DESIGN OF BUCKLING RESISTANCE OF COMPRESSED HSS-CHANNELS

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ABSTRACT

The use of high-strength steels (HSS) has been limited in building constructions. This is mainly because the regulations do not cover HSS steels. The design resistances of the presented calculation method for flexural and torsional-flexural buckling of channels, based mainly on Eurocode 3, are compared with the results of 24 own and 33 other compression tests. The columns of U, C and hat channels of thickness 1.7 - 6.0 mm cover yield strengths ranging from 220 to 610 N/mm^2 and also local buckling of flat and stiffened plate elements. It was observed that the given design method was always conservative and its accuracy was independent of the material yield strength.

INTRODUCTION

Both the Finnish design codes B7 [1] ja B6 [2] and the corresponding European ones EC3/Part 1 [3] and EC3/Annex A [4] concerning steel and thin-walled steel structures still do not cover HSS-steels. The calculation of the buckling resistance of cold-formed channels, including the local buckling of plates and global flexural or torsional-flexural buckling, is also shown in a different formulation.

This study forms part of a Finnish research programme, which studies the use high-strength steels in building constructions. This study has been financed by the Technology Development Centre in Finland (TEKES), Rautaruukki Oy and the Ministry of the Environment.

This article describes the bases of a design procedure, which is compared to the test results of 24 test columns compressed in the Technical Research Centre of Finland [5] and 33 test columns compressed in the University of Sydney [6]. The tests consist of 3 channel (U), 5 lipped channel (C) and 4 hat sections. The material yield strength is 220 - 550 N/mm^2 and the thickness 1.7 - 6.0 mm. The maximum width/thickness-ratio of the flat plates is 55 and the non-dimensional slenderness 1.36.

DESIGN METHOD

The design method described here is for the most part in accordance with the design codes B7 [1] and EC3/Part 1 [3]. These codes determine the elastic buckling force of a column on the basis of the gross cross-section instead of the effective area concept. However, these codes do not cover the torsional-flexural buckling of channels. Also, they do not give the reduction of the stiffener thickness which is due to the buckling of the lip of C or hat sections.

The elastic torsional-flexural buckling load and the reduction of the stiffener thickness is determined here in principle in accordance with EC3/Annex A [4], but the treatment of the stiffener is simplified. The results of the simplified method for torsional-flexural buckling given in the Finnish code B6 [2] are inaccurate. The reduction of the stiffener thickness presented here is more conservative than B6. The research note [5] compares the results of six different calculation models with the test results.

Buckling resistance N_{Rc}

According to B7, the buckling resistance of an axially loaded bar is determined using the equation

$$N_{Rc} = f_{ck} \cdot A_e/\gamma_m \qquad (1)$$

Code B7 uses a safety factor of $\gamma_m = 1.0$ for the material property, which is also used here (EC3/Part 1 gives $\gamma_m = 1.1$). A_e is the effective cross-sectional area.

Characteristic buckling stress f_{ck}

The characteristic buckling stress f_{ck} in B7 is determined with the aid of the non-dimensinal slenderness

$$\bar{\lambda} = \sqrt{(N_R/N_{el})} \qquad (2)$$

from eqs. (3) - (5). The factor α depends on the buckling curve used. On the basis of the test results, the use of the buckling curve C ($\alpha = 0.49$) is recommended. The Q-factor used in EC3 is 1 here.

$$f_{ck}/f_y = 1 \quad , \text{ when } \bar{\lambda} \le 0.2 \qquad (3)$$

$$f_{ck}/f_y = \beta - \sqrt{(\beta^2 - 1/\bar{\lambda}^2)} \quad , \text{ when } \bar{\lambda} > 0.2 \qquad (4)$$

$$\beta = \frac{1 + \alpha \cdot (\bar{\lambda} - 0.2) + \bar{\lambda}^2}{2\,\bar{\lambda}^2} \tag{5}$$

The resistance of eq. (2)

$$N_R = A \cdot f_y/\gamma_m \tag{6}$$

and the elastic buckling force N_{el} is based on the values of the gross cross-section.

The given principles are valid in the calculation of both flexural and torsional-flexural buckling resistances. In the case of channels, the torsional-flexural buckling is often determinant, which results in more complicated calculations than in the case of in-plane buckling. The elastic buckling forces for flexural buckling about x- and y- axes are

$$N_{elx} = \frac{\pi^2\, EI_x}{L_{cx}^2} \quad \text{and} \tag{7}$$

$$N_{ely} = \frac{\pi^2\, EI_y}{L_{cy}^2} \tag{8}$$

In the case of torsional-flexural buckling

$$N_{elFT} = \frac{1}{2\beta^*} \{(N_{elx}+N_{elT}) - [(N_{elx}+N_{elT})^2 + - 4\beta^* N_{elx} N_{elT}]^{1/2}\} \tag{9}$$

where

$$\frac{1}{\beta^*} = 1 + \frac{x_o^2}{i_x^2 + i_y^2} \quad ; i_x = \surd(I_x/A) \text{ ja } i_y = \surd(I_y/A) \tag{10}$$

$$N_{elT} = \frac{1}{i_x^2 + i_y^2 + x_o^2} \left(GI_v + \frac{\pi^2\, EI_w}{L_{cT}^2}\right) \tag{11}$$

The symbols used in eqs.(7)-(9) are according to Fig. 1:

E	- modulus of elasticity ($E = 210\,000$ N/mm^2)
G	- shear modulus ($G = 80\,000$ N/mm^2)
A	- gross cross-sectional area
I_x, I_y	- moments of inertia with respect to x and y axis
I_v	- torsion constant

I_w - warping constant
L_{cx}, L_{cy}, L_{cT} - buckling lengths and
x_o - the distance from the shear centre to the centre of gravity.

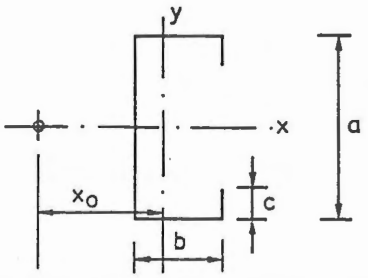

Fig. 1. Cross-sectional notations.

The lowest elastic buckling force of eqs. (7) - (9) gives the buckling resistance and the buckling mode.

Effective cross-sectional area A_e

Flat plates and stiffened plate sections have post-buckling capacity, which is expressed with the aid of the concept of effective widths and thicknesses. The effective cross-sectional area consists of the effective areas of the flat or stiffened plate sections. The effective widths of the flat plate elements (Fig. 2) are determinend from eqs. (13) - (14) with the aid of the non-dimensinal slenderness

$$\bar{\lambda}_p = \sqrt{(f_y/\sigma_{el})} \qquad (12)$$

$$\frac{w_e}{w} = 1 \qquad , \text{ when } \quad \bar{\lambda}_p \le 0.673 \qquad (13)$$

$$\frac{w_e}{w} = (1 - 0.22/\bar{\lambda}_p)/\bar{\lambda}_p \qquad , \text{ when } \quad \bar{\lambda}_p > 0.673 \qquad (14)$$

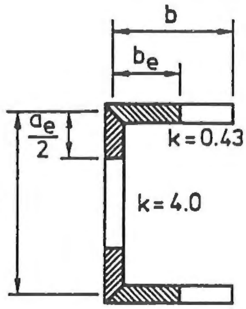

Fig. 2. The effective widths of the flat plate elements.

The elastic buckling stress of flat plate element

$$\sigma_{el} = k \cdot \frac{E\,\pi^2}{12\,(1 - \nu^2)} \frac{t^2}{w^2} \qquad (15)$$

where Poisson's ratio $\nu = 0.3$. The buckling coefficient $k = 4.0$ for webs and lipped flanges and $k = 0.43$ for unstiffened flanges and lips.

Because of the probable buckling of the stiffened edge of lipped and hat sections, the thickness of the edge (Fig. 3) is reduced with the aid of the non-dimensional stiffener slenderness

$$\bar{\lambda}_j = \sqrt{(f_y/E)} \cdot \left[\frac{(1 - \nu^2)[1 + 1.5(a/b)]}{I_j/A_j^{\,2}} \frac{b^3}{t^3}\right]^{1/4} \qquad (16)$$

The equation is based on the buckling of an indefinitely long bar on an elastic foundation. The foundation is composed of the web and flange. The moment of inertia I_j and area A_j are determined for a beam which is formed from the lip and half of the flange (Fig 3).

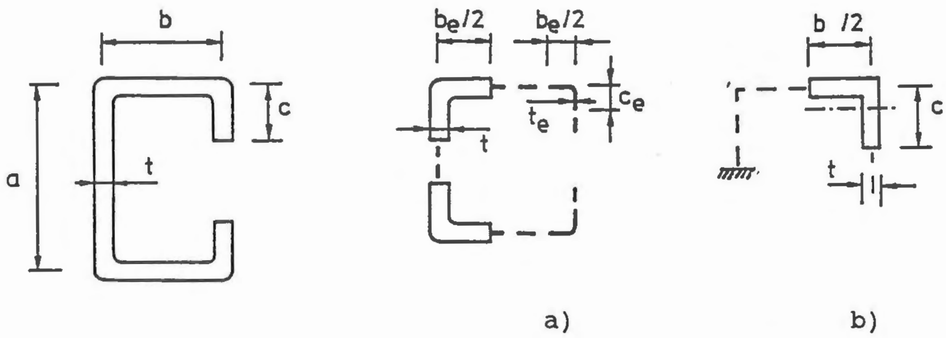

Fig. 3. Lipped channel and
a) the effective cross-section and
b) the beam section representing the stiffened edge.

The thickness of the stiffened area is reduced according to the buckling curve A (α = 0.21). When the stiffener slenderness of eq. (16) is used in eqs. (3) - (5), the reduced thickness is

$$\frac{t_e}{t} = \frac{f_{ck}}{f_y} \tag{17}$$

In the case of pure compression, the effective cross-sectional area may be determined using in eqs (12) and (16) the charasteric stress f_{ck} of eqs. (3) and (4) instead of the yield stress f_y. The tests give good reasons for the procedure and it offers an advantage in the case of slender columns. However, when, as is uaually the case, a compressive axial load acts together with a bending moment, the procedure has no grounds.

COMPARISON WITH THE TEST RESULTS

Tests

The cross-sectional dimensions of 57 tested columns are shown in Table 1. The values are average values measured from test specimens. The sections 1 - 6 are fabricated by roll-forming in the Rautaruukki Oy Toijala works and tested in the Laboratory of Structural Engineering of VTT. The sections 7 - 12 were fabricated by brake-pressing and they were tested in the University of Sydney [6]. During the tests, both ends of the specimen were fixed. The theoretical buckling length of the test columns is half of the specimen length in both flexural and torsional-flexural buckling.

The slenderness (eq. (2)) of the shortest specimens is less than 0.2 and that of the biggest ones 0.8 - 1.3. With the exception of sections 1 and 3, the torsional-flexural buckling is determinant in design.

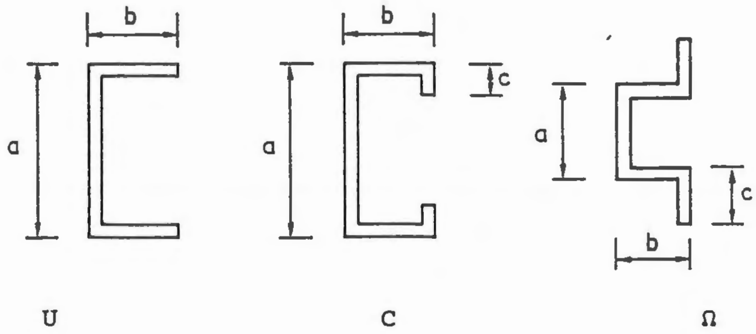

Fig. 4. Cross-sectional dimensions

Table 1. The dimensions and biggest slendernesses of the test sections.

No	Form (no)	Strength f_y [N/mm^2]	Dimensions [mm] a	b	c	t	$\bar{\lambda}_{max}$/mode
1	U (4)	609	127.0	40.6	-	5.98	1.1/flex.
2	U (4)	570	186.4	79.8	-	5.96	1.1/tors.-f.
3	U (4)	576	287.0	78.9	-	5.94	1.1/flex.
4	C (4)	565	200.2	79.3	25.7	3.95	1.1/tors.-f.
5	C (4)	558	130.1	129.3	23.2	4.89	1.1/tors.-f.
6	Ω (4)	533	111.9	150.0	53.5	5.10	1.1/tors.-f.
7	C (6)	404	91.9	70.7	14.7	1.67	0.9/tors.-f.
8	C (5)	229	91.7	71.1	14.6	2.00	0.7/tors.-f.
9	C (5)	493	90.4	67.8	15.0	2.40	1.0/tors.-f.
10	Ω (6)	392	92.7	80.5	14.6	1.67	1.1/tors.-f.
11	Ω (6)	221	93.2	81.2	14.6	1.97	0.8/tors.-f.
12	Ω (5)	496	87.4	90.5	15.2	2.38	1.3/tors.-f.

Test results

The deformations of a test series are shown in **Fig. 5.**

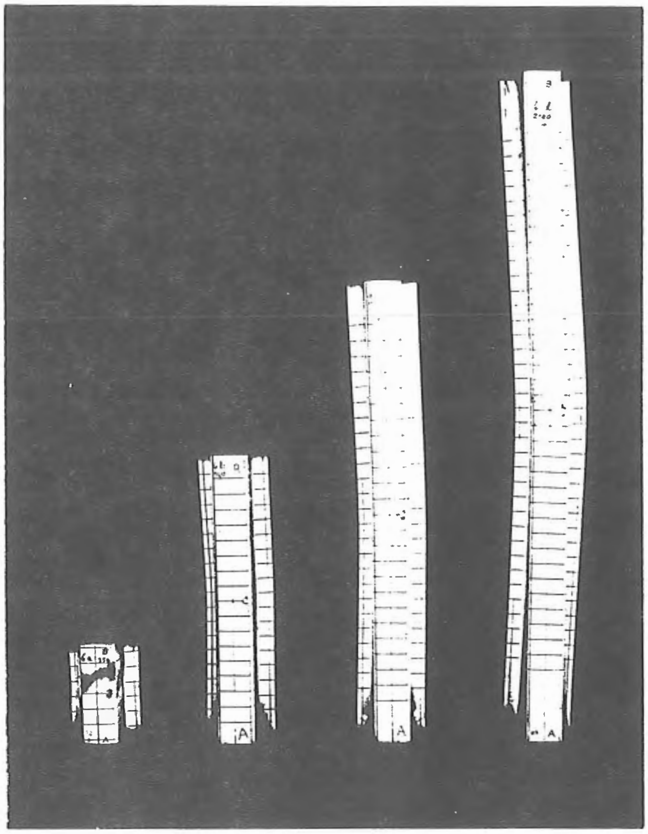

Fig. 5. The deformations of hat profiles (No 6) after testing.

The ratios of measured resistances to calculated ones are shown in Fig. 6. The figure also gives the results if the design resistance of eq. (1) is determined according to the gross cross-sectional area instead of the effective area.

The calculation here is based on the buckling curve B (α = 0.34) and therefore the design resistances are about 11 % higher than the above recommended curve C would give. In addition, this analysis uses the effective cross-sectional area, which is determined using in eqs (12) and (16) the charasteric stress f_{ck} instead of the yield stress f_y.

With the exception of section 1, the design method gives safe results. The maximum resistances in the tests are one third bigger than in the calculation. The accuracy of the design method does not seem to depend on either the type or the material of the channels.

Because the dimensions of section 1 are relative small compared to the thickness, it is very possible that the effect of the residul stresses due to cold-forming have in this case more effect than in the case of the other sections. Only when the design is made according to buckling curve D (α = 0.76), the accuracy correspond to the results of the other sections. Because of this exception, the use of buckling curve C is recommended instead of curve B; in special cases even buckling curve D may be considered.

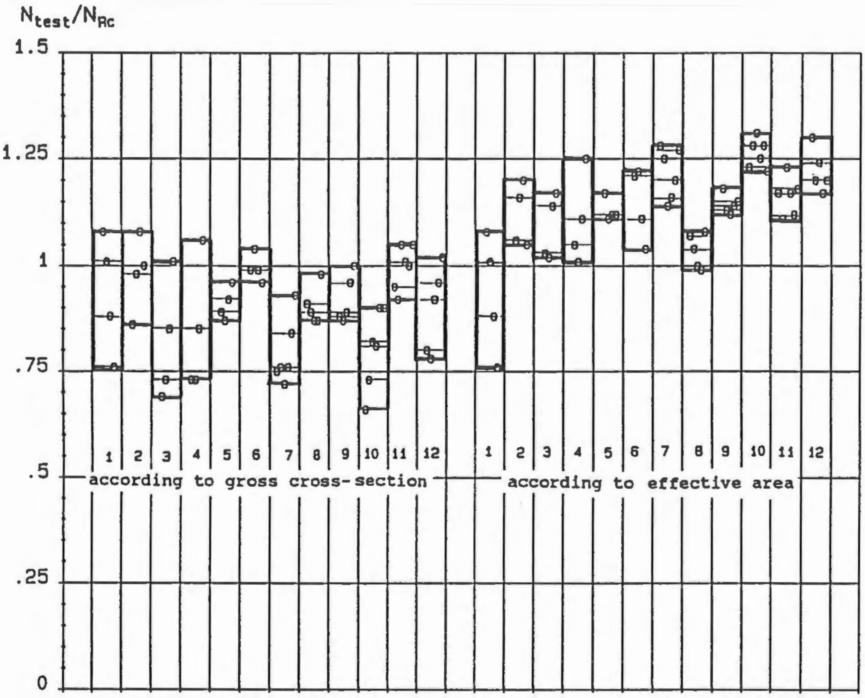

Fig. 6. Comparison of measured and calculated resistances.

CONCLUSIONS

On the grounds of the presented comparisons with the test results of U, C hat sections, which were made by cold-forming from both normal and high-strength steels, the given design method is valid in determining the buckling resistance. The accuracy of the design method was observed to be independent of the material yield strength. However, because of the residual stresses due to the cold-forming, special care must be taken in the case of relatively small but thick cross-sections.

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