingly, however, uniform elongation (37.5 mm gauge length outside fracture zone) was close to zero for coupons with a high upper-yield-stress-to-yield-stress ratio, while it reached a maximum of nearly 5 per cent for other coupons. In the slab test, failure occurred when the steel fractured at a mid-span deflection of almost span/50, which is acceptable considering that a single line-load was used in the test and there was a steep moment gradient. Fracture occurred at a major flexural crack and extended across a whole sheeting pan and through an adjacent rib. The peak of the curve in Figure 6(b) could be predicted very accurately. More complex situations involving the effects of partial shear connection and support friction have also been accurately predicted (within 10 per cent) using the theory [18]. The residual stresses described in Section 3.2.3 created during roll-forming can be ignored in the analysis, as can any initial flexural stresses arising during construction prior to composite action.

The shear capacity of the positive moment regions of Bondek II slabs has been investigated experimentally [19], and it has been recommended that for this profile the occurrence of vertical shear failure (viz. by diagonal splitting or flexure shear) can be ignored under uniform-loading conditions. This recommendation applies only when the amount of reinforcing steel taken into account in determining the moment capacity of the slab is less than a certain amount [15].

3.3.2 Design for Deflection Control

For normal situations experienced in steel-frame buildings, design is satisfactory treating the sheeting as fully effective reinforcement and using methods applicable to reinforced-concrete for calculating section properties. Limits on total and incremental deflections (Δ_{tot} and Δ_{inc}) are normally specified. However, design for deflection control of Bondek II composite slabs in long-spanning applications is currently under review and a variety of tests are being performed.

3.3.3 Design for Crack Control and Durability

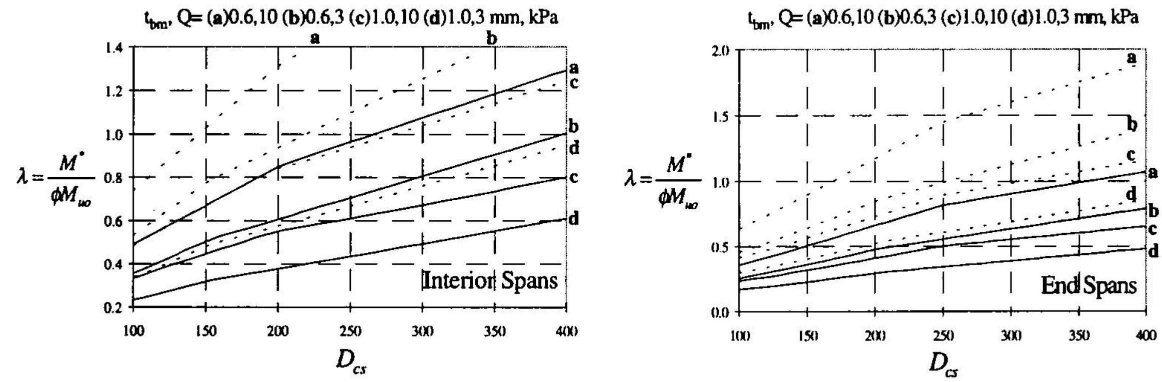
The negative moment regions of a composite slab are susceptible to top-surface flexural cracking. Controlling such cracking is particularly important as excessive crack widths can give an overall impression of poor quality and can limit the types of floor coverings that can be successfully used. Excessively wide cracks can also provide a pathway for the ingress of corrosive substances such as water. As a general rule, a designer should aim to detail the member such that under service loading the tensile strain at the top surface will be distributed over a large number of narrow cracks rather than a small number of wide cracks.

The main design principles are: (i) crack width is calculated using a simplified equation derived using the method for calculating crack width defined in BS 8110, Part 2 [20]; (ii) a limiting width of 0.3 mm specified in BS 8110, Part 2 is adopted; (iii) elastic analysis is used to calculate design bending moments under both short-term and long-term service loading; (iv) the stress in the reinforcement is calculated using elastic, cracked-concrete modular ratio theory and is kept below the yield stress under short-term service loading and 80 per cent of the yield stress under long-term service loading; and (v) the nominal negative moment capacity of the support regions is to exceed the cracking moment by at least 20 per cent so that more than one flexural crack will form.

3.4 Factors affecting Utilisation of Sheet Steel Tensile Capacity

Bandbeams are wide, shallow beams that are supported in a parallel arrangement on isolated columns. Slabs are formed between the bandbeams. Both the slabs and bandbeams are assumed to exhibit one-way action. While slab depths in steel-frame buildings seldom exceed 200 mm, slabs spanning between bandbeams can reach 400 mm or more. The span-to-overall-depth ratio of a slab (L/D_{cs}) in either of these situations typically varies between 20 and 40, depending on the support conditions, magnitude of loading and deflection limits.

The design rules described in Section 3.3 have been used to prepare sets of solutions for examining the effect of base metal thickness, live load, yield stress, moment redistribution and slab support conditions on the utilisation of sheet steel tensile capacity in Bondek II slabs. For brevity, only some of the results are presented here. Slab overall depth was considered to vary from 100 to 400 mm. All solutions satisfy the criteria concerned with design for strength, deflection control ($\Delta_{tot}/L \leq 1/250$ and $\Delta_{inc}/L \leq 1/350$), and crack control when appropriate. Utilisation is examined for the positive moment region using the ratio of design positive bending moment to design positive moment capacity, i.e. $\lambda = M^*/(\phi M_{uo})$. The sheeting is assumed to be the only bottom-face reinforcement, and it is assumed to be fully anchored at the critical cross-section. Cases for which $\lambda > 1$ imply that the tensile capacity of the sheeting is fully utilised and additional bottom-face reinforcement is required for strength.



(a) Yield stress varied from 550 to 350 MPa *(b) Moment redistribution of 100%*

Figure 7 Factors affecting utilisation of G550 sheet steels in Bondek II slabs

The cases shown in Figure 7(a) are for interior spans. The solid and dashed lines are for $f_y = 550$ and 350 MPa, respectively. Solutions for different combinations of base metal thickness t_{bm} and live load Q are also labelled. It can be seen that G550 steel is most effectively utilised in situations corresponding to bandbeam construction when depths are large (>250 mm), live load is heavy (10 kPa) and base metal thickness is least (0.6 mm). However, the solutions for G350 steel show that its tensile capacity can be exceeded in steel-frame building situations and hence bottom-face reinforcement would be required to supplement the sheeting.

The cases shown in Figure 7(b) are for end spans using only G550 steel The solid and dashed lines are for 0% and 100% redistribution respectively from negative to positive moment. The values of λ can be seen to increase substantially with moment redistribution indicating better utilisation of the high strength G550 steel. (Crack control reinforcement must be provided separately for this case if required.)

The effects of partial shear connection on the curves in Figure 7 can be ignored for slabs incorporating Bondek II on account of the strong mechanical resistance it develops.

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