RESISTANCE OF COLD-FORMED STEEL MEMBERS BY NEW EUROSTANDARD

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Structural Mechanics, Vol. 39
No. 2, 2006, pp. 3-21

SUMMARY

The paper deals with the new European standard EN 1993-1-3 for cold-formed steel structures. The purpose is to describe and to study using numerical examples the effects of some new clauses to design cases. The purpose of this paper is to look at the safety margin of the new European standard compared to tests available in the literature and to other codes. This study deals with only the members. The main scope of this study is to consider the cases when designing the cold-formed members which are not supported between supports, e.g. members of the truss, or are supported continuously at both sides along the members, e.g. members of the wall or floor elements. Finally, some conclusions are given based on the results of the study.

INTRODUCTION

In recent years, much research and development has been directed towards the use and behavior of cold-formed steel structures and structural elements (Pedreschi, Sinha, 1996). Cold-formed steel purlins, sheeting rails, steel deck flooring, and composite cladding panels have all become part of the standard language of building. Figure 1 illustrates typical structural assemblies of cold-formed members.

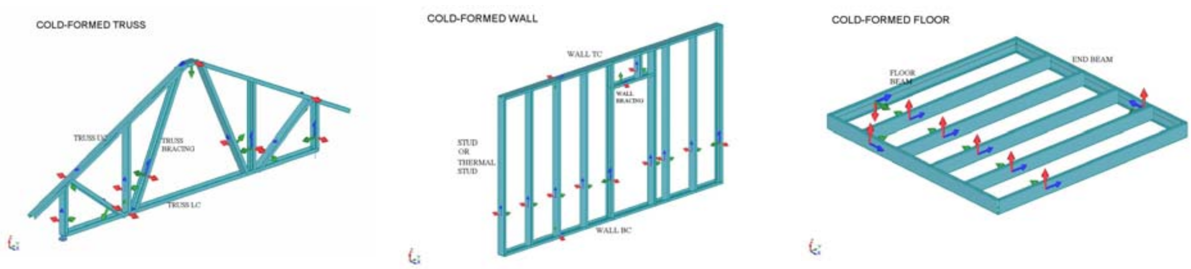

Figure 1. Cold-formed members in structures

The structural design is governed by codes of practice. In this paper the most novel draft of the new European standard (EN 1993-1-3, December 2004) for cold-formed steel structures is studied. This study deals with the axially loaded columns, bending and interaction of compression and bending. This part of Eurocode is not yet accepted officially by the member states of CEN for practical use when writing this paper, but it is believed that the methods presented will not change dramatically for the final voting.

The ultimate loads for the cases under consideration were compared to the failure loads observed in tests and calculated by the following codes, if found in the reference papers:

- American, AISI (1996),
- Australian/New Zealand, AS/ANZ (1996),
- European, ENV 1993-1-3 (1993),
- New European, EN 1993-1-3 (2004),
- North American Specification, NAS (2001),
- Canadian, S136 (1994).

This paper is a summary of the more detailed report (Heinisuo, Kukkonen, 2005). Most of the calculations are originating from the diploma thesis of the other author (Kukkonen, 2005).

In the new European standard there is stated, that the effect of rounded corners should always be taken into account when defining the cross-sectional properties. A method based on German DIN standard is used in this paper. It was found (Kukkonen, 2005) to give proper results for design purposes.

COMPRESSION OF COLD-FORMED MEMBERS

As an introduction consider the column with the channel profile presented in Figure 2.

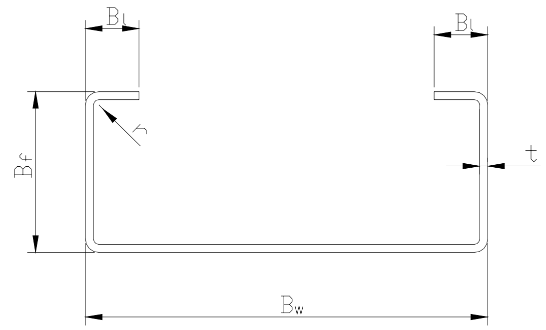

Figure 2. Cross-section of the channel and definitions of symbols

The test results and the analytical results originating form the three references are used for the comparisons:

- Tests by Lau, Hancock (1988), [1)] in Table 1,
- Test by Young, Rasmussen (1998) [2)],
- Analysis by Young, Yan (2002), Non-linear Abaqus Version 5.8 using thin shell elements S4R5, both material and geometrical non-linearities are taken into account [3)].

The analysis model [3)] was calibrated against test results and it can be regarded as a good reference for the failure loads. The results from that analysis are named as P_{test} in the following.

The column is supported by fixed supports at the both ends. Fixing means fixing against two displacements and four rotations including warping. The translational degree of freedom in the axial direction at the top of the column is free. Young and Rasmussen (1998) have stated, that requirement in considering the shift of the neutral axis due to local buckling (as stated in EN 1993-1-3) can be neglected with the fixed boundary conditions. If the load is acting at the centroid of the cross-section, then this case may be considered as a purely axially loaded member. The column lengths ranged from 280 mm to 3000 mm.

The cross-sectional dimensions have been measured in tests. The dimension t_{cor} is the base material thickness and $\sigma_{0.2}$ is the measured static 0.2 % tensile proof stress and it is used as the yield strength f_y. The material and the load factors are equal to one in these comparisons. The dimensions and properties presented in Table 1 are used. The measured values are mean values. Only three cases are presented here. More cases to all the examples can be found in the reference Heinisuo, Kukkonen (2005).

Table 1. Dimensions and properties of the channels

Case Name	*Web B_w (mm)*	*Flange B_f (mm)*	*Lip B_l (mm)*	*Thickness t_{cor} (mm)*	*Radius r (mm)*	*Yield Strength f_y (MPa)*	**Young's Modulus E (MPa)**
CH17 [1)]	91.9	70.7	14.7	1.67	0[*)]	393	210000[*)]
L36 [2)]	97.3	37.0	12.5	1.48	0.85	505	210000
L3.0W200 [3)]	200	80	15	3.0	0	505	210000
[*)] Estimated							

It must be noted, that the ratio of the ultimate strength and the yield strength does not fulfill in every case the recommended value 1.10 given in EN 1993-1-1, 2003, but fulfills the recommended value 1.05 given in EN 1993-1-12, 2004.

When calculating the buckling loads, the effective lengths for minor axis flexure, for major axis flexure and for torsion and warping were taken as one-half of the column length for the fixed-ended columns.

The effective area for compression and for bending is defined by taking into account the local buckling of the plate elements of the cross-section and the distortional buckling. The local buckling is taken into account using the well-known effective width method or alternatively new effective thickness method for outstand elements. The reduction equations of the widths or thicknesses are given in the EN 1993-1-5.

Distortional buckling is taken into account by reducing the thickness of the buckling part of the cross-section. The method is based on the buckling of the beam on Winkler foundation. The beam is the buckling part of the cross-section and the rest of the cross-section forms the Winkler foundation. Figure 3 illustrates the buckling part in this case. The notations follow EN 1993-1-3.

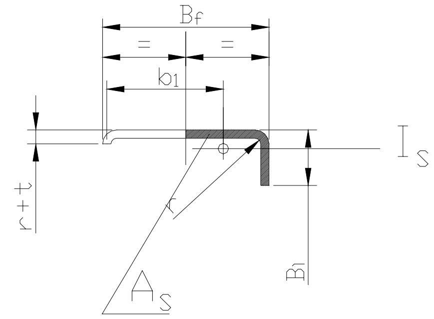

Figure 3. Buckling part (A_s, I_s) of the cross-section

The effective areas presented in Figure 4 are the results for the case L36.

Local buckling only Local buckling + distortional buckling

Figure 4. Effective areas for compression, case L36

At least three effective areas can be defined in this case:

- Effective area for local buckling A_{effl},
- Effective area for local buckling and distortional buckling A_{eff},
- Effective area for local buckling and short column (140 mm in this case) distortional buckling A_{effd}.

The first two are presented in Figure 4. It is known, that the distortional buckling have its wave length. If the column is shorter than this wave length, then the buckling part shown in Figure 3 will not buckle via distortional buckling. This case can be analysed as a flexural buckling case with the proper buckling length. The wave length or the buckling length for the distortional buckling is given by Timoshenko, Gere (1960) and it is

$$L_{cr,D} = \frac{\pi}{\sqrt[4]{\frac{K}{E \cdot I_s}}} \tag{1}$$

where K is the stiffness of the Winkler foundation, E is the Young's modulus and I_s is the inertia moment of the buckling part as shown in Figure 3.

The area of the gross cross-section is in this case 277.71 mm^2 and in the following is given the ratios of the effective areas to this value. The three effective areas for the case L36 are:

$$
\begin{aligned}
&A_{effl} = 211.87\,mm^2 \quad 211.87/277.71 = 0.76,\\
&A_{eff} = 190.24\,mm^2 \quad 190.24/277.71 = 0.68,\\
&A_{effd} = 197.79\,mm^2 \quad 197.79/277.71 = 0.71.
\end{aligned}
\tag{2}
$$

The yield stress in this case is 505 MPa, so the compressive resistance for the short column is 197.79*505/1000 = 99.9 kN. The compression test by Young, Rasmussen (1998) gave the ultimate load 100.2 kN and the Eurocode gives in this case very exact result compared to the test.

It is essential to note, that there are different kinds of ways to loose the stability for cold-formed sections and members. When considering the design of structures presented in Figure 1, there arise many situation in practice. When, for example, designing wall or floor elements, then in many cases distortional buckling may be neglected in the design. This happens, if the member is supported continuously by the sheeting on both sides and connected to the member properly. In the case L36 the equation (1) gives the distortional buckling length 313 mm (member height about 100 mm, thickness about 1.5 mm). This means that if the sheetings are connected using e.g. typical center to center distance 150,…, 200 mm, then the distortional buckling of the member may be neglected. In the literature, rules are presented to control the stability conditions to typical structures (Telua, Mahendran, 2000, Kesti, 2000). These rules are collected and based on them. The proposal for the automatic design of typical structures appearing in Figure 1 is given in Kukkonen (2005).

For longer columns flexural buckling in two directions, torsional and torsional-flexural buckling and lateral-torsional buckling must be considered following the Eurocode. The European buckling curve *b* must be used for these cases in every buckling mode. Figure 5 illustrates the different limit loads as a function of the slenderness in the weak direction i.e. the buckling length in the weak direction divided by the corresponding radius of gyration.

Figure 6 illustrates the resistance calculated using the European buckling curves *a* and *b*, correspondingly.

In this case the buckling curve *a* gives safe and better results compared to tests than the buckling curve *b*. It can be seen, also, that the new Eurocode gives rather reasonable and safe results in this case. For intermediate long columns (slenderness 40,…, 100) the results seem to be more safe than for short and very long columns.
The results for the cases of Table 1 are presented in Tables 2-4.

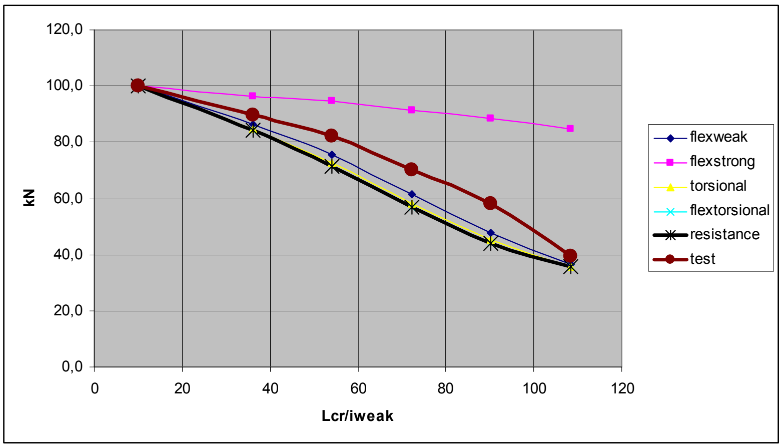

Figure 5. Limit loads as function of the slenderness in the weak direction

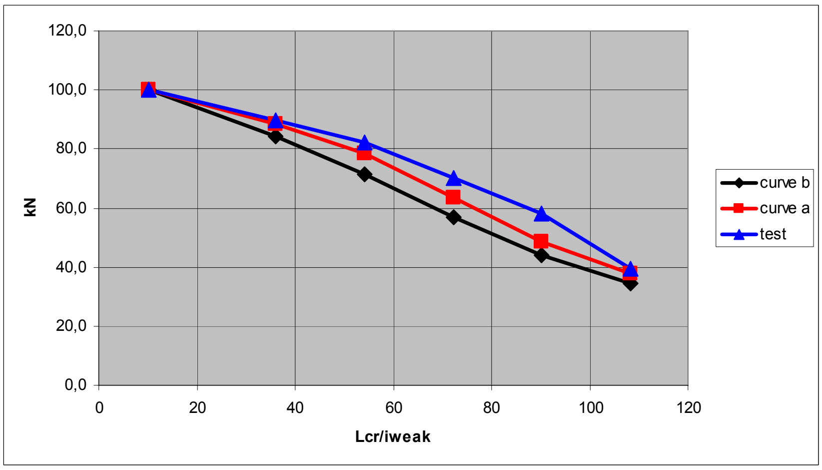

Figure 6. Resistances calculated using buckling curves *a* and *b*.

Table 2. Result for case CH17

CH17 [1)]	*Length (mm)*	P_{test} *(kN)*	P_{test}/P_{EC3}	$\mathbf{P_{test}/P_{EC3NEW}}$
	300	126.1	1.22	1.04
	700	123.0	1.20	1.15
	1100	108..5	1.11	1.09
	1370	106.6	1.14	1.12
	1640	103.5	1.20	1.16
	1900	104.5	1.28	1.25
Mean			1.19	1.14

The following coding is used for the failure modes of the columns in tests.

- L = Local buckling,
- D = Distortional buckling,
- F = Minor axis flexural buckling,
- FT = Flexural-torsional buckling.

Table 3. Result for case L36

L36 [2)]	*Length (mm)*	P_{test} *(kN)*	*Failure Mode*	P_{test}/P_{EC3}	$\mathbf{P_{test}/P_{EC3NEW}}$
	280	100.2	L+D	1.07	1.00
	1000	89.6	L+D+F	1.09	1.06
	1500	82.4	L+F+FT	1.17	1.15
	2000	70.1	L+F+FT	1.25	1.23
	2500	58.1	F+FT	1.33	1.33
	3000	39.3	F+FT	1.25	1.11
Mean				1.19	1.15

Table 4. Result for case L3.0W200

L3.0W200 [3)]	*Length (mm)*	P_{test} *(kN)*	*Failure Mode*	P_{test}/P_{AISI}	$P_{test}/P_{AZ/NZS}$	P_{test}/P_{EC3}	$\mathbf{P_{test}/P_{EC3NEW}}$
	500	380.9	L+D	1.13	1.13	1.10	1.09
	1000	365.4	L+D	1.10	1.10	1.07	1.07
	1500	352.7	L+D	1.09	1.09	1.09	1.08
	2000	348.7	L+D+F	1.11	1.11	1.13	1.13
	2500	339.8	L+D+F	1.13	1.13	1.17	1.19
	3000	329.2	L+F	1.15	1.15	1.23	1.25
Mean				1.12	1.12	1.13	1.13

The results for the case L36 with three spans (500, 1000 and 1500 mm) are given in Table 5, but in this case the column is pin-ended and the load is acting in the centroid of the cross-section. Note, that the shift of the centroid is not taken into account in the calculations. The effective length for the minor axis flexure is the column length and for the major axis flexure and for torsion and warping the effective lengths are taken as half of the column length due to supporting conditions. The test results for this case are taken from the reference Young, Rasmussen (1998).

Table 5. Axial resistances of the member in the case L36, pin-ended

$L_{cr,z}=L_{cr,y}/2=L_T/2$ *(mm)*	$N_{b,Rd,Fz}$ *(kN)*	$N_{b,RdFy}$ *(kN)*	$N_{b,Rd,T}$ *(kN)*	$N_{b,Rd,TF}$ *(kN)*	$N_{b,Rd}$ *(kN)*	P_{test} *(kN)*	P_{test}/N_{EC3NEW}	**Failure Mode/test**
500	81.7	90.4	88.5	88.7	81.7	82	1.00	L+F
1000	59.4	90.4	80.2	79.8	59.4	70	1.17	L+F
1500	36.1	88.9	69.2	68.5	36.1	40	1.10	F
Mean							1.09	

Consider the lipped channel profile presented in Figure 7. The test results are originating from the reference Yan, Young (2002).

Tested columns were fixed-ended as in the previous case. The column lengths ranged from 500 mm to 3500 mm. The results for one case are presented in Table 6.

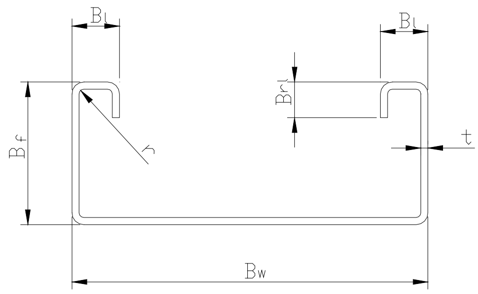

Figure 7. Cross-section of the lipped channel and definitions of symbols

Table 6. Result for case T1.5F80

Case Name	*Web B_w (mm)*	*Flange B_f (mm)*	*Lip B_l (mm)*	*Lip return B_{rl} (mm)*	*Thickness t_{cor} (mm)*	*Radius r (mm)*	*Strength f_y (MPa)*	**Young's Modulus E (MPa)**
T1.5F80	153.6	83.5	28.0	17.6	1.482	2.0	522	204000

T1.5F80	*Length (mm)*	*P_{test} (kN)*	*Failure Mode*	*P_{test}/ P_{AISI}*	*P_{test}/ $P_{AZ/NZS}$*	**P_{test}/ P_{EC3NEW}**
	500	172.0	L	0.94	1.11	1.06
	1000	166.9	L+D	0.93	1.07	1.25
	1500	163.4	L+D	0.95	1.05	1.27
	2000	161.7	L+D	0.99	1.04	1.32
	2500	158.8	L+FT	1.04	1.04	1.36
	3000	154.8	L+FT	1.11	1.11	1.41
	3500	124.4	L+FT	0.99	0.99	1.22
Mean				0.99	1.06	1.27

Consider the channel profile with inclined lips presented in Figure 8. The test results are originating from the reference Young, Hancock (2002).
Tested columns were fixed-ended as in the previous case. The column lengths were about 1500 mm. The results one of these cases is presented in Table 7.

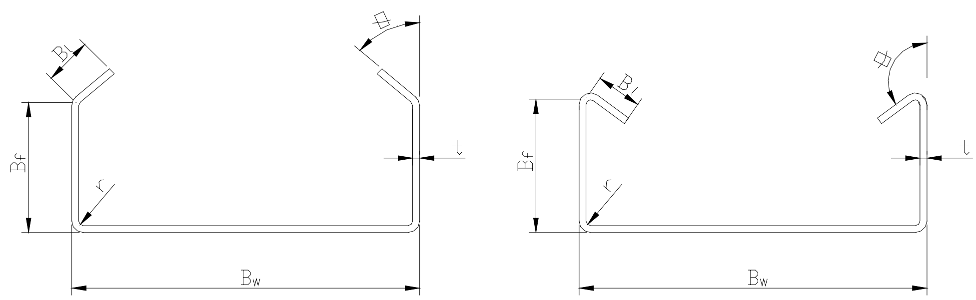

Figure 8. Cross-section of the channel with inclined lips and definitions of symbols

Table 7. Results for case ST15

Case Name	*Web B_w (mm)*	*Flange B_f (mm)*	*Lip B_l (mm)*	*Thickness t_{cor} (mm)*	*Radius r (mm)*	*Yield Strength f_y (MPa)*	**Young's Modulus E (MPa)**
ST15	99.7	50.1	10.7	1.49	1.0	515	200000

	Length (mm)	*Angle (°)*	*P_{test} (kN)*	*P_{test}/ P_{NAS}*	*P_{test}/ P_{AISI}*	*P_{test}/ $P_{AS/NZS}$*	**P_{test}/ P_{EC3NEW}**
ST15A30	1504.1	32.0	76.0	1.01	1.01	1.06	1.03[1)]
ST15A45	1503.0	46.2	81.3	1.04	1.04	1.05	1.10
ST15A60	1503.5	61.1	83.4	1.06	1.06	1.06	1.11
ST15A90	1503.7	89.5	97.3	1.23	1.29	1.29	1.31
ST15A120	1503.8	119.8	102.2	1.31	1.38	1.38	1.46
ST15A135	1504.2	134.0	90.4	1.22	1.29	1.29	1.34
ST15A150	1502.9	140.5	97.3	1.32	1.40	1.40	1.47[*)]
Mean				1.17	1.21	1.22	1.26

[*)] *c/b* < 0.2

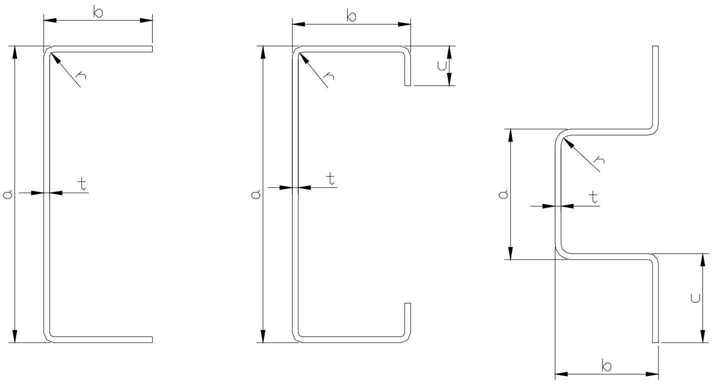

Figure 9. Cross-sections of U, C and hat profiles

Note, that in this case the conditions for *c/t* and *c/b* are checked using the projection of the lip as stated in the new Eurocode.
Consider U, C and hat profiles presented in Figure 9. The test results are originating from the reference Talja (1990).
The columns are fixed at both ends as in the previous cases. Results for one case is presented in Table 8.

Table 8. Results for case U-186

Case	*a* (mm)	*b* (mm)	*c* (mm)	t_{cor} (mm)	*r* (mm)	f_y (MPa)	**E** ***(MPa)***
U-186x80x6	186.4	79.8	-	5.96	7.5	570	210000[*)]

[*)] Estimated

	Length (mm)	*P_{test} (kN)*	*Failure Mode*	**P_{test}/ P_{EC3NEW}**
U-186x80x6	490	980	L	1.04
	1200	1020	L+F	1.24
	2100	900	L+FT	1.37
	3100	640	F	1.29
Mean				1.23

BENDING OF COLD-FORMED MEMBERS

Consider next some bending cases. Consider the channel profile presented in Figure 10. The test results are originating from the reference Beale, Godley, Enjily (2001).

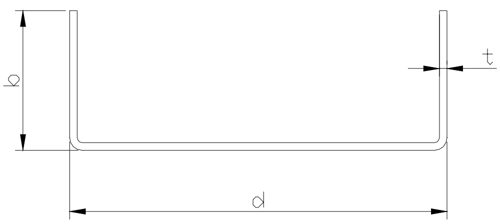

Figure 10. Cross-section of the channel with unstiffened flanges and definitions of symbols

The loading is such, that the bending is around the weak axis and the web is at the tensile side. The beam is simple supported beam and the load is at the mid-span acting via special rollers to the beam.

Figure 11 illustrates the ultimate load versus *b/t* ratio for two spans. It must be noted, that the limit *b*/*t* = 50 exist in the Eurocode and many of the test results are beyond this limit. Moreover, it can be seen that Eurocode gives very much safe results for this case (mean 73% too safe). Too much from the flanges is buckling away following the Eurocode method in this case. In this case the so called Murray yield line patterns covers the ultimate limit state. This case was analysed also by using the method of effective thicknesses for outstand elements as stated in the new Eurocode. It did not change the results very much in this case.
Consider next the C and Z profiles presented in Figure 12. The test results are originating from the reference Yu, Schafer (2003).

The members were simply supported and loaded by two point loads locating at *L*/3 from the supports. The top and bottom flanges were supported laterally by the sheeting and the resistance of the member was calculated by multiplying the yield stress by the effective bending modulus. Typical results for this case are presented in Table 9.
Consider the C, Z, Sigma and hat profiles presented in Figure 13. The test results are originating from the reference Heinisuo, Laine (1995).

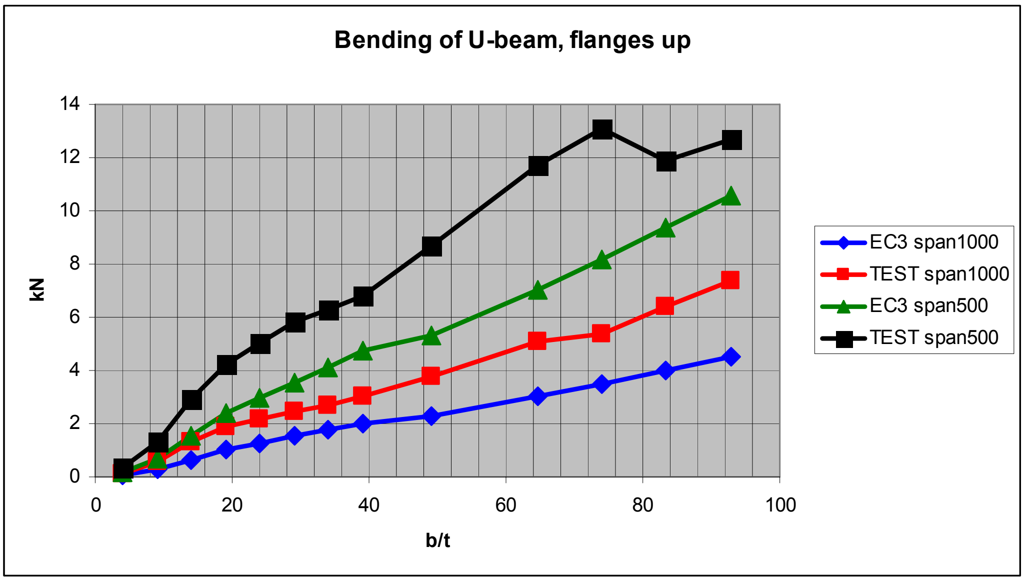

Figure 11. Ultimate load versus *b/t* ratio

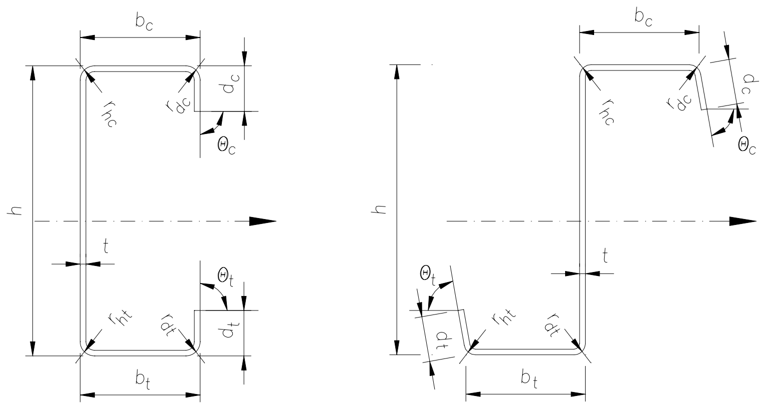

Figure 12. Cross-sections of C and Z profiles

Table 9. Results for the case 8.5Z

Case	h (mm)	b_c (mm)	d_c (mm)	θ_c (°)	b_t (mm)	d_t (mm)	θ_t (°)	$r^{*)}$ (mm)	t_{cor} (mm)	f_y (MPa)	**E** ***(MPa)***
8.5Z120	215	66	24	47.5	62	25	48.9	9	3.00	418	203000
8.5Z105	215	68	25	50.6	60	24	48.7	8.5	2.66	467	203000
8.5Z092	214	66	24	52.4	61	24	50.6	7.5	2.27	393	203000
8.5Z082	215	64	24	48.5	61	25	51.4	7.5	2.03	401	203000
8.5Z073	216	64	23	49.6	61	24	50.9	7.5	1.83	380	203000
8.5Z065	215	62	20	47.4	62	21	47.2	7	1.63	367	203000
8.5Z059	216	64	20	50.7	59	18	49.7	7	1.50	404	203000

*) mean of rounding

Case	*Span (mm)*	M_{test} *(kNm)*	M_{test}/M_{AISI}	M_{test}/M_{S136}	M_{test}/M_{NAS}	$\mathbf{M_{test}/M_{EC3NEW}}$
8.5Z120	4878	31.7	1.06	1.06	1.06	1.12
8.5Z105	4878	30.2	1.06	1.07	1.05	1.08
8.5Z092	4878	20.5	1.00	1.03	1.00	1.03
8.5Z082	4878	18.3	1.00	1.05	1.00	1.11
8.5Z073	4878	14.2	0.94	1.01	0.93	1.07
8.5Z065	4878	10.8	0.88	0.98	0.88	1.07
8.5Z059	4878	11.3	0.97	1.05	0.97	1.28
Mean			0.99	1.04	0.98	1.11

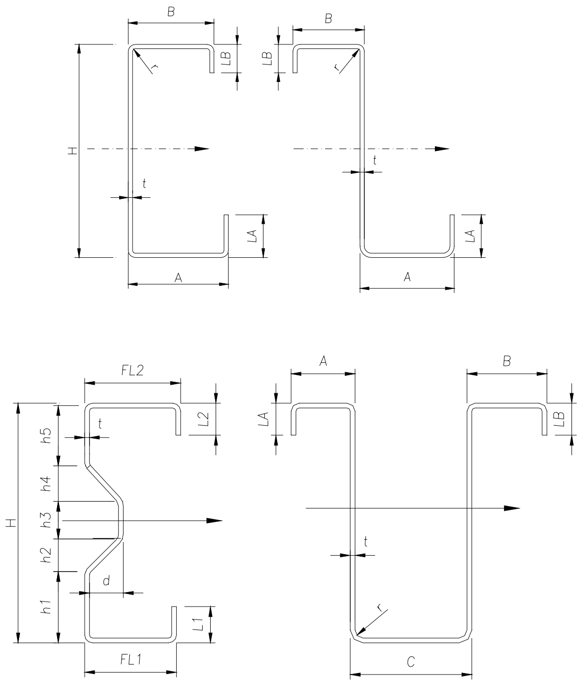

Figure 13. Cross sections of C, Z, Sigma and hat profiles

The tested beams were simply supported and loaded with a point load acting at the mid-span. Only the loading points were supported from the webs as were the support points. The bending was strong axis bending and the load was acting vertically to the web. So, the flanges were free to deform in any direction between the supports.

The calculated resistances were determined firstly by multiplying the yield stress by the effective bending modulus as was done in the reference. This was done to compare the ENV results to EN results. The resistances were calculated also, by taking into account the possibility for lateral-torsional buckling. The buckling lengths *L* and *L*/2 were used. The critical moment was calculated using the well-known code equation for uniformly distributed bending moment for the simply supported beam.
Typical results for one case are presented in Table 10.

Table 10. Results for Z profile bending

Case	H (mm)	A (mm)	LA (mm)	B (mm)	LB (mm)	t_{cor} (mm)	r [*)] (mm)	f_y (MPa)	**E** ***(MPa)***
Z1,2	100.5	45.2	16.8	38.8	17.1	0.94	1.88	353	197381
Z3	101.9	45.2	16.9	39.3	16.9	1.93	3.86	341	189444
Z4,5	249.0	71.7	25.5	64.3	25.5	1.97	3.94	360	189500
Z6	251.5	73.1	23.0	63.6	25.4	2.92	5.84	375	212857
Z7	350	81.0	22.8	74.2	22.9	1.44	2.88	350	208574
Z8,9	350	81.2	23.1	74.4	22.6	1.48	2.96	350	207330

[*)] corner radius 2r

Case	F_{test} *(kN)*	*Span (m)*	*M (kNm)*	M/M_{EC3}	M/M_{EC3NEW}	M/M_{EC3NEW} $L_{LT}=L/2$	$\mathbf{M/M_{EC3NEW}}$ $\mathbf{L_{LT}=L}$
Z1,2	3.9	1.5	1.49	0.91	0.91	0.95	1.11
Z3	11.6	1.5	4.37	1.03	1.09	1.14	1.27
Z4,5	27.8	2	13.92	0.82	0.81	0.85	0.97
Z6	44.2	2.5	27.63	0.88	0.89	0.94	1.13
Z7	23.5	2	11.76	0.85	0.89	0.92	1.03
Z8,9	17.2	3	12.97	0.96	0.93	1.01	1.36
Mean				0.91	0.92	0.97	1.15

It can be seen, that the most proper buckling length for the lateral-torsional buckling may be in this case something between L and $L/2$.

BENDING AND AXIAL COMPRESSION OF COLD-FORMED MEMBERS

Two references, Talja (1992) and Salmi, Talja (1993), for bending and axial compression of cold-formed members are used in the following. The cases were eccentrically compressed members. The eccentricity is marked with e in the following figures. The test cases include high strength steels to which the new Eurocode system is also valid (see EN 1993-1-12).

Case 1 is originating form the reference Talja (1992). The dimensions are presented in Figure 14.

The member was free to loose its stability between supports. The member length is L. The buckling lengths $L_{crT} = L_{crTF} = L$ and $L_{crLT} = L/2$ are valid for all the members. The flexural buckling length for U-profile is $L_{crx} = L$ and $L_{cry} = 0.5\ L$ and for C-profile and hat profile $L_{crx} = 0.5\ L$ and $L_{cry} = L$ using the axis notations presented in Figure 14.

The results for "short" members are given in Table 11. The initial data and the results for "long" members are given in the reference Heinisuo, Kukkonen (2005).

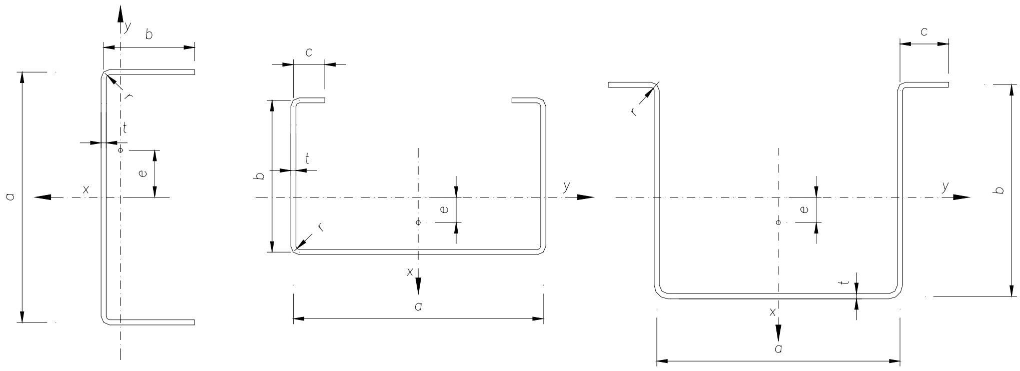

Figure 14. Cross sections for the case 1.

The notations for the observed failure limit states in the tests are as follows:

- P means bending failure in the plane,
- T means observed torsion of the member,
- S means buckling of the lip,
- L means local deformations of the cross-section.

The notation *YV 1-3* means the results of the interaction equation for bending and compression given in EN 1993-1-3. *YV M1* means the results of the interaction equation for bending and compression given in EN 1993-1-1, Method 1.

Table 11. Results for the case 1, short members

Case	a (mm)	b (mm)	c (mm)	t_{cor} (mm)	r (mm)	f_y (MPa)	**E** ***(MPa)***
UU1	121	37.0	-	6.05	8.0	620	210000
C1	196	75.4	23.6	3.95	3.9	560	210000
C2	125	124	20.8	4.95	1.9	570	210000
H1	146	145	33.1	5.95	8.6	670	210000
H2	126	124	55.3	5.95	8.4	650	210000

Case	F_{test} *(kN)*	*e (mm)*	*Length (m)*	*M (kNm)*	*Failure Mode*	*YV 1-3*	**YV M1**
UU1/NR1	520	30.5	0.240	15.86	L	1.54	1.45
C1/NR1	412	-22.5	0.620	9.27	S	1.48	1.58
C1/NR2	445	15.5	0.620	-6.90	L	1.37	1.06
C2/NR1	614	-22.5	0.480	13.82	S	1.35	1.23
C2/NR2	714	42.0	0.480	-29.99	L	1.75	1.55
H1/NR1	1305	-21.0	0.480	27.41	S	1.42	1.32
H1/NR2	1209	44.5	0.480	-53.80	L	1.63	1.46
H2/NR1	1172	-32.0	0.480	37.50	L	1.37	1.27
H2/NR2	1028	53.0	0.480	-54.48	L	1.51	1.40
Mean						1.49	1.37

Case 2 is originating form the reference Salmi, Talja (1993). The dimensions are presented in Figure 15.

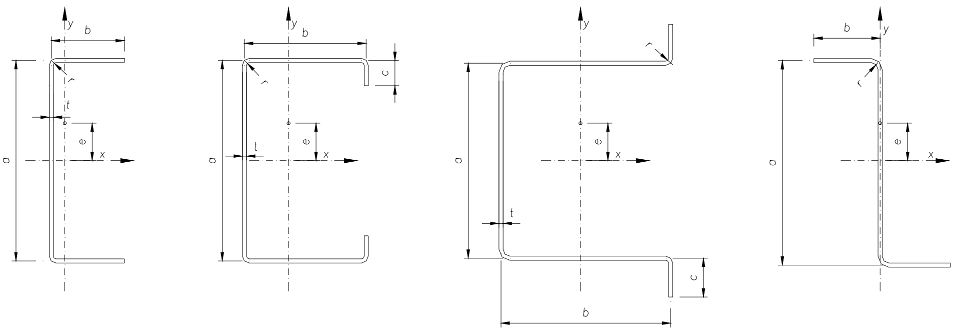

Figure 15. Cross sections for the case 2.

In this case $L_{crT} = L_{crTF} = L_{crx} = L$ and $L_{cry} = 0.5L$ and $L_{LT} = L/2$.

The results for "long" members are presented in Table 12. In this case R means bending failure perpendicular to the plane of loading.
Other notations are the same as in the case 1.

Table 12. Results for the case 2, long members

Case	*a* (mm)	*b* (mm)	*c* (mm)	t_{cor} (mm)	*r* (mm)	f_y (MPa)	**E *(MPa)***
U2	136	53.7	-	3.95	5.3	655	210000
C2	125	124	20.4	4.95	2.1	565	210000
C3	215	74.8	28.9	4.95	4.8	565	210000
H1	146	144	33.0	5.95	9.3	645	210000
Z1	149	76.5	-	6.80	12.5	400; 475*)	210000

*) Yield strength 475 MPa was used for cases NR5 and NR7

Case	F_{test} *(kN)*	*e (mm)*	*Length (m)*	*M (kNm)*	*Failure Mode*	*YV 1-3*	**YV M1**
U2/NR6	163	68.0	2.200	11.08	T	1.45	1.92
U2/NR7	80	68.0	3.500	5.44	T	1.11	1.22
C2/NR6	246	62.5	3.500	15.38	T	1.23	1.16
C3/NR6	306	107.5	3.500	32.90	T	1.29	1.46
H1/NR6	390	73.0	3.500	28.47	T	1.36	1.21
Z1/NR6	273	74.5	2.200	20.34	T+R	1.32	1.39
Z1/NR7	165	74.5	3.500	12.29	T+R	1.04	0.97
Mean						1.26	1.33

CONCLUSIONS

Mean of all results in the study gives some picture for the safety margin of the new Eurocode compared to test results. The critical cases are such where the code gives unsafe results. The following results and summary as presented in Table 13 can be collected from the results in the study. All cases considered in the references Heinisuo, Kukkonen (2005) and Kukkonen (2005) are taken into account in the following. Total amount of 272 cases were considered, 178 compression, 64 bending and 30 bending and compression cases.

Table 16. Mean of all results in the study

Case	Mean: Test/Code
C compression	1.17
Lipped C compression	1.28
Inclined lips C compression	1.17
U compression	1.14
Hat compression	1.18
U bending weak axis	1.74
C bending	1.04
Z bending	1.08
Sigma bending	1.06
Hat bending	1.03
U compression&bending, EN 1993-1-3	1.33
U compression&bending, EN 1993-1-1	1.46
C compression&bending, EN 1993-1-3	1.34
C compression&bending, EN 1993-1-1	1.33
Hat compression&bending, EN 1993-1-3	1.29
Hat compression&bending, EN 1993-1-1	1.30
Z compression&bending, EN 1993-1-3	1.07
Z compression&bending, EN 1993-1-1	1.06

It can be seen, that the new Eurocode is 10-30% conservative for compression members. For pure bending the safety margin is less than 10%. The interaction for compression and bending the safety margin varies from 6% to over 40% in these cases. For the bending of U beam around weak axis and with compression at flanges the code gives typically about 70% safety margin to tests and for these cases more refined method is recommended to be used when designing these structures.

On the other hand, the general goal is that the new Eurocode covers very many kinds of structures and cross-sections. In this study typical cold-formed cross-sections and one span members are dealt only. No intermediate longitudinal stiffeners at flanges and webs were present in the cases considered. From this point of view it seems, that the safety margin appearing in the codes is welcome.

Many cases where the new Eurocode gave unsafe results were outside the scope of the code due to limitations of the code for the dimensions of the cross-section. In some cases it was possible to exceed these limitations safely. The upper limit for *c/t* ratio seemed to be proper, when applying it for the projection of the lip if the lip was inclined.

When considering the stability conditions for cold-formed structures, the European buckling curve *a* seemed to possible to be used in most of the cases.

When considering the safety margins between European and other codes, it seems that the Australian/New Zealand code gives generally best results for compression members when comparing the calculated results against the test results. The American codes gives in many bending cases unsafe results and Eurocode gives in these cases safe results. The differences between the results calculated using ENV and EN was not very large in these cases.

The new European code seems to be suitable to perform safe designs in the cases considered generally. The European code is suitable for the whole range of materials covered by the codes up to 690 MPa yield limit. In some cases it is so safe that it is recommended to use other methods to perform the economical design. It must be noted, that in every case for mass production members it is recommended to use the design assisted by tests. It is the most suitable method for often very complicated geometrical forms of cold-formed structures.

Further studies are needed for cross-sections with intermediate longitudinal stiffeners at the flanges and at the webs. The profiles including perforations, e.g. thermal studs, must be studied in the future. Also, more complicated complete structures with members and joints must be studied. The problem to study those is that the test results are extremely hard to find for those structures.

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