SDSS'Rio 2010 STABILITY AND DUCTILITY OF STEEL STRUCTURES
E. Batista, P. Vellasco, L. de Lima (Eds.)
Rio de Janeiro, Brazil, September 8 - 10, 2010

INFLUENCE OF FLANGE-TO-WEB CONNECTION ON THE PATCH LOAD RESISTANCE OF I BEAMS

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Keywords: patch load, steel and aluminium I-beams, curved flange-to-web connection, non-linear numerical simulation, interaction of bending and transverse load resistance, Eurocode.

Abstract. *Current design methods for calculating transverse load resistance of web plates neglect the effect of the flange-to-web connection. This simplification is on the conservative side and in deep plate girders its influence is practically negligible. However, in case of hot-rolled or extruded profiles, the fillet corner shaping through its geometry, rigidity and strength may highly increase the patch load resistance. Using non-linear numerical analysis, the author completed a parametric study on simply supported girders subjected to transverse load in order to study a) the effect of the connection and b) the capacity in interaction of transverse load and bending. The study clearly confirms the beneficial influence of the curved corners. To take this effect into account the author proposes a modification in the Eurocode formulation. The results of the simulation prove the validity of the proposed method.*

1 INTRODUCTION

It is well known that transverse (or patch) load resistance of plate girders is influenced by the load case (patch load, opposite patch load, end patch load), the web and flange material properties (i.e. yield limits), the dimensions (a, h_w) and thickness (t_w) of the unstiffened web panel, and the flange dimensions (b_f, t_f). Additionally, the loaded length (s_s) has major importance in the resistance, too. These details are typically considered in the design method as well, just like in current formulations of corresponding Eurocodes, such as Eurocode 3 Part 1-5 for steel plated structures (EC3 [1]) and Eurocode 9 Part 1-1 for aluminium structures (EC9 [2]).

However, effect of the connection between the flange and web, thus the effect of the fillet curved corners is fully neglected. Even though this simplification is on the conservative side and reasonably accurate for deep slender girders, this type of connection shaping may highly increase the resistance, as it is confirmed in [3]. Firstly, – similarly to the calculation of local plate buckling under compression – one may claim to consider the fillet reducing the web height. Secondly, it widens the effective loaded length of the web. Thirdly, it may highly influence the plastic hinge capacity of the flanges; that is the base of the mechanism solution model of patch loading originally recommended by Roberts et al [4].

Reviewing the development of the actual code formula, the reason why this effect is out of consideration can be found. (Note that EC9 applies the same method as given in the steel standard EC3.) According to Lagerqvist et al [5], the calibration of the semi-empirical design method was completed on the basis of 388 test specimens made of steel, including 358 welded girders, 11 European and 19 American rolled beams. Most of these tested girders come with high, slender web and only few cases represent rolled/extruded profiles with stocky webs and relatively large curved corners. Consequently, the calibration is directly valid for slender webs only. Needless to say, welded connection that has smaller extent than practically applied curved corners of rolled profiles results in much smaller influence on the resistance. It can be also stated that the higher and more slender the web is, the less the effect of the edge boundary condition is. Note that this effect can be much higher in case of aluminium, because extruded profile usually comes with larger radius due to fabrication and material reasons.

The author completed a parametric numerical study on various steel and aluminium I-beams configurations and the corner effect is quantified. Based on the results, the author proposes a simple modification in the Eurocode method to account for this detail and its beneficial effect.

2 STANDARD EUROCODE METHOD

This section summarizes the current basic procedures for bending and transverse load resistance calculations according to EC3 and EC9 (referred as standard method hereafter).

In this study, only compact, ductile profiles (classified as Class 1 sections) with I-shape cross-section (Figure 1) are considered, i.e. local plate buckling due to axial stresses does not affect the static behaviour of the girder. Accordingly, **plastic bending resistance** $M_{c,Rd}$ is calculated. EC9 alternatively allows to consider strain hardening through the application of Ramberg-Osgood law, [2]. Thus,

$$\text{steel: } M_{c,Rd} = W_{pl}\frac{f_y}{\gamma_{M0}} \qquad \text{aluminium: } M_{c,Rd} = \alpha_{M,1} W_{el}\frac{f_o}{\gamma_{M1}} \tag{1/a,b}$$

where W_{pl} and W_{el} are the elastic and plastic section modulus, respectively; $\alpha_{M,1}$ stands for the correction factor to account for the plastic overstrength including strain hardening; f_y and f_o are the characteristic yield strength and the proof strength, respectively; while γ_M is the partial safety factor.

Both EC3 and EC9 prescribe exactly the same mechanism-solution based procedure for the **transverse load resistance** calculation. As per EC9, for simple patch load case the method follows:

$$\text{critical load: } F_{cr} = 0.9 k_F E\frac{t_w^3}{h_w} = 0.9\left[6 + 2\left(\frac{h_w}{a}\right)^2\right] E\frac{t_w^3}{h_w} \tag{2}$$

$$\text{dimensionless parameters: } m_1 = \left(f_{of} b_f\right)/\left(f_{ow} t_w\right); \quad m_2 = 0.02\left(h_w / t_f\right)^2 \tag{3/a,b}$$

$$\text{effective loaded length: } l_y = s_s + 2t_f\left(1 + \sqrt{m_1 + m_2}\right) \le a \tag{4}$$

$$\text{slenderness: } \lambda_F = \sqrt{\frac{l_y t_w f_{ow}}{F_{cr}}} \tag{5}$$

$$\text{reduction factor: } \chi_F = 0.5/\lambda_F \le 1.0 \tag{6}$$

$$\text{transverse load resistance: } F_{Rd} = \chi_F l_y \frac{f_{ow}}{\gamma_{M1}} \tag{7}$$

where E is Young's modulus; k_F is the buckling coefficient; a is the length of the unstiffened web panel (Figure 1); h_w and t_w are the web height and thickness; b_f and t_f are the flange width and thickness; s_s is the stiff bearing length; f_{of} and f_{ow} are the proof strength of the flange and the web, respectively.

The **interaction of design bending moment M_{Ed} and transverse loading F_{Ed}** shall be checked through the following interaction formula:

$$\frac{F_{Ed}}{F_{Rd}} + 0.8\frac{M_{Ed}}{M_{c,Rd}} \le 1.4 \tag{8}$$

For simplicity, this study does not deal with the complex interaction of shear, bending and transverse load. (Note that influence of shear load on the patch load resistance is currently not covered by the basic method of Eurocode. A useful method is discussed in [6].) Additionally note that the following results are corresponding to the simple patch load case; the author did not deal with opposite and end patch loading.

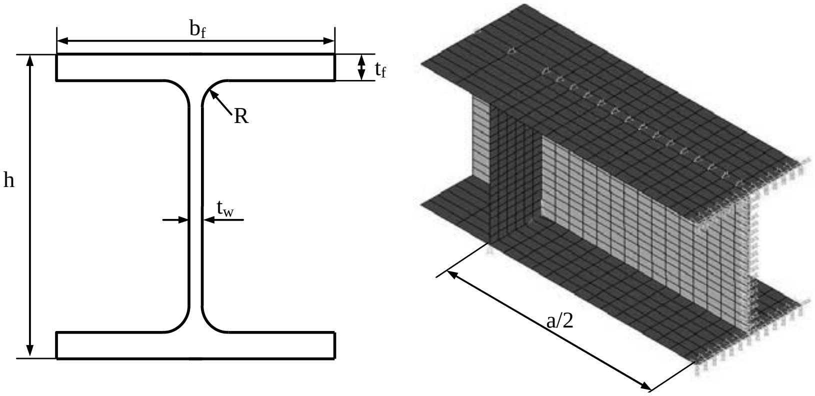
3 PARAMETRIC NUMERICAL STUDY

3.1 Programme

The author completed a parametric study in order to quantify the effect of the curved corners on patch load resistance of rolled steel and extruded aluminium girders. Simply supported girders (with I-shape cross-section shown in Figure 1) subjected to transverse concentrated load at midspan is considered. Varying parameters are the section geometry, the span a (= length of unstiffened web panel), the loaded length s_s and the radius r of the curved corner. The parametric study programme is summarized in Table 1. One series of analysis was carried out assuming HEA sections made of steel grade S235 and one with modified HEA sections made of a specific AlMgSi alloy. In the aluminium case, profile modification – namely, change of flange width – aimed to obtain ductile sections. Altogether, the analysis series include more than 150 cases.

The cross-sections are ductile (Class 1) in each case. Varying the span permits of analysing cases of dominant bending failure, dominant web crippling or their interaction. Three different loaded lengths are investigated: 0 mm, 50 mm and 200 mm, respectively. The basic radius for the different sections are 12 mm, 18 mm and 27 mm, respectively; these values are multiplied by 0, 1, 1.5 and 2.

For simplification, interaction with shear is not discussed here: cases where the influence of shear on design bending resistance is larger than 5% are excluded.



a) cross-section b) shell-element model

Figure 1: Parameters and numerical model for the parametric study.

3.2 Numerical modelling technique

For the parametric study, geometrically and materially non-linear analysis is completed using ANSYS [7]. Figure 1/b shows the shell-element geometrical model. The 4-node SHELL181 element is suitable to model the curved corner: the element may have linearly varying thickness along its edges. Bilinear approximation – illustrated in Figure 2/a – is applied in such way that the same joint section area is provided and thus the resulting transverse plate rigidity is certainly not overestimated.

One-bow geometrical imperfection is applied in the web with a magnitude of $h/200$. Elastic-perfectly plastic bilinear material model is adjusted to the steel cases: to overcome numerical problems a fictive strain hardening with a tangent slope of E/10000 is applied. To the aluminium specimens the elastic-hardening non-linear model shown in Figure 2/b is adjusted. It is assumed that the profiles are manufactured as a whole and no welding is necessary. Thus, welding does not influence the material behaviour of the aluminium specimens.

The numerical model is validated in [3]. Note that according to the Eurocodes, – beside the standardized procedure – such numerical simulation can be alternatively applied for design purposes.

Table 1: Programme and results

"HEA 100" / "HEA 100-75"

h [mm]	b_f [mm]	t_f [mm]	t_w [mm]	r [mm]	s_s [mm]	a [mm]	$F_{Rd,FEM}$ Steel [kN]	$F_{Rd,FEM}$ Alu [kN]
96	100 (St) 75 (Al)	8	5	0	0	150	112.9	101.5
						300	106.1	93.9
						500	95.2	83.5
						750	81.5	71.9
					50	300	139.3	129.9
						500	123.8	118.6
						750	104.1	91.7
					200	300	211.0	209.4
						500	147.0	154.8
						750	124.0	112.4
				12	0	150	135.8	130.3
						300	125.0	116.7
						500	109.7	101.9
						750	93.0	84.4
					50	150	189.9	184.2
						300	155.0	154.7
						500	134.0	135.3
						750	110.9	99.5
					200	300	223.0	246.8
						500	148.0	168.2
						750	127.0	121.9
				18	0	500	123.0	116.8
						750	103.2	94.6
				24	0	150	187.2	190.7
						300	159.0	163.2
						500	137.7	134.7
						750	114.2	106.9

"HEA 200" / "HEA 200-100"

h [mm]	b_f [mm]	t_f [mm]	t_w [mm]	r [mm]	s_s [mm]	a [mm]	$F_{Rd,FEM}$ Steel [kN]	$F_{Rd,FEM}$ Alu [kN]
190	200 (St) 100 (Al)	10	6.5	0	0	500	204.0	158.8
						1200	180.0	131.6
						2000	149.0	103.6
					50	500	256.1	204.8
						1200	231.0	173.3
						2000	188.0	122.8
					200	500	391.0	346.3
						1200	325.0	230.7
						2000	212.0	135.5
				18	0	500	245.0	200.7
						1200	212.0	163.8
						2000	173.8	124.2
					50	500	303.4	251.6
						1200	269.6	207.9
						2000	205.2	139.7
					200	500	417.0	401.3
						1200	329.0	256.3
						2000	222.0	150.7
				27	0	500	282.2	240.1
						1200	243.0	194.5
						2000	197.0	142.4
				36	0	500	331.1	288.4
						1200	283.7	231.7
						2000	221.0	164.8

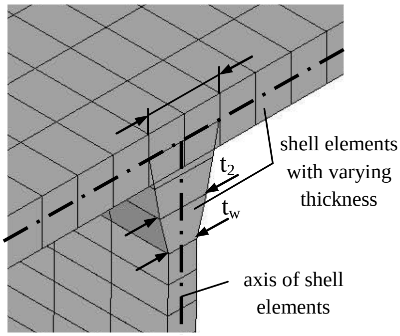
"HEA 300" / "HEA 300-140"

h [mm]	b_f [mm]	t_f [mm]	t_w [mm]	r [mm]	s_s [mm]	a [mm]	$F_{Rd,FEM}$ Steel [kN]	$F_{Rd,FEM}$ Alu [kN]
290	300 (St) 140 (Al)	14	8.5	0	0	1000	371.8	277.3
						2000	334.5	233.4
						3000	293.0	195.3
					50	1000	441.0	333.2
						2000	402.0	286.8
						3000	350.6	230.2
					200	1000	645.0	---
						2000	598.8	386.9
						3000	435.0	262.6
				27	0	1000	460.1	355.5
						2000	404.2	300.0
						3000	348.1	244.3
					50	1000	537.9	419.0
						2000	482.0	355.7
						3000	409.8	270.9
					200	1000	691.0	622.3
						2000	641.0	441.4
						3000	463.0	293.9
				40.5	0	1000	535.7	434.0
						2000	471.5	362.4
						3000	404.1	286.5
				54	0	1000	626.5	529.7
						2000	558.0	437.4
						3000	465.0	338.1

h – total section height; b_f – flange width; t_f – flange thickness; t_w – web thickness; *r* – radius of corner; s_s – loaded length; *a* – span (= length of unstiffened web panel); $F_{Rd,FEM}$ – FEM load capacity

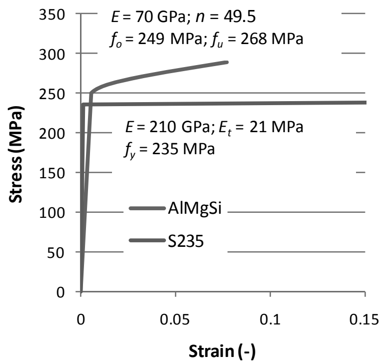
Table 2: Comparison to $r = 0$.

r =	Steel %	Steel avg %	Alu %	Alu avg %
r	5 - 24	16	8 - 29	20
1.5 r	27 - 44	36	32 - 57	46
2 r	40 - 67	54	49-91	74



Note: these are not body elements

a) approximation of curved corners



b) material models

Figure 2: Modeling details.

3.3 Discussion of results

3.3.1 Results

The load capacities obtained by the analyses are tabulated in Table 1. Utilization factor for bending moment is calculated as the ratio of the ultimate load F_{RD} (or M_{Rd}) computed from the numerical analysis to the design bending moment resistance $M_{c,Rd}$ calculated in accordance with Eurocodes. Utilization factor for transverse load resistance (F_{RD} to $F_{patch,RD}$) is similarly determined. The interaction results are illustrated by the help of these parameters in Figure 3 where the EC interaction curve is also plotted.

3.3.2 Reliability of the results

The applied non-linear numerical analysis is an alternative design procedure allowed for by both Eurocodes. In Figure 3/a, the interaction points corresponding to the reference case r = 0 lie in the vicinity of the standard interaction curve, which in general confirms the validity of the results. On average the deviation is within 10%. However, in many cases the numerical simulation gives conservative result compared to the standard procedure. Different interpretations may be given for this observation:

1) On the one hand, this may indicate that the numerical model is conservative, i.e. especially the applied geometrical imperfection, or the way of joint discretization, etc. is conservative.
2) Reference comparison perhaps should be done to cases of normal radius, as the calibrated design method may indirectly include some connection effect.
3) The fact that zero loaded length cannot be kept in experimental environment queries the reliability of the standard method: it may overestimate the capacity for cases s_s = 0. When excluding these cases, only 5 of the points fall below the standard interaction curve.

As a consequence, further study is required to accurately evaluate the reliability of the alternative design methods.

Despite the discussed uncertainties, it can be stated that the numerical results are in accordance with the standardized method and the analysis with different radius and loaded length gives a solid base for the following comparative study.

3.3.3 Effect of curved corners

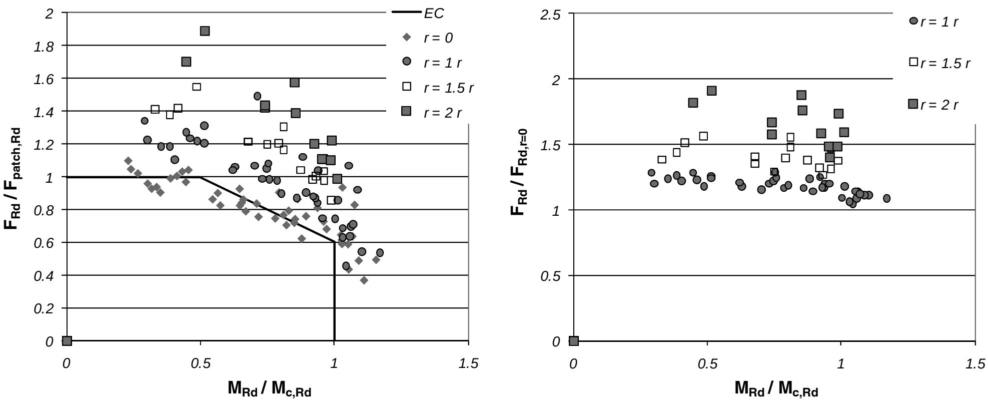
The results confirm that the curved corner may highly influence the transverse load resistance as well as the resistance in interaction with bending. Compared to the reference cases r = 0, even the consideration of normal radius leads to notable increase in the capacity, as Figure 3/a,b and Table 2 prove. When using double radius, up to 67% and 91% increase can be achieved in case of steel and aluminium, respectively.

Load vs. deflection curves and deformed shapes of Figure 4 well demonstrate the quantitative and qualitative change in capacity and in nature of behaviour. For example, compare post-ultimate behaviour of cases s_s = 200 mm in Figure 4/a: in case of no radius web crippling dominates as indicated by the sudden drop in the post-peak range, while the existence of curved corner leads to governing bending failure. In the latter case, due to the ductile (Class 1) section, long yield plateau can develop, followed by the capacity drop due to instability at the very end of the curve.

The larger influence in case of aluminium alloy can be explained by two reasons:

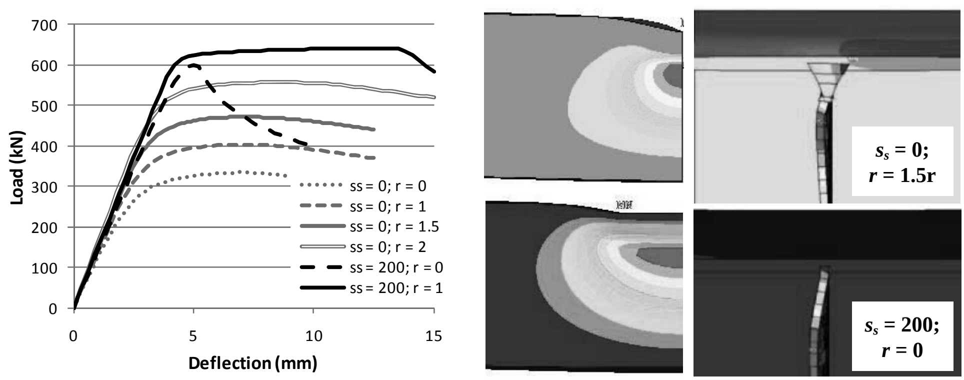
1) The selected aluminium profiles are more sensitive to web crippling than the steel ones, which is also reflected by the reduction factor χ_F calculated in accordance with EC: it ranges within 0.9~1.0 for the steel and within 0.53~1.0 for the aluminium cases, respectively.
2) Strain hardening is considered in the aluminium calculations. Note that the manual calculation also accounts for the strain hardening in the bending moment resistance formulation, but not in the transverse load resistance.

The results thus promise that improved capacity values in the steel cases can possibly be achieved by implementing advanced (more accurate and realistic) non-linear simulation.



a) M-F interaction b) corner effect

Figure 3: Interaction results and corner effect.



a) load vs. deflection curves b) deformed web at ultimate load

Figure 4: Typical simulation results – steel, HEA300, a = 2 m.

4 MODIFICATION PROPOSAL FOR RESISTANCE CALCULATION

Based on the previous observations and parametric study results, the author proposes two simple modifications in the standard design method in order to utilize the advantageous effect of the curved connection configuration.

On the one hand, the curved configuration results in larger transverse plate bending rigidity of the web (analogous to the bending capacity of haunched girder – column connection). Consequently, it is supposed that – similarly to the calculation of the plate buckling due axial stresses – the clear web height h_w between the inner ends of the radius is used instead of the full web depth, as shown in Figure 5/a.

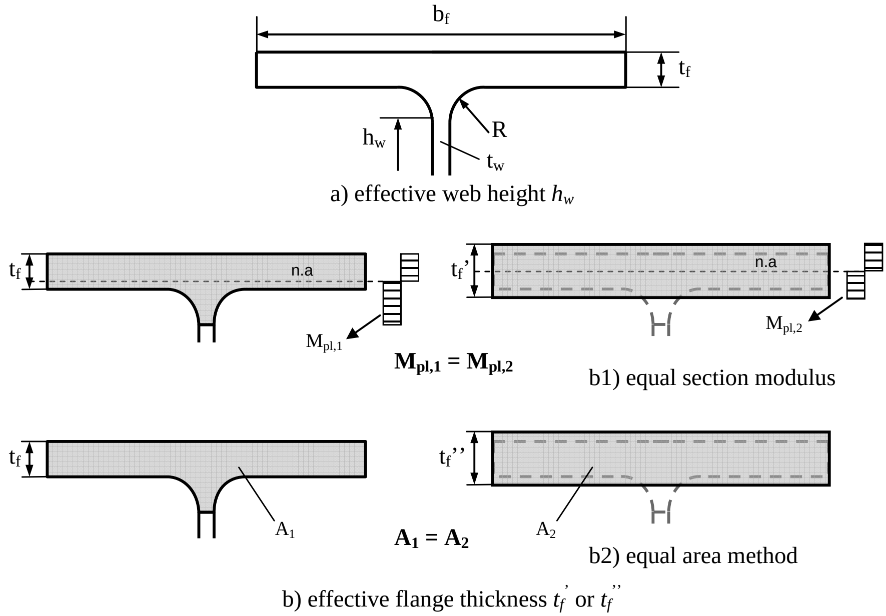


Figure 5: Determination of modified section properties.

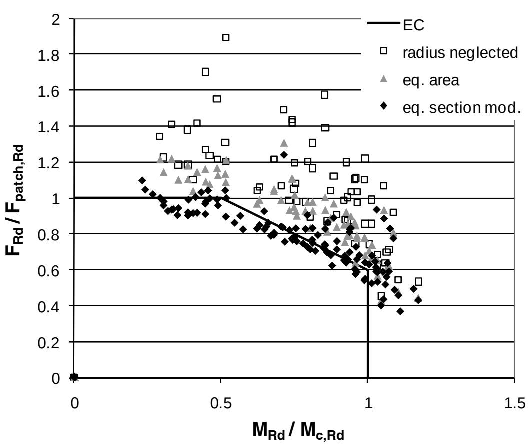


Figure 6: Comparison of methods.

On the other hand, the plastic hinge of the flanges occurring in the ultimate stage (plastic hinge mechanism) can extend to the curved corner area. Thus, the curved corner can be considered as part of the flange. Since the resistance against transverse loading is dominated by the flange plastic hinge, one has to calculate the plastic bending capacity of a fictive section including the flange and the accompanying corner area. Without rebuilding the existing design formula, this can be easily achieved by introducing an effective flange thickness providing the same local plastic capacity (Figure 5/b1). This is referred as *equal section modulus method* hereafter. As a simplification, the effective thickness can be conservatively calculated by simply smearing the curved corner area to the flange (Figure 5/b2, referred as *equal area method*). This latter method gives smaller effective thickness than the previous, more accurate one; consequently, it is always on the safe side.

Introducing these two modifications into the design method, the interaction relation shown in Figure 6 is obtained for the studied configurations. Regardless to the mentioned uncertainties (knife-edge load, etc.), it is concluded that the proposed modification gives more accurate evaluation of the patch load resistance.

5 CONCLUSIONS

Parametric study is completed on simply supported girders made of rolled steel or extruded aluminium profiles, subjected to transverse load. Based on the results, the following conclusions are found:

Influence of the curved-corner web-to-flange joint on the transverse load resistance can be significant in case of stocky webs; the increase in capacity may reach 60-90%.

To take this beneficial effect into account, the author proposed a simple modification in the current Eurocode design method. The modified procedure utilizes the clear web height and effective flange thickness; thus, more accurately representing the actual connection rigidity and plastic flange strength.

The proposed procedure well estimates the transverse load – bending interaction capacity computed by non-linear numerical simulation.

Further study is needed on the relation of the basic standardized procedure and the numerical simulation with respect to reliability. The research shall include study on the role of imperfection, knife-edge loading case, material modelling.

The method should be validated to other load application cases, as well.

Interaction of transverse load, bending and shear is additionally subject to further research.

REFERENCES

[1] *EN 1993-1-5:2005 Eurocode 3 – Design of steel structures – Part 1-5: Plated structural elements*, Final Draft, CEN, Brussels, 2005.

[2] *EN 1999-1-1:2007 Eurocode 9 – Design of aluminium structures – Part 1-1: General structural rules*, CEN, Brussels, 2007.

[3] Vigh, L.G., *Virtual and real test based analysis and design of non-conventional thin-walled metal structures*, PhD dissertation, Budapest University of Technology and Economics, 2006.

[4] Roberts, T.M., “Slender plate girders subjected to edge loading”, *Proc. Inst. Civil Engineers*, Part 2, Vol. 71, 805-819, Sept. 1981.

[5] Lagerqvist, O. and Johansson, B., “Resistance of I-girders to Concentrated Loads”, *Journal of Construction Steel Research*, **39** (2), 87-119, 1996.

[6] Kövesdi, B., Kuhlmann, U., Dunai, L., “Combined shear and patch loading of girders with corrugated webs”, *Periodica Polytechnica Civil Engineering*, 2010. (submitted for publication)

[7] *ANSYS Structural Analysis Guide*, Online Documentation ANSYS Inc., 2005.