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Reliability-Based Evaluation of Design Guidelines for Cold-Formed Steel-Concrete Composite Beams

This paper presents an analysis of design guidelines for steel-concrete composite beams, formed by concrete-filled cold-formed steel sections. The study is based on experimental results for connector resistance (push-out) and for four full-scale beam bending tests. The accuracy of analytical design equations is evaluated by comparing their predictions with experimental results. Model bias and model uncertainty of analytical design equations are evaluated. The uncertainty in design variables (steel and concrete resistance, dead and live loads, model errors) is taken into account, and reliability index of code-compliant beams is evaluated. Results show that the models for shear connector and for beam bending resistance are fairly accurate, and represent very little contribution to problem uncertainty and failure probabilities. Results show that for practical beam lengths, full material interaction is guaranteed, and failure is dominated by bending. Reliability indexes of the order of 2.2 to 2.8 are obtained, reflecting reliability of the design procedures studied. These values are low, in comparison to target reliability levels of 3.0 used in code calibration, and should be interpreted carefully in future code revisions.
Keywords: *steel structures, steel-concrete composite beams, thin-walled steel sections, structural safety, non-linear finite element analysis*

Introduction

Lightweight composite beams are becoming increasingly popular in recent years due to economic and mechanical advantages over more usual construction techniques. Composite steel-concrete beams have been increasingly applied in construction of buildings and bridges, in part due to development of large amounts of theoretical and experimental investigations. However, existing research addresses mainly hot-rolled and welded steel shapes. In terms of cold-formed steel sections, research and theoretical results are still needed. This paper addresses behavior and design procedures for concrete-filled thin-walled open steel-beam sections.

Modern architecture includes aesthetical, economical and mechanical structural requirements. The incorporation of steel-concrete composites in building construction provides new possibilities of balance between these requirements.

Open steel-beam sections allow easy casting of in-fill concrete, avoiding use of temporary formwork. Steel acts as formwork at construction stage and as reinforcement when in service. The fabrication process is very simple. In-fill concrete is less likely to be affected by adverse conditions during construction. The concrete does not need to be of high strength, as its main purpose is to prevent local buckling of the metal sheeting.

In Brazil, large availability of steel sheets favors use of steel-concrete composite systems in small and mid-height buildings. However, limited experience with this construction technique still defers broader application.

To assess the feasibility of practical construction of various types of thin-walled composite beams, detailed questionnaires were sent to two major construction and manufacturing industries, as well as to practicing engineers and clients. The main favorable characteristic identified in these questionnaires was the high beam strength, despite complexity in fabrication. Responses indicated that this form of construction has great potential as precast units.

Nomenclature

a = neutral axis position, cm
A_a = steel shape cross-section area, cm²
b = slab effective width, cm
b_f = steel shape width, mm
C_c = compression force on slab, kN
d = steel shape height, mm
d_1 = isolated steel shape center of gravity, mm
E = electric strain gage
E_c = concrete elasticity modulus, MPa
E_{cs} = concrete secant elasticity modulus, MPa
E_s = steel elasticity modulus, MPa
f_{cm} = concrete compression resistance, MPa
f_{ck} = concrete characteristic compression resistance, MPa
f_y = steel yield strength, MPa
h_f = distance between neutral axis if interaction is lost, mm
P_F = probability of failure, dimensionless
Q_{Rd} = shear connector resistance, MPa
t_c = slab height, cm
t_{cs} = shear connector thickness, mm
t_w = steel shape thickness, mm
T_a = tension force on steel shape, kN

Greek Symbols

α = sensitivity factor; curve parameter, dimensionless
β = reliability index, dimensionless
β_{vm} = connection rotation capacity given as 1.0, dimensionless
γ = load and resistance factors, dimensionless
γ_{a1} = steel safety factor given as 1.1, dimensionless
γ_{cs} = connector safety factor given as 1.25, dimensionless
Φ = cumulative standard normal distribution

Subscripts

0   relative to initial condition
d   relative do design value
D   relative to dead load
i   vector index
L   relative to live load
n   relative to nominal value
R   relative to resistance
S   relative to load

Experimental Investigation

This paper is based on the experimental analysis of four full-scale simply-supported steel-concrete composite beams (Chaves, 2009). Push-out tests to study connector resistance were also performed, but are described later in the article. A universal servo-hydraulic *INSTRON* testing system, with a strength capacity of up to 500 kN was used. Beams were observed, instrumented and measured for structural behavior, resistance, stiffness and collapse modes. Beams were tested until failure. Geometric and material details of tested beams are presented in Tables 1 and 2. Six samples were cut from the steel shape produced with ASTM A370 standards, and three samples of concrete were taken for each beam to obtain material properties.

Table 1. Geometrical details of tested thin-walled composite beams.

Specimen	Steel Shape (mm)	Length (mm)	Concrete Slab (mm)
A1	200 x 80 x 20 x 3.17	2850	700 x 100
A2	200 x 80 x 20 x 3.14	2850	700 x 100
B1	200 x 80 x 20 x 3.05	2850	700 x 100
B2	200 x 80 x 20 x 3.08	2850	700 x 100

Table 2. Material details of tested thin-walled composite beams.

Specimen	f_y (MPa)	E_s (MPa)	f_{cm} (MPa)	E_{cs} (MPa)
A1	282	205332	23.68	21977
A2	285	216365	24.88	23265
B1	277	199748	26.78	25184
B2	288	223659	23.32	21985

The beams were instrumented as sketched in Fig. 5, with the intention of measuring vertical displacements and specific strains. A *KYOWA* displacement transducer, with sensibility of 0.001 mm, was used to measure vertical displacements. Strain gauges and transducer were placed at mid span.
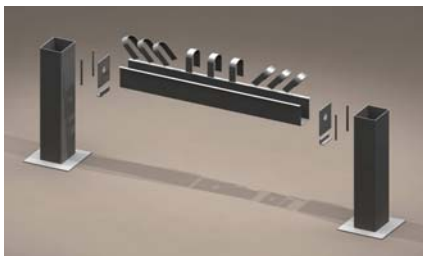
**Figure 1. Representation of the studied constructional system.**

Loading was applied in increments. At each increment strains and central deflection were recorded by a computerized VISHAY data acquisition system (*SYSTEM 5000*). The overall behavior of the beam, including failure modes, cracking and strains in steel shapes were observed during the entire loading history. A panoramic view of the test set-up is shown in Fig. 2 and in more detail in Fig. 5.


Figure 2. Test set-up.

Non-Linear Finite Element Analysis

A tree-dimensional numerical model of the tested beams was also constructed, in order to provide a three-way comparison with analytical and experimental results (Chaves, 2009). The analysis was conducted using the *ANSYS* (2006) program. The beam was modeled using *shell* elements (Fig. 3) and considering material and geometrical non-linearities.
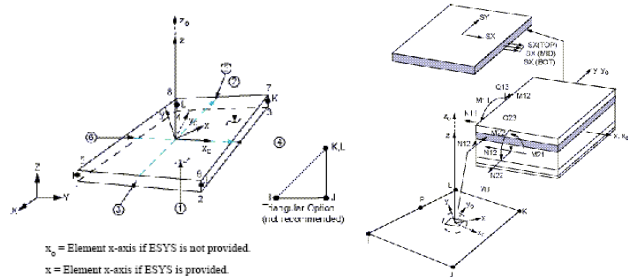
Figure 3. ANSYS SHELL 181 finite elements.

The model consisted of two sets of finite element groups: one set modeling the concrete slab and one set representing the steel shape. The model was meshed so that interface nodes for the two materials coincided (merge of coincident nodes). Forces were applied so as to reproduce the static loading of the experimental setting. Figure 4 shows the finite element model used in the numerical analysis.
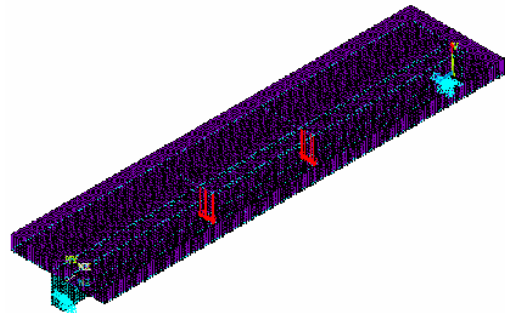
Figure 4. Numerical model of the steel-concrete composite beam.

For the steel, a multi-linear elastic-plastic material model was used, with isotropic hardening. For modeling of concrete, a cast-iron ANSYS material model was used. This model combines two non-linear materials in the same element or group of elements, allowing any element of the concrete slab to be in tension or compression, simultaneously.

Mesh refinement was adopted by taking into consideration the computational effort and accuracy of the results. The non-linear system of equations was solved using the *full Newton-Raphson* scheme, characterized by continuous updating of the tangent stiffness matrix in each interaction.

Design Guidelines (ABNT NBR 8800:2008)

Design guidelines for steel-concrete composite beams are provided in Brazilian code ABNT NBR 8800:2008 – *Design of steel and steel-concrete composite structures for buildings*. Following these guidelines, beams must be verified for three limit states: bending resistance; failure of shear connectors (or loss of interaction between steel and concrete) and cross-section shear failure. The following conditions apply to the beams studied in this paper:
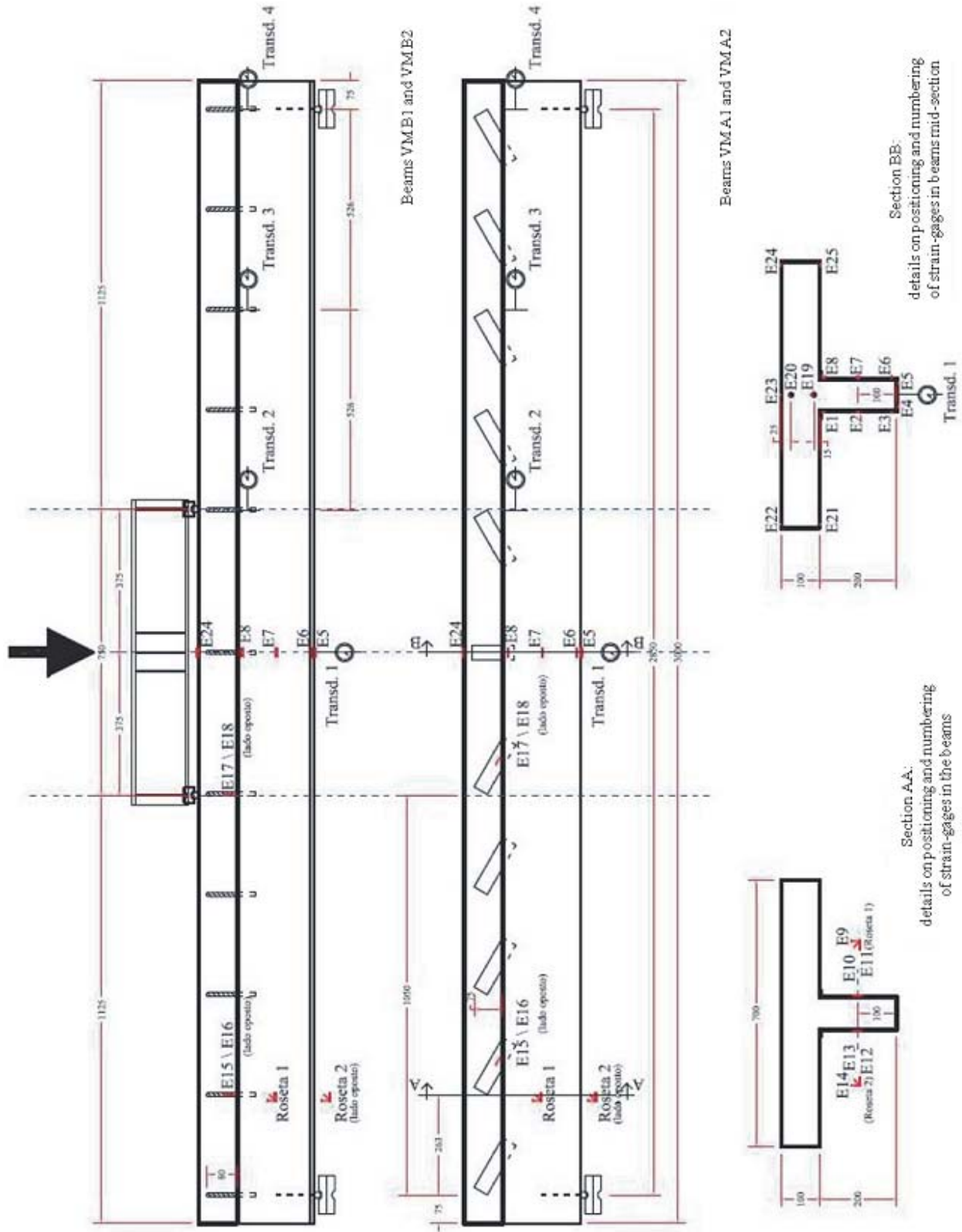
Figure 5. Composite beams in detail, including instrumentation and loading (Chaves, 2008).

- simply supported beams with solid concrete slab;
- steel shape used as formwork during construction: hence filled with same concrete used in slabs;
- effective slab on each side of beam taken as 1/8 of beam length, measured between supports;
- full interaction between steel and concrete;
- neutral axis positioned within the concrete slab.

For full material interaction, the resistance and number of shear connectors must be such that:

$$\sum Q_{Rd} \geq A_a \cdot f_{yd} \tag{1}$$

where Q_{Rd} is the resistance of each shear connector, A_a is steel cross-section area and f_{yd} is the design strength of steel. Equation (1) means that full interaction between materials will be maintained by shear connectors even if the steel shape starts to yield. Shear connectors play a vital role in the performance of concrete-filled steel sections, as they ensure full interaction between materials. Shear connectors used elsewhere were adapted for use in cold-formed steel shapes (Chaves, 2009). Following ABNT NBR 8800:2008, the individual resistance of a shear connector is given by:

$$Q_{Rd} = 0,3 \cdot \frac{t_{cs} \cdot b_f}{\gamma_{cs}} \cdot \sqrt{f_{ck} \cdot E_c} \tag{2}$$

The condition of neutral axis within concrete slab is fulfilled if the compression force in concrete slab is greater then the tension force in steel shape. This is given by:

$$0,85 \cdot f_{cd} \cdot b \cdot t_c \geq A_a \cdot f_{yd} \tag{3}$$

With these conditions, binary forces can be calculated as:

$$C_{cd} = 0,85 \cdot f_{cd} \cdot b \cdot a \tag{4}$$

$$T_{ad} = A_a \cdot f_{yd} \tag{5}$$

From Eqs. (5) and (3), the position of the neutral axis is found as:

$$a = \frac{T_{ad}}{0,85 \cdot f_{cd} \cdot b} \leq t_c \tag{6}$$

The bending resistance can be evaluated by:

$$M_{Sd} \leq M_{Rd} \tag{7}$$

$$M_{Rd} = \frac{\beta_{vm} \cdot T_{ad}}{\gamma_{a1}} \cdot \left(d_1 + h_F + t_c - \frac{a}{2} \right) \tag{8}$$

where M_{Sd} is the design "load" moment and M_{Rd} is the design resistant moment.

Cross-section shear resistance also has to be evaluated. For the beams studied herein, and considering no local buckling effects, shear strength is given only by steel shape resistance, following Eqs. (9) and (10):

$$V_{Sd} \leq V_{Rd} \tag{9}$$

$$V_{Rd} = \frac{1,20 \cdot d \cdot t_W \cdot f_y}{\gamma_{a1}} \tag{10}$$

where V_{Sd} is the design value of shear load and V_{Rd} is the design shear resistance.

Bending Resistance and Model Error

Figure 6 shows final configuration and collapse mode of tested and numerically simulated beams. The four beam specimens tested failed by bending. Table 3 compares the maximum bending strength obtained via testing, numerical analysis and analytical design guidelines. It can be seen in Table 3 that analytical and numerical results showed very good agreement. The four experimental results showed some reserve strength, in comparison to analytical and numerical predictions.
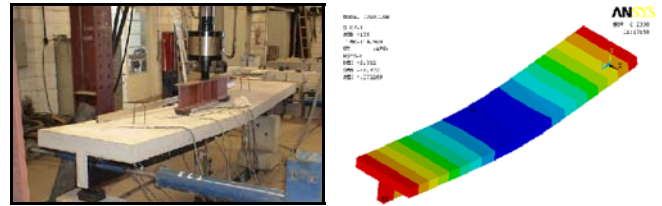

Figure 6. Final aspect and failure mode of studied beams.

Table 3. Maximum experimental, analytical and numerical strengths (kN).

beam	experiment	analytical	numerical	exp./anl.	exp./num.
A1	193.12	163.24	164.97	1.18	1.17
A2	181.99	163.16	164.97	1.11	1.12
B1	185.93	163.24	164.97	1.13	1.12
B2	185.06	163.16	164.97	1.13	1.12

With the objective of comparing the ultimate bending resistance of the beams, as calculated by ABNT NBR8800:2008 design guidelines, with experimental results, a model error (M_e) variable is introduced:

$$M_e^{bending} = \frac{M_R^{experimental}}{M_R^{analytical}} \tag{11}$$

In Equation (11), the analytical resistance is evaluated from Eq. (8), but using mean values for material resistances and unitary safety factors. Hence, this is the non-conservative prediction of bending resistance. Four samples of the model error random variable are obtained from experimental results, as shown in Table 3. Samples are adjusted to a Log-normal distribution, resulting in a mean of $E[M_e^{bending}] = 1.1325$ and c.o.v. = 0.025. The variance is a measure of the random error of the model, whereas the mean is known as bias factor. A bias larger than one represents a conservative resistance model, that is, the resistance evaluated via model is smaller than the actual resistance.

Connector Resistance and Model Error

Shear connectors are fundamental for the proper performance of concrete-filled steel sections, as they ensure full interaction between materials. The resistance of shear connectors used in the tested beams was evaluated from experimental push-out tests, following guidelines of EUROCODE 4:2001. Experimental results and model errors are considered here. Details of the experimental setting are found in Chaves (2009). The model error for shear connector resistance is evaluated as:

$$M_e^{connector} = \frac{Q_R^{experimental}}{Q_R^{analytical}} \qquad (12)$$

In Equation (12), the analytical resistance is evaluated from Eq. (2), but using mean values for material resistances and unitary safety factors. Two types of shear connectors (flat bar arc shape section and round bar arc shape section) where tested in three-replicate tests. Results of push-out tests are presented in Table 4.

Table 4. Experimental results of push-out tests for connector resistance.

Type	Exper. (kN)	Analytical (kN)	$M_e^{connector}$=Exp./Anl.
Flat bar arc	43.00	44.03	0.98
	47.03	44.03	1.07
	41.95	44.03	0.95
Round bar arc	31.28	34.13	0.92
	35.05	34.13	1.03
	35.65	34.13	1.05
Mean	---	---	0.9976
C.O.V.	---	---	0.0586

Results in Table 4 show that the design equation for connector resistance presents no model bias, since $E[M_e^{connector}] \approx 1.0$. The coefficient of variation of the model for connector resistance is also quite small at C.O.V. $\approx$ 0.06. Assuming a Normal distribution for the model error statistics in Table 4, a safe design factor for connector resistance would be obtained as $0.9976 + 1.6448 \cdot 0.0586 = 1.0939$ (for a 95% confidence level). The recommended safety factor for connector resistance, in Eq. (2), is $\gamma_{cs} = 1.25$, hence well above the value just found.

Structural Reliability Analysis

The model error for bending resistance is a measure of our ability (or inability) to predict the strength of a steel-concrete composite beam in exact form. This measure is given by model error bias and variance, as described below, as well as by some probability distribution function. In a similar way, our inability to exactly predict the strength of materials or loadings on a structure can be expressed by using random variable or random process models to describe these variables.

Structural reliability theory can be used to properly represent problem uncertainty, and to study the effects of uncertainty in structural performance. Perhaps the most striking effect of uncertainty is the possibility (measured as a probability) of undesirable structural response. And since most random variable or random process distributions models are unbounded, it turns out that this failure probability can be made as small as possible, but cannot be zero.

Any structural system has to fulfill a number of requirements, many of which related to safety. Safety requirements are directly related to structural failure modes, which are generally formulated into design equations. The same failure modes can be formulated in terms of limit state equations. Given a vector of random resistance or loading parameters $\mathbf{X}$, a general limit state equation $g(\mathbf{X})$ is written in such a way that it divides the domain of $\mathbf{X}$ in safety (D_S) and failure domains (D_f):

$$D_f = \{\mathbf{X} \mid g(\mathbf{X}) \leq 0\}$$
$$D_s = \{\mathbf{X} \mid g(\mathbf{X}) > 0\} \qquad (13)$$

The failure probability can then be computed as the probability that the problems variables belong to the failure domain:

$$P_f = P[g(\mathbf{X}) \leq 0] = \int_{D_f} f_\mathbf{X}(\mathbf{x})\,d\mathbf{x} \qquad (14)$$

where $f_\mathbf{X}(\mathbf{x})$ is the joint probability density function of the problems random variables. Computation of the failure probability hence amounts to evaluating a multi-dimensional integral over the failure domain.

Solutions of Eq. (14) involve approximations for the joint probability density function (integrand) as well as approximations of the failure (or integration) domain. Different solution methods correspond to different levels on these approximations. The First Order Reliability Method (FORM) allows consideration of any form of probability distribution and of correlation between the random variables, and involves an approximation of the integration domain by a hyper-plane (Beck, 2008; Melchers, 1999). The hyper-plane approximation leads to the well-know result:

$$P_f = \Phi(-\beta) \qquad (15)$$

where β is known as the reliability index and Φ is the cumulative standard Gaussian distribution. This is evaluated as the distance between the most probable failure point and the mean of the problems random variables. FORM is used in this paper to evaluate the reliability of steel-concrete composite beams.

In the very particular case or two Gaussian random variables R and S (R for resistance and S for solicitation), and a linear limit state function of the form $g(\mathbf{X}) = R - S = 0$, the reliability index is obtained with μ (mean) and σ (standard deviation) as:

$$\beta = \frac{\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2}} \qquad (16)$$

In the general case of any number of variables, with non-Gaussian distributions and failure probability given by Eq. (14), the reliability index is defined as:

$$\beta = -\Phi^{-1}(P_f) \qquad (17)$$

In this paper, the structural reliability analysis was performed within the *Mathematica* environment. A special purpose structural reliability module was developed (Beck, 2007). Figure 7 shows a flowchart of the algorithmic procedures required in the solution of the problem. The flowchart follows guidelines presented in (Beck and Doria, 2008).
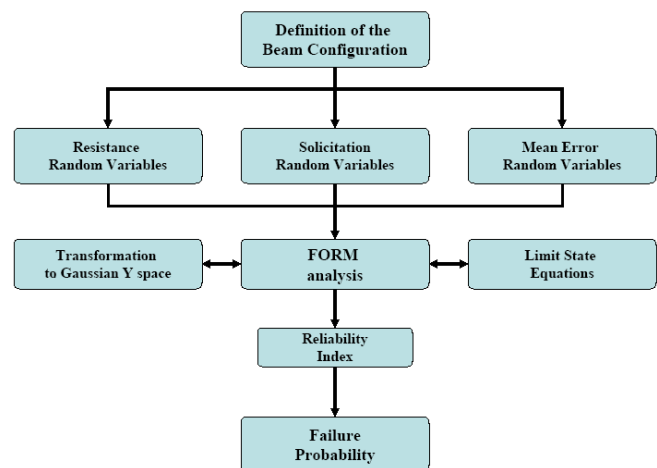
Figure 7. Flowchart of structural reliability analysis.

Limit State Equations

Structural safety requirements can be formulated in terms of limit state functions, which are generally (but not always) related to design equations. Each limit state function corresponds to one (possible) failure mode of the structure or structural system.

For steel-concrete composite beams, three limit state functions are considered in this paper. For bending resistance, one has:

$$g_1(\mathbf{X}) = M_e^{bending} \cdot M_R(A_a, f_y, f_c) - M_S(D, L) = 0 \qquad (18)$$

where M_R is the resisting moment and M_S is the moment caused by external loads. The analytical resisting moment M_R is evaluated from Eq. (8), with material resistance modeled as random variables (i.e., random variables instead of characteristic values).

The limit state for shear connector resistance (condition of full interaction) is:

$$g_2(\mathbf{X}) = M_e^{connector} \cdot Q_R(f_c, E_c) - Q_S(D, L) = 0 \qquad (19)$$

where Q_R is shear connector resistance and Q_S is shear load.

For cross-section shear resistance, the limit state is:

$$g_3(\mathbf{X}) = C_R(A_a, f_y) - C_S(D, L) = 0 \qquad (20)$$

where C_R is cross-section shear resistance and C_S is cross-section shear caused by external loading.

Resistance Random Variables

The uncertainty in material properties can be represented by means of random variables. This includes the assumption of a particular probability distribution model. In general, it is the response to static and time dependent mechanical loading that matters for structural design. However, the response to physical, chemical and biological actions is also important as it may affect the mechanical properties and behavior. Mechanical models should be based on (standardized) tests, representing the actual environmental and loading conditions as good as possible (JCSS, 2001). Table 5 shows the parameters and distributions of random resistance variables considered in this paper.

Table 5. Statistics of random resistance variables.

Variable	Symbol	Distribution	E[.]	c.o.v.	Unit
Steel cross-section area	A_a	Normal	15.54	0.03	cm^2
Steel yield stress	f_y	Log-normal	28.26	0.07	MPa
Concrete ultimate stress	f_c	Log-normal	2.47	0.13	MPa
Concrete elast. modulus	E_c	Log-normal	2491	0.05	MPa
Model error for bending	$M_e^{bending}$	Log-normal	1.132	0.025	-
Model error for connector	$M_e^{connector}$	Normal	0.9976	0.059	-

Load Random Variables

The design value of beam bending strength is given as:

$$M_{Sd} \leq M_{Rd} \qquad (21)$$

To evaluate reliability of steel-concrete composite beams in actual condition of service, the dead load (D) and live load (L) variables are incorporated. The design value of bending load (F_D) is given by the factored sum:

$$F_D = \gamma_D \cdot D_N + \gamma_L \cdot L_N \qquad (22)$$

For dead loads in steel structures, ABNT NBR8800:2008 gives $\gamma_D = 1.35$. When variable actions or live loads are considered, the factor $\gamma_L = 1.5$ is recommended. Equations (21) and (22) are not sufficient to obtain nominal values of dead load (D_N) and live load (L_N). A proportionality relation between these loads must be considered. Five values of the ratio L_N/D_N are considered in this paper: $L_N/D_N = \{0.5; 0.75; 1.0; 1.5; 2.0\}$. Statistics of dead and live loads are taken from Ellingwood (1994) and are presented in Table 6.

Table 6. Load random variables distribution and parameters.

Variable	Symbol	Distribution	E[.]	c.o.v.
Dead load	D	Normal	$1.05D_N$	0.10
Live load	L	Gumbel	$1.00L_N$	0.25

Reliability Analysis Results

A total of 30 representative beam configurations were analyzed, by varying beam lengths (100 cm, 500 cm and 1000 cm), the shear connector type (flat and round arc connectors) and the L_N/D_N ratios as given earlier. Each beam cross-section was designed according to code rules presented earlier, and considering the respective beam spans. Design guidelines should guarantee full material interaction up to the bending limit. Figures 8 to 10 summarize the results obtained for the beam configurations studied.
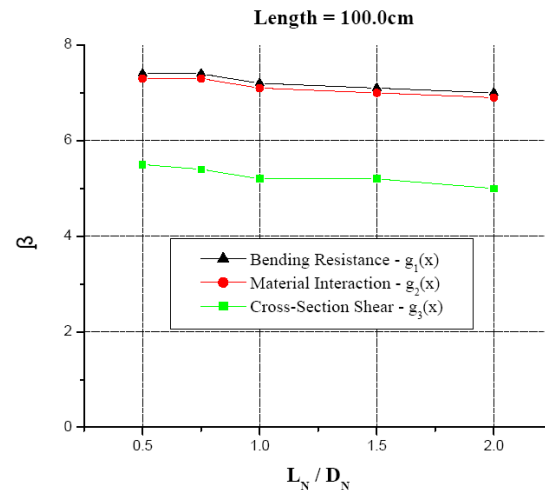

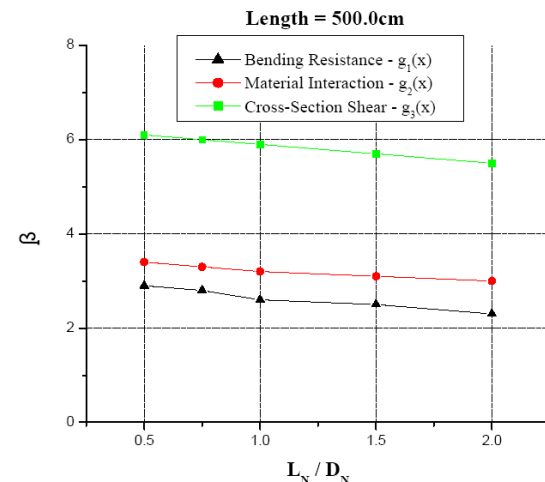**Figure 8. Reliability index for different ratios of L_N/D_N.**

Figure 9. Reliability index for different ratios of L_N/D_N.
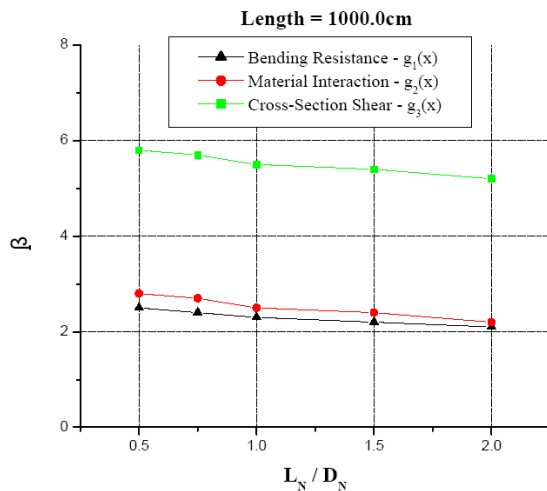
Length = 1000.0cm

Figure 10. Reliability index for different ratios of L_N / D_N.

One can see that when beam length is relatively small, failure probabilities are dominated by shear of the cross-section. For longer beams, the bending limit state equation dominates. It is worth nothing that reliability indexes for material interaction and bending resistance limit states are closely spaced in all situations. This is due to the number of connectors being calculated based on the maximum bending resistance, which is done to guarantee full interaction between the materials. The interaction limit state is slightly more conservative than bending resistance, since the required number of connectors is always rounded to the largest integer. It is also noted that for longer beam spans, the distance between the two curves is reduced.

Reliability indexes are high for the short beam, but significantly smaller for the longer spans. The beam span of 1000 cm is unusual in actual design, and is considered here only for illustration purposes. Results for the 500 cm span are closer to what one would expect in practice.

For the 500 cm beam, reliability indexes vary between 2.8 and 2.2. In comparison, a target reliability index of 3.0 was used in calibration of American design codes (Ellingwood and Galambos, 1982), for combinations involving dead and live loads. The additional safety margin provided by a larger-than-one model error bias (1.13) is already reflected in these results. Without this conservative bias, reliability indexes would be smaller. Hence, reliability indexes obtained for the 500 cm beam are below target values used in code calibration. This result should be carefully interpreted in future revisions of ABNT NBR8800. Reliability index values of 2.8 to 2.2 encountered here are compatible with results obtained for I section steel columns (Beck and Doria, 2008) and for steel-concrete composite columns (Beck et al., 2009) using the same design code ABNT NBR8800.

Important sub-products of a FORM reliability analysis are the sensitivity coefficients (α), which give the contribution of individual random variables in failure probabilities. In the transformed space Y, sensitivity coefficients are simply evaluated as:

$$\alpha = \frac{\nabla g\left(y^*\right)}{\left\|\nabla g\left(y^*\right)\right\|} \tag{23}$$

Sensitivity coefficients vary in the range {-1.0; 1.0}. Negative values represent "load" variables, since an increase in these variables produces a decrease in the limit state function; positive sensitivity coefficients represent "resistance" variables. Table 7

presents the random variables coefficients. The absolute value of the sensitivity coefficient is a measure of the contribution of the random variable towards failure probabilities.

Table 7. Sensitivity coefficients of random variables for $L_N / D_N = 1.0$.

Random Variable	$(\alpha^2) - g_1(x)$	$(\alpha^2) - g_2(x)$	$(\alpha^2) - g_3(x)$
M_e	0.0412	---	---
A_a	0.0578	---	0.5761
f_y	0.0189	---	---
f_c	0.0064	0.3352	---
E_c	---	0.2829	---
D	-0.0370	-0.0161	-0.0167
L	-0.8394	-0.3656	-0.3781

Results presented in Table 7 show that live load uncertainty dominates failure in the bending mode. In comparison, the contribution of design model uncertainty (model error) is marginal. This serves as a positive validation of the design procedures.

The material interaction failure mode depends almost equally on the uncertainties in yielding stress, concrete elasticity modulus and live load. Uncertainty in steel cross-section area dominates shear failure of the cross-section, followed by uncertainty in live loads. The important contribution of live load uncertainty to failure probabilities is known to be a consequence of high variability in this load. This result is also a consequence of representing live load random variable by an extreme value distribution, which has higher probability content at the upper tail when compared with the normal distribution of the dead load.

Concluding remarks

The flexural bending strength of composite beams, formed by concrete-filled cold-formed steel sections, was investigated in this paper. Experimental, numerical and reliability analysis where used to evaluate guidelines given in NBR8800 for the design of steel-concrete composite structures. Analytical design procedures showed very good agreement with results of numerical (F.E.) analysis in prediction of beam bending strength. Experimental test results for beam bending revealed an additional safety margin (model bias) of 13% (on average), and very small model uncertainty (2.5%). Experimental push-out results for connector resistance showed very good agreement with analytical design predictions, with no bias and small uncertainty (5.6%)

For medium or long beam spans, the design procedure of NBR8800 guarantees full interaction between steel and concrete, and failure is dominated by bending. Reliability indexes for typical beam spans obtained in this study ranged from 2.2 to 2.8, depending on load ratios. These values are below target values used in code calibration ($\beta = 3.0$), and should be carefully interpreted in future code revisions.

Acknowledgements

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