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ON THE STRUCTURAL BEHAVIOR OF COMPOSITE BEAMS USING COLD-FORMED SHAPES

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SUMMARY

Cold-formed shapes have been increasingly employed in the construction of small multiple-floored buildings in Brazil, because wide flange shapes required to meet the demand are unavailable on the Brazilian market. Because existing codes fail to cover the design of composite beams constituted of cold-formed shapes, Brazilian design engineers have usually disregarded the contribution of the slab or have adopted improvised calculations that may to lead to unsafe results.

The structural behavior of composite beams, constituted of cold-formed double channel shapes and connected to the concrete slab by means of shear connectors that were also cold-formed, was investigated experimentally. Simply supported beams were tested in which the degree of connection (number of connectors) and the type of load (concentrated or distributed load) were varied. The results were then compared to those obtained through classic theoretical models of composite beam design, which supplied relevant conclusions for the design of these elements.

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INTRODUCTION

Structures using cold-formed steel shapes have been increasingly employed in Brazil, mainly due to the insufficient availability of hot-rolled shapes and the relatively high cost of welded shapes. The application of cold-formed steel, which in the past was limited basically to the construction of light-weight industrial buildings, has become more diversified and today these steel shapes are employed in small buildings (up to four floors), industrial floors with low overloads, foot-bridges, etc.

The Brazilian code for cold-formed steel structural members does not include recommendations for the design of composite structures. This has led to two situations in terms of design: disregard of the behavior of composites, which in turn represents a higher consumption of steel and, hence, higher costs, or the use of calculation procedures of the codes destined for hot-rolled shapes (Brazilian Code NBR 8800 or the AISC Specification), which may be hazardous to safety.

A theoretical and experimental investigation was carried out to assess the structural behavior of beams composed of cold-formed steel (double channels) connected to the concrete slab by means of shear connectors that are also cold-formed (angles). The experimental results were compared to the theoretical ones based on code recommendations, which allowed for the establishment of relevant conclusions regarding the design of these elements.

SHEAR CONNECTORS

Initially, shear connectors constituted of cold-formed steel shapes (channel and angle) were analyzed, with dimensions and strength compatible with the beams tested. The load-slip behavior and the capacity of the connectors were obtained based on push-out tests (figure 1a), the results of which were presented at the 14th Conference, MALITE et al. (1998).

All the connectors tested presented the load-slip behavior illustrated in figure 1b, characterizing a flexible (ductile) behavior. This behavior allows for uniform spacing of the connectors along the length of the beam, since flexibility implies a redistribution of the shear flow to the adjacent connectors, leading to a situation of uniform shear flow in the steel-concrete interface.
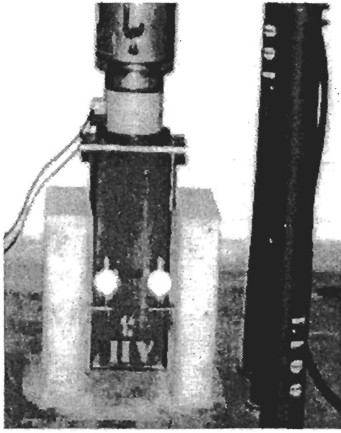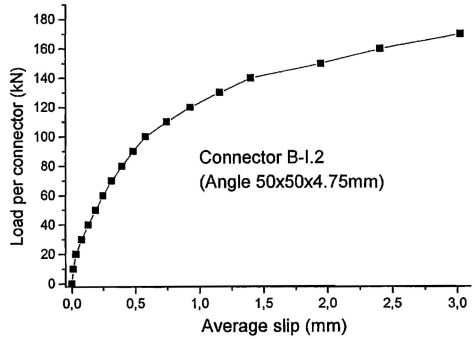
Fig. 1a – *Push-out test*

Fig. 1b – *Typical load-slip curve*

STRENGTH OF BEAMS WITH SHEAR CONNECTORS

For regions having a positive moment, the AISC Specification (1993) and Brazilian Code NBR 8800 (1986) present similar procedures to determine the nominal flexural strength in composite beams with shear connectors, considering two cases:

- beams with compact web: $h/t_w \leq 3.5(E/f_y)^{0.5}$

The nominal flexural strength is calculated based on the plastic distribution of stresses in the composite section and is, therefore, applicable only to beams with compact web. The plastic neutral axis (PNA) may be located in the concrete slab, in the top flange or in the web of the steel beam (figure 2). For fully composite beams, the nominal flexural strength of the composite section is calculated using the following expressions:

    a) Plastic neutral axis in the concrete slab

$$M_n = T\big[y_1 + h_F + d_c - 0.5a\big] \tag{1}$$

    b) Plastic neutral axis in the top flange or in the web of the steel beam

$$M_n = C'\big[d_s - y_t - y_c\big] + C\big[0.5d_c + h_F + d_s - y_t\big] \tag{2}$$

For partially composite beams, the compressive strength in the slab *(C)* is limited as a function of the steel-concrete connection capacity *(ΣQₙ)*, and the relative slip between steel and concrete leads to two neutral axes: one in the concrete slab and the other in the steel

beam. Figure 3 illustrates the plastic distribution of stresses, and the nominal flexural strength is calculated by expression (3).

$$M_n = C'[d_s - y_t - 0.5y] + C[d_s + d_c + h_F - 0.5a - y_t] \qquad (3)$$

For fully and partially composite beams, the flexural design strength is calculated by $\phi_b M_n$, with $\phi_b = 0,85$ according to the AISC Specification (1993) and $\phi_b = 0,90$ by the Brazilian Code NBR 8800 (1986).

- beams with slender web: $h/t_w > 3.5(E/f_y)^{0.5}$

The nominal flexural strength should be calculated based on the elastic distribution of stresses, so that the maximum tension stress in the steel beam does not exceed the yield stress and the maximum compressive stress does not exceed the strength of the concrete. The transformed section should be considered, which is obtained by dividing the effective width of the concrete slab by modulus of elasticity ratio $n = E/E_c$, disregarding the concrete in tension (figure 3). The maximum stresses in the steel and the concrete are calculated by:

$$\sigma_s = \frac{M}{I_{tr}} y_{bottom} \qquad (4)$$

$$\sigma_c = \frac{1}{n} \frac{M}{I_{tr}} y_{top} \qquad (5)$$

For partially composite beams, the relative slip between steel and concrete alters the distribution of stresses, reducing the stiffness of the beam and, hence, increasing the tension stress in the steel beam, which is calculated by expression (4), altering the value of the moment of inertia of the transformed section I_{tr} to I_{eff}, according to expression (6).

$$I_{eff} = I_s + \sqrt{\frac{\Sigma Q_n}{V_h}} (I_{tr} - I_s) \qquad (6)$$

In expression (6), V_h is the smallest value between $(A f_y)$ and $(0.85 f_c \, b \, d_c)$.

For fully and partially composite beams, the design flexural strength is calculated by $\phi_b M_n$, with $\phi_b = 0,90$ by the AISC Specification (1993) and the Brazilian Code NBR 8800 (1986).
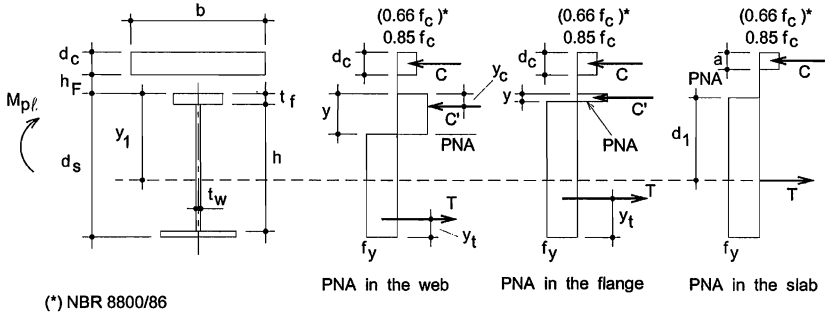
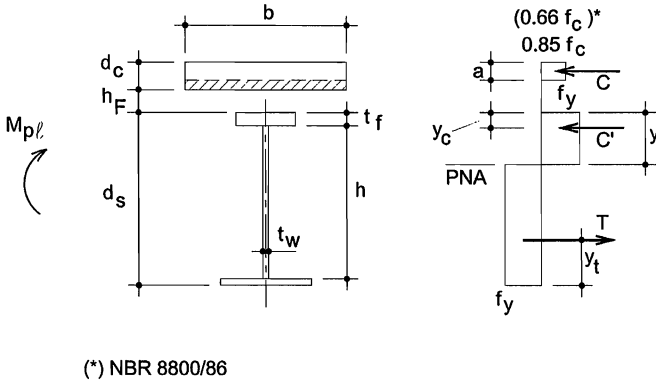*Fig. 2 – Plastic stress distribution for fully composite beams*

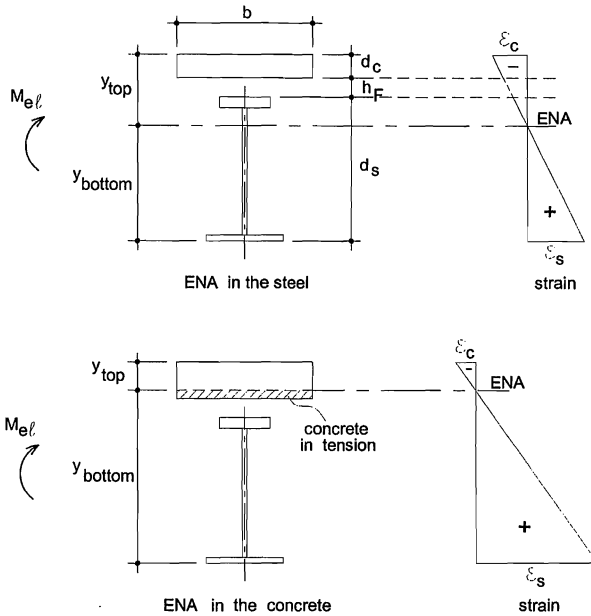*Fig. 3 – Plastic stress distribution for partially composite beams*

Fig. 4 – *Elastic stress distribution for composite beams*

EXPERIMENTAL ANALYSIS

The structural behavior of the composite beams simply supported, constituted of cold-formed steel shapes (double channel 250x75x3.75mm) was analyzed on the basis of the tests carried out on four beams, VM1 to VM4. As a parameter for comparison, two isolated steel beams, VA1 and VA2, were initially tested (without the concrete slab), having the same characteristics as the composite beams. The dimensions of the steel beam and the concrete slab were the same in the four tests, with variations in the degree of interaction and the type of load, as described in table 1 and illustrated in figure 5.

Table 1 – *Description of steel beams and composite beams*

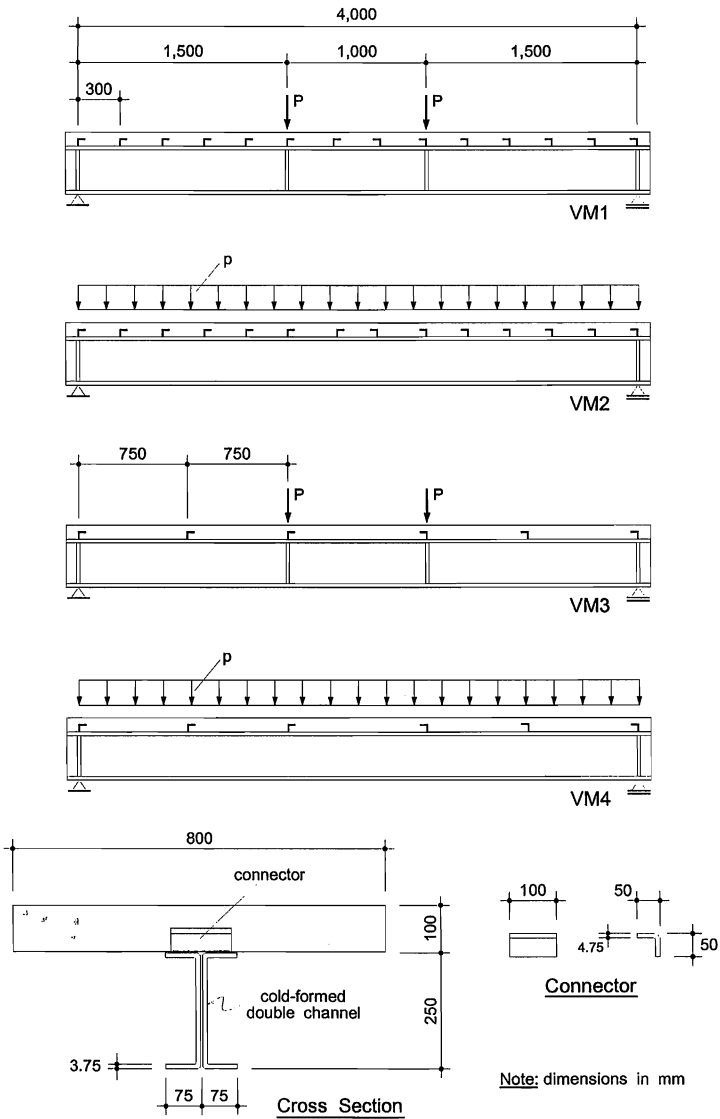| Beam	Description	Shear connection	Load
VA 1	Steel beam	-	Concentrated loads
VA 2	Steel beam	-	Uniformly distributed load
VM 1	Composite beam	Full	Concentrated loads
VM 2	Composite beam	Full	Uniformly distributed load
VM 3	Composite beam	Partial	Concentrated loads
VM 4	Composite beam	Partial	Uniformly distributed load

Fig. 5 – *Dimensions and loads of the tested beams*

Cold-formed shear connectors (angle 50x50x4.75mm, length 100mm) with an average capacity of $Q_n = 186kN$, determined based on push-out tests shown earlier herein, were used for all the composite beams.

The mechanical properties of the steel of the beams and the connectors (virgin steel) were determined based on tension tests following the ASTM A370 (1992) code; the average values are listed in table 2. The compressive strength and the modulus of elasticity of the concrete of the slab were established based on compression tests (cylindrical specimens), and the average values for the four composite beams are given in table 3.

Table 2 – *Average tensile properties of virgin steel - ASTM A370 (1992)*

| Element	f_y (MPa)	f_u (MPa)	E (MPa)	A(%)
Beam	377	543	203,000	29.2
Shear connector	246	360	203,000	39.8
Where f_y = yield strength; f_u = tensile strength; E = modulus of elasticity A = elongation (in 50mm)				

Table 3 – *Average properties of concrete at 28 days*

Composite beam	$f_{c,28}$ (MPa)	$E_{c,28}$ (MPa)
VM 1	31.9	36,844
VM 2	29.6	33,467
VM 3	28.1	33,172
VM 4	29.0	38,194
Where $f_{c,28}$ = compressive strength at 28 days $E_{c,28}$ = modulus of elasticity of concrete at 28 days		

The vertical displacements were measured in all the tests at five points along the span of the beam and on the supports, using dial gages with 0.01mm sensitivity. Strain gages were fixed to several sections of the steel beam and the concrete slab to measure strains along the section. Figure 6 gives a general view of composite beam VM3 in the final phase of testing, and the instrumentation on beams VM1 to VM4. The isolated steel beams (VA1 and VA2) were instrumented in the same way as the composite beams.
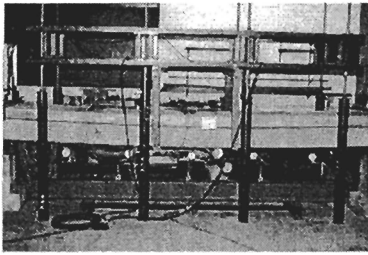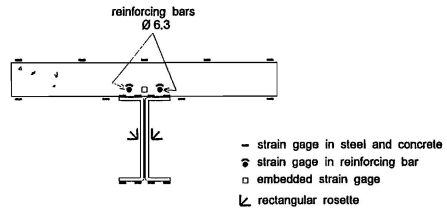
| a) General view of the test | b) Instrumentation of the composite beams |

Fig. 6 – *General view of the test and instrumentation of the composite beams VM 1 to VM 4*

ANALYSIS OF THE RESULTS

The experimental results of the tests of the two steel beams and the four composite beams were compared to the theoretical values deriving from the elastic analysis, calculated with the geometrical properties of the transformed section. The calculation adopted the modulus of elasticity ratio $n = E/E_c = 6$, which corresponds to the mean value obtained for the four beams. Table 4 indicates the failure modes observed in the tests as well as the theoretical and experimental values in the ultimate moment of the steel beams and the composite beams, while figure 7 illustrates the failure modes of the steel beams and the partially composite beams.

Figure 8 reveals the displacements at mid-span for the beams with concentrated load (VA1, VM1 and VM3) and uniformly distributed load (VA2, VM2 and VM4), showing the difference in stiffness between the fully and partially composite beams and after the loss of steel-concrete adherence. The theoretical values in the two graphs correspond to the fully composite beam, calculated with $I_{tr} = 11,038cm^4$.

The difference in stiffness can also be seen by the strains in the bottom flange of the steel beam, whose mean theoretical and experimental values are given in figure 9. In this case, the theoretical values also correspond to the fully composite beam. At the moment when the slip between the steel beam and the slab occurs, the shear flow is resisted by the connectors, causing a sudden drop in the stiffness of the partially composite beams.

The difference in the behavior between the fully and partially beams is also illustrated in figure 10, which characterizes the instant when the slip between the beam and the slab occurs. In the case of fully composite beam, the elastic neutral axis is positioned on the slab; hence, the top flange of the steel beam is tensioned. There are two neutral axes in partially composite beam, one in the slab and the other in the beam, causing the top flange of the steel beam to become compressed.

Table 4 – *Theoretical and experimental values of the ultimate moment*

Beam	Failure mode [*]	M_n (kN.m) (elastic theor.) *(1)*	M_n (kN.m) (plastic theor.) *(2)*	M_u (kN.m) (experimental) *(3)*	(3)/(1)	(3)/(2)
VA 1	A	55	86	67	1.21	0.78
VA 2	A	55	86	74	1.34	0.86
VM 1	B	155	224	195	1.26	0.87
VM 2	B	155	224	211	1.36	0.94
VM 3	C	132	185	165	1.25	0.89
VM 4	C	132	185	178	1.35	0.96

(*) *A:* flange local buckling (see figure 7a)
    B: excessive strain of the bottom flange
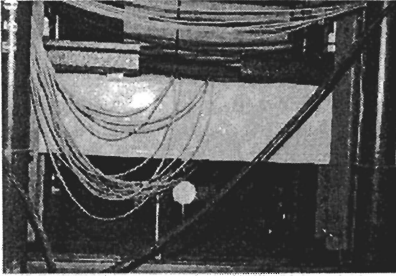   *C:* rupture of the concrete at the shear connector (see figure 7b)

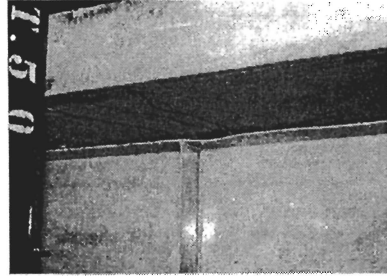**Fig. 7a** – *Flange local buckling of the steel beam*

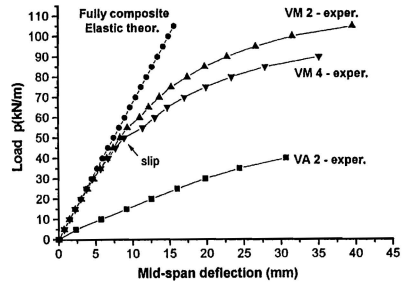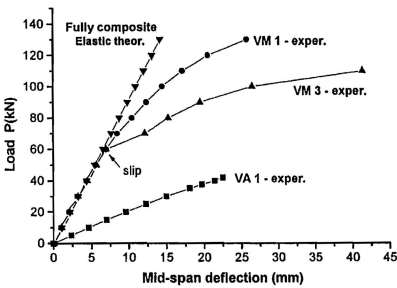**Fig. 7b** – *Failure of the concrete in the partially composite beams*

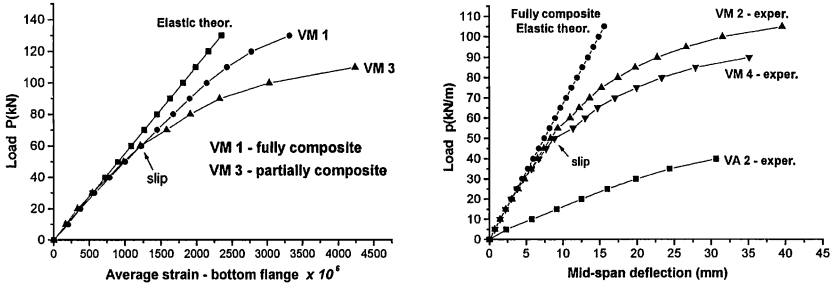**Fig. 8** – *Load versus mid-span deflection curves*

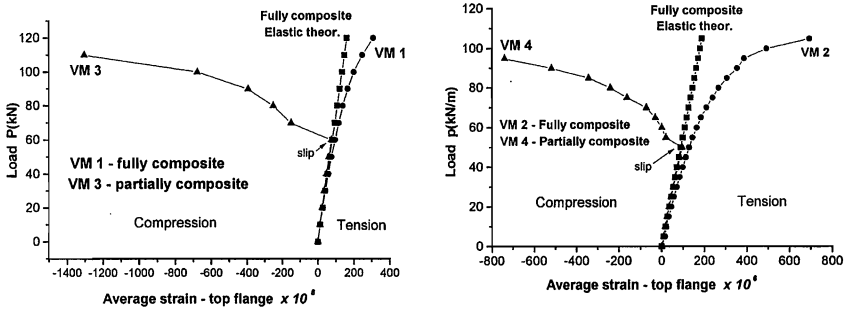**Fig. 9** – *Load versus strain curves: bottom flange*

Fig. 10 – *Load versus strain curves: top flange*

CONCLUSIONS

Based on the experimental analysis developed in this study, one can conclude the following:

The isolated steel beams presented a failure mode of flange local buckling, as foreseen by the theoretical analysis. In the composite beams, the presence of the concrete slab prevented the occurrence of this phenomenon, showing a failure mode of excessive strain of the steel in the fully composite beams (VM1 and VM2) and rupture of the concrete close to the connectors in the partially composite beams (VM3 and VM4).

The degree of interaction does not influence the structural behavior of the beams in the initial stage of the tests, since the shear flow was resisted by the adherence at the steel-concrete interface. After slip, the shear flow was resisted by the connectors and the partially composite beams presented a sudden drop in stiffness (see figures 8 and 9). This phenomenon was more marked in the beam with concentrated load (VM3), since the shear flow is constant in the regions between the supports and the points where force is applied.

The difference in behavior as a function of the degree of interaction can be clearly seen from the strains in the top flange of the beam (see figure 10). In the case of fully composite beams, the tension in the top flange indicated that the neutral axis remained located in the slab. For partially composite beams, the slip altered the distribution of stresses in the section, with compression occurring in the top flange.

For all the composite beams tested, the experimental values of the ultimate moment were, in average, 30% above the theoretical values obtained based on the initiation of yielding of the section, and 9% below the plastic bending moment of section (see table 4).

It is recommended, in design, that the nominal flexural strength be determined based on the initiation of yielding of the transformed section, owing to the inability of the cold-formed steel shapes generally used to form plastic hinges. To assess displacements it is recommended to adopt the moment of inertia of the transformed section in the case of fully composite beams (I_{tr}), and the reduced moment of inertia calculated by expression (6) in the case of partially composite beams (I_{eff}).

ACKNOWLEDGEMENT

APPENDIX A - REFERENCES

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APPENDIX B - NOTATION

A	= cross-sectional area of steel
A	= elongation (in 50mm)
a	= effective concrete slab thickness of the composite section
b	= effective concrete flange width of the composite beam
C	= compressive force in the concrete slab
C'	= compressive force in the steel beam
d_c	= concrete slab depth
d_s	= steel beam depth
E	= modulus of elasticity of steel (= 203,000MPa)
E_c	= modulus of elasticity of concrete
$E_{c,28}$	= modulus of elasticity of concrete at 28 days
f_c	= specified compressive strength of concrete
$f_{c,28}$	= compressive strength of concrete at 28 days
f_y	= yield strength of virgin steel
f_u	= tensile strength of virgin steel
h	= clear distance between flanges
h_F	= steel deck depth
I_{eff}	= transformed moment of inertia of the section in the partially composite beams
I_s	= moment of inertia of the steel section alone
I_{tr}	= transformed moment of inertia of the section in the fully composite beams
M	= beam bending moment
M_{el}	= elastic bending moment
M_n	= nominal flexural strength
M_{pl}	= plastic bending moment
M_u	= bending moment at failure (experimental)
n	= modulus of elasticity ratio, E/E_c
Q_n	= nominal strength of connector
T	= tension force in the steel beam
t_w	= web thickness
V_h	= total horizontal force transferred by the shear connections
y	= distance from top flange of steel section to plastic neutral axis
y_c	= distance from top flange of steel section to compressive force C'
y_t	= distance from bottom flange of steel section to tensile force T
y_{top}	= distance from top edge of composite section to CG of the composite section
y_{bottom}	= distance from bottom edge of composite section to CG of the composite section
y_1	= distance from top flange to CG (center of gravity) of steel section
ϕ_b	= resistance factor for bending
σ_c	= maximum bending stress in the concrete
σ_s	= maximum bending stress in the steel