STAINLESS STEEL GIRDERS –
RESISTANCE TO CONCENTRATED LOADS AND SHEAR

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Abstract

This paper is a summary of work performed in Sweden since 1998, addressing the resistance of welded I-girders made of stainless steel subjected to concentrated forces or shear.

The study on concentrated forces included five tests and a series of numerical simulations. The results from these were compared to the ultimate resistance calculated according to ENV1993-1-1:1992 and ENV 1993-1-5:1997. It was shown that ENV 1993-1-5:1997 is in better agreement with the results from this study than ENV 1993-1-1:1992, and can be used for predicting the resistance to concentrated loads.

The experimental part in the area of shear resistance comprised a total of eight tests made of two different steel grades. With support of numerical simulations of the tests and a parametric study, it was concluded that the design procedure with respect to shear in ENV 1993-1-4:1996, Supplementary rules for stainless steel, is too conservative. A design procedure which allows the shear resistance of welded stainless steel girders to be fully utilised was derived. It is similar to the procedure in ENV 1993-1-5:1997 and compared to the one currently in ENV 1993-1-4:1996 it is a large improvement.

1 INTRODUCTION

Concentrated forces introduced into the webs of girders are common in most steel structures, whether the structures are made of rolled beams, welded or cold formed girders. However, except for some few tests on girders made of aluminium [1] all research made in the past concerning I-shaped welded girders has been focused on carbon, carbon-manganese or quenched and tempered steel. Although research concerning the resistance for stainless steel cold-formed beams can be found in the literature, no studies have been published on the resistance to concentrated forces of welded girders made of stainless steel.

At the time when the study on shear buckling was performed, the only experimental work known to address shear resistance of stainless steel girders was the work by Carvalho et al. [2]. The results from that study origin from tests on cold-formed channel sections with unit aspect ratio, i.e. a shear panel depth equal to its length. These are the only results with which the design procedure originally presented in Design Manual for Stainless Steel [3] and subsequently also given in ENV 1993-1-4:1996 [4] was compared. The work by Carvalho et al. [2] was reviewed by the second author in [5] and it was concluded that the test results presented in [2] are not relevant for girder webs and that the theoretical analysis presented in [3] was based on too conservative assumptions.

2 EXPERIMENTAL WORK

2.1 General

The tests presented herein comprise five tests of the patch load resistance and eight tests of the shear resistance of welded I-girders. The girders subjected to patch loading were all made of the same steel grade whereas two steel grades were used for the shear tests. Steel grades used were the austenitic grade EN 1.4301 which has a relatively low strength and the austenitic-ferritic grade EN 1.4462 which has a fairly high strength. The former was used for both patch loading and shear buckling and the latter only for the shear tests. The mechanical properties were determined from a total of 24 uniaxial tensile coupon tests and stress-strain curves, proof stresses and ultimate strengths obtained can be found in [5]. It is though worthwhile to note that in the case of the 4 mm material used for the webs, the proof strength exceeded the nominal strength by as much as 36%.

In addition to the tests, the study comprised numerical studies which were used to extrapolate the test results. The numerical studies are however not presented in this paper but can be found in [6] and [5] respectively.

2.2 Patch loading resistance

The specimens used in the patch load tests were doubly symmetrical I-girders with the same nominal flange dimensions and the web slenderness h_w/t_w ranging from 46 to 107. The loaded length was given two different values, 40 mm and 80 mm. The dimensions of the girders are given in Table 1 with notations according to Figure 1.

Table 1 Dimensions of the girders used for the patch load tests. Notations according to Figure 1.

Girder	h_w [mm]	t_w [mm]	b_{f1} [mm]	b_{f2} [mm]	t_{f1} [mm]	t_{f2} [mm]	a [mm]	s_s [mm]
Pli 4301:1	238.3	4.1	118.9	118.1	11.8	11.7	998	40
Pli 4301:2	238.3	4.1	121.1	118.8	12.0	11.8	996	80
Pli 4301:3	316.0	4.1	121.9	120.1	11.9	11.9	1397	40
Pli 4301:4	438.4	4.1	121.4	121.1	11.9	12.0	1623	40
Pli 4301:5	400.9	8.8	120.4	120.5	12.0	12.0	1682	40

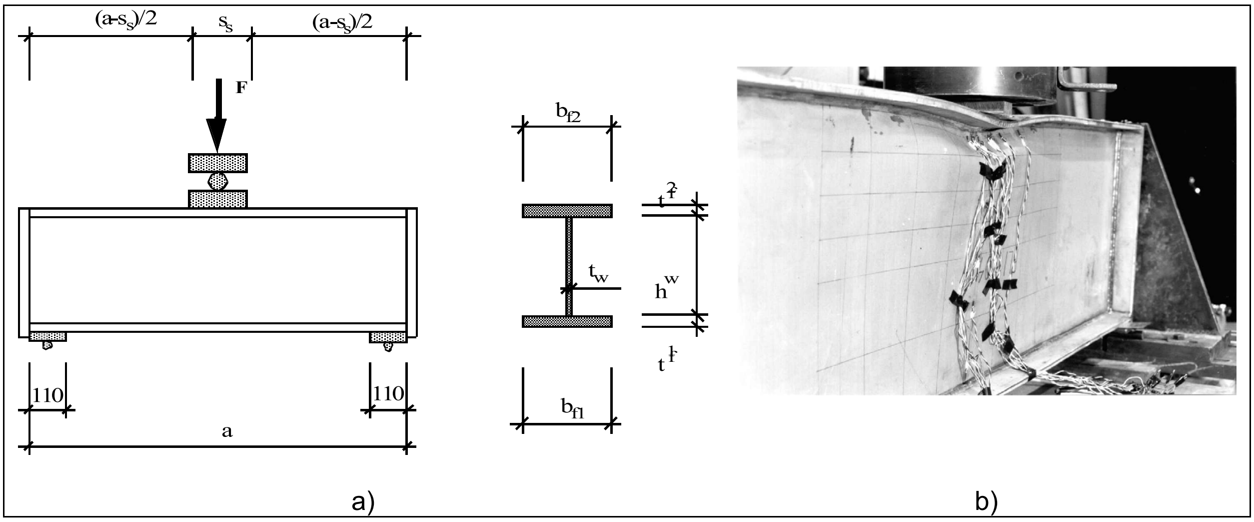

Figure 1 a) Specimen geometry used for the patch load tests. b) Girder Pli 4301:4 at ultimate load.

For the tests a rig with a hydraulic Instron actuator with a capacity of 1000 kN was used. The actuator was controlled through a Dartec control unit and the load history was registered. A constant stroke rate of 0.005 mm/s was used until the ultimate load was reached; thereafter the rate was increased to 0.05 mm/s. An extensive description of the tests is presented in [6].

2.3 Shear buckling resistance

All girders within the shear buckling study were doubly symmetric with the same nominal flange dimensions and web thickness and four different web depths. The web slenderness h_w/t_w was varied between 37.5 and 200 and the web aspect ratio a/h_w between 2 and 3. With notations according to Figure 2, the dimensions of the girders are given in Table 2.

Table 2 Dimensions of the girders used for the shear tests. Notations according to Figure 2.

Girder	h_w [mm]	t_w [mm]	b_{f1} [mm]	b_{f2} [mm]	t_{f1} [mm]	t_{f2} [mm]	a [mm]	l [mm]
SB 4301:1	146	4.0	200.2	200.2	11.9	11.9	449	1049
SB 4301:2	297	4.0	198.8	198.8	11.9	11.9	901	2100
SB 4301:3	597	4.0	199.8	199.9	11.9	11.9	1200	2998
SB 4301:4	793	4.0	200.8	201.2	12.3	12.3	1600	3997
SB 4462:1	148	4.0	199.6	199.6	13.1	13.1	450	1051
SB 4462:2	298	4.0	200.3	200.3	13.1	13.1	900	2100
SB 4462:3	597	4.0	202.7	200.2	13.0	13.0	1200	2996
SB 4462:4	795	4.0	202.1	196.5	13.0	13.0	1600	3997

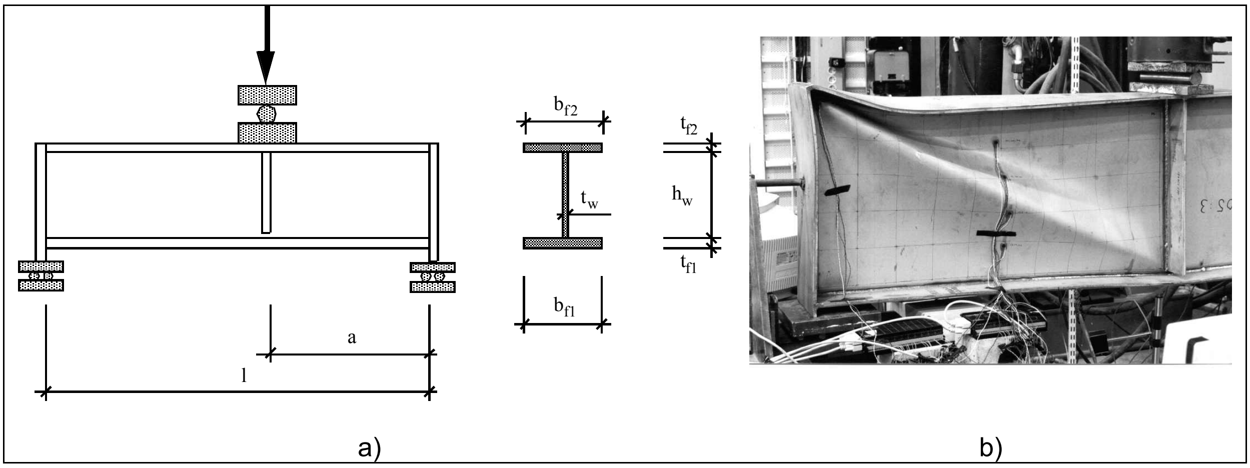

Figure 2 a) Specimen geometry used for the shear buckling tests. b) Girder SB 4462:3 at the end of the test.

All tests were performed in a tests rig with a hydraulic INSTRON actuator with a capacity of 1000 kN controlled by a DARTEC control unit. The load was applied at the upper flange through a 100 mm long steel plate. In order to avoid introduction of normal forces into the girder, roller bearings were used at both supports. A constant vertical displacement rate of 0.005 mm/s was used until the ultimate resistance was passed; thereafter the rate was increased to 0.025 mm/s. The tests are presented in detail in [5].

3 RESULTS AND COMPARISONS

3.1 Patch loading resistance

For the design resistance of welded girders with respect to concentrated forces, ENV 1993-1-4:1996 [4] refers to ENV 1993-1-1:1992 [7]. In the Swedish NAD to ENV 1993-1-4:1996 [4], the reference to ENV 1993-1-1:1992 [7] is however changed to ENV 1993-1-5:1997 [8]. The test results are in Table 3 compared with predicted resistances according to ENV 1993-1-1:1992 [7] and ENV 1993-1-5:1997 [8]. According to the former, the resistance should be taken as the smaller of the crushing resistance and the crippling resistance. The design model presented in ENV 1993-1-5:1997 [8] incorporates three parts, an expression of the yield resistance, the elastic buckling load and a resistance function.

Table 3 Comparison between ultimate patch load resistance and predicted resistance according to ENV 1993-1-1:1992 [7] and ENV 1993-1-5:1997 [8].

Girder	λ	Test	ENV 1993-1-1:1992		ENV 1993-1-5:1997	
		F_u [kN]	F_R [kN]	F_u/F_R	F_R [kN]	F_u/F_R
Pli 4301:1	0.883	176	121	1.45	141	1.25
Pli 4301:2	0.970	196	133	1.47	155	1.26
Pli 4301:3	1.052	168	119	1.41	127	1.33
Pli 4301:4	1.298	169	117	1.44	113	1.49
Pli 4301:5	0.494	478	254	1.88	344	1.39

As can be seen in Table 3 the design procedure presented in 1993-1-5:1997 [8] gives a better prediction of the ultimate load than the method in ENV 1993-1-1:1992 [7].

3.2 Shear buckling resistance

The results from the shear buckling tests are presented in Table 4. These results are also compared with resistances calculated according to ENV 1993-1-4:1996 [4] and ENV 1993-1-5:1997 [8].

Table 4 Comparison between ultimate shear resistance and predicted resistance according to ENV 1993-1-4:1996 [4] and ENV 1993-1-5:1997 [8].

Girder	λ	Test	ENV 1993-1-4:1996		ENV 1993-1-5:1997	
		V_u [kN]	$V_{b,R}$ [kN]	$V_u/V_{b,R}$	$V_{cwf,R}$ [kN]	$V_u/V_{cwf,R}$
SB 4301:1	0.435	178.5	78.7	2.27	120.2	1.49
SB 4301:2	0.919	190.0	123.3	1.54	195.0	0.97
SB 4301:3	1.799	226.1	174.0	1.30	207.5	1.09
SB 4301:4	2.401	242.3	189.4	1.28	203.8	1.19
SB 4462:1	0.612	269.1	133.9	2.01	235.0	1.15
SB 4462:2	1.280	294.5	201.9	1.46	283.3	1.04
SB 4462:3	2.499	366.3	265.6	1.38	307.6	1.19
SB 4462:4	3.343	388.0	280.3	1.38	298.4	1.30

It can be seen in Table 4 that the design procedure in ENV 1993-1-4:1996 [4] gives an underestimation of the shear resistance for the tested girders, especially for the ones with a stocky web. It can also be noticed that the design procedure for structural steel in ENV 1993-1-5:1997 [8] overestimates the resistance for one of the girders.

4 DEVELOPMENT OF DESIGN GUIDANCE

4.1 Patch load resistance

With the experimental and the numerical results as a basis, the use of the design procedure presented in ENV 1993-1-5:1997 [8] is recommended. The resistance to concentrated loads is given by

$$F_R = F_y \chi(\lambda)$$

where: F_y is the yield resistance; $\chi(\lambda)$ is a resistance function. These are given by

$$F_y = f_{yw} t_w l_y$$

$$\chi(\lambda) = \frac{0.5}{\lambda} \leq 1$$

where: f_{yw} is the yield strength; t_w is the web thickness; l_y is the effective loaded length; λ is the slenderness.

In Figure 3 the ultimate resistance from tests and numerical simulations divided by the yield resistance are depicted as a function of the slenderness. The resistance function presented in ENV 1993-1-5:1997 [8] is also included as a reference.

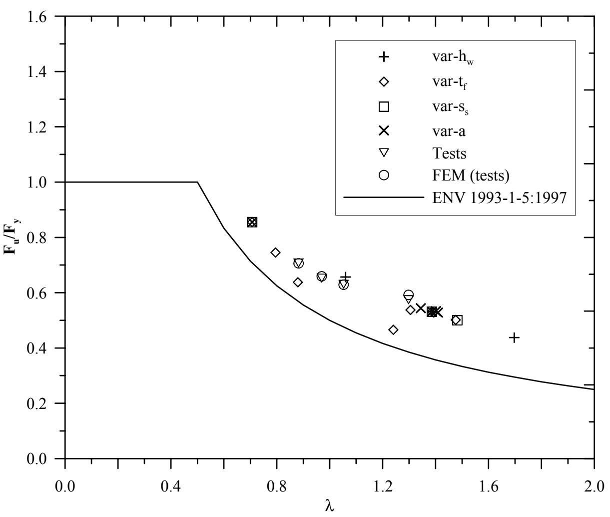

Figure 3 Ultimate resistance/yield resistance as a function of the slenderness, λ. The resistance function from ENV 1993-1-5:1997 [8] is also included.

A statistical evaluation of the patch load resistance according to Annex D of EN 1990:2002 [9] was performed resulting in a corrected partial factor of 1.03. Hence this appraisal supports the use of ENV 1993-1-5:1997 [8] for prediction of the resistance. The full statistical evaluation can be found in [6].

Besides the tests and simulations of the patch load resistance, the influence of a simultaneous bending moment was studied by numerical simulations, see [6]. In Figure 4 the results from this study as well as from the tests are shown as F_u/F_R versus M_S/M_R, with F_R according to ENV 1993-1-5:1997 [8] and M_R according to ENV 1993-1-4:1996 [4]. The interaction formula given in ENV 1993-1-5:1997 [8]

$$\frac{F_S}{F_R} + 0.8\frac{M_S}{M_R} = 1.4$$

is included to get a comparison of the results.

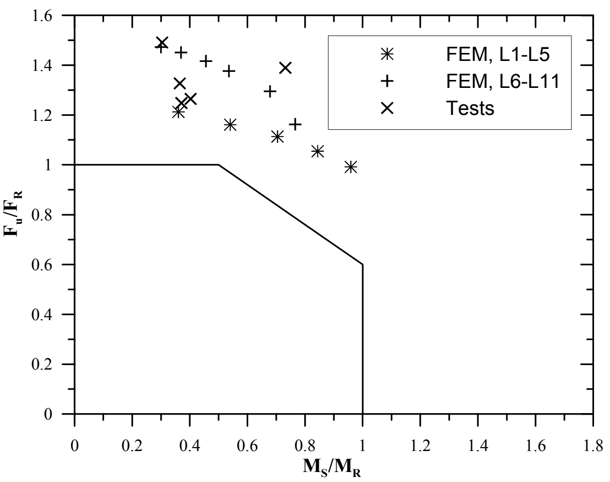

Figure 4 F_u/F_R versus M_S/M_R with the interaction formula given in ENV 1993-1-5:1997 [8] included.

4.2 Shear buckling resistance

Based on the tests and the numerical simulations the design shear resistance of stainless steel is proposed to be given by

$$V_{cR} = (\chi_w + \chi_f) f_{yw} h_w t_w / \sqrt{3}$$

which actually is the same as Equation (4.20) in ENV 1993-1-5:1997 [8]. The contribution in this proposal is the modification of the functions χ_w and χ_f. The contribution from the web χ_w is given in Table 5. The theoretical maximum value of the function χ_w is 1.0 if $R_{p0.2}$ is taken as the yield strength. In the case of stainless steels it is however justified to utilise strain hardening which is why χ_w is proposed as 1.2 for slendernesses below 0.5. A thorough description of the development of the design guidance can be found in [5].

Table 5 Contribution from the web χ_w to the shear buckling resistance.

$\bar{\lambda}_w$	χ_w
≤0.50	1.2
>0.50	$0.11 + \frac{0.64}{\bar{\lambda}_w} - \frac{0.05}{\bar{\lambda}_w^2}$

If the flanges are not fully utilised to withstand bending, i.e. $M_S < M_{fR}$, a contribution from the flanges may be included according to

$$\chi_f = \frac{b_f t_f^2 f_{yf} \sqrt{3}}{c t_w h_w f_{yw}} \left[1 - \left(\frac{M_S}{M_{fR}} \right)^2 \right]$$

where the distance c between the plastic hinges in the flanges is given by

$$c = a \left[0.17 + \frac{3.5 b_f t_f^2 f_{yf}}{t_w h_w^2 f_{yw}} \right]; \quad \frac{c}{a} \le 0.65$$

In Figure 5 the predicted contribution from the web according to Table 5 is given together with test and numerical results. Predictions according to ENV 1993:1-1:1992 [7] (simple post critical method), ENV 1993:1-4:1996 [4] and ENV 1993:1-5:1997 [8] are also included. It can be seen that resistance according to Table 5 is in better agreement than the different design codes.

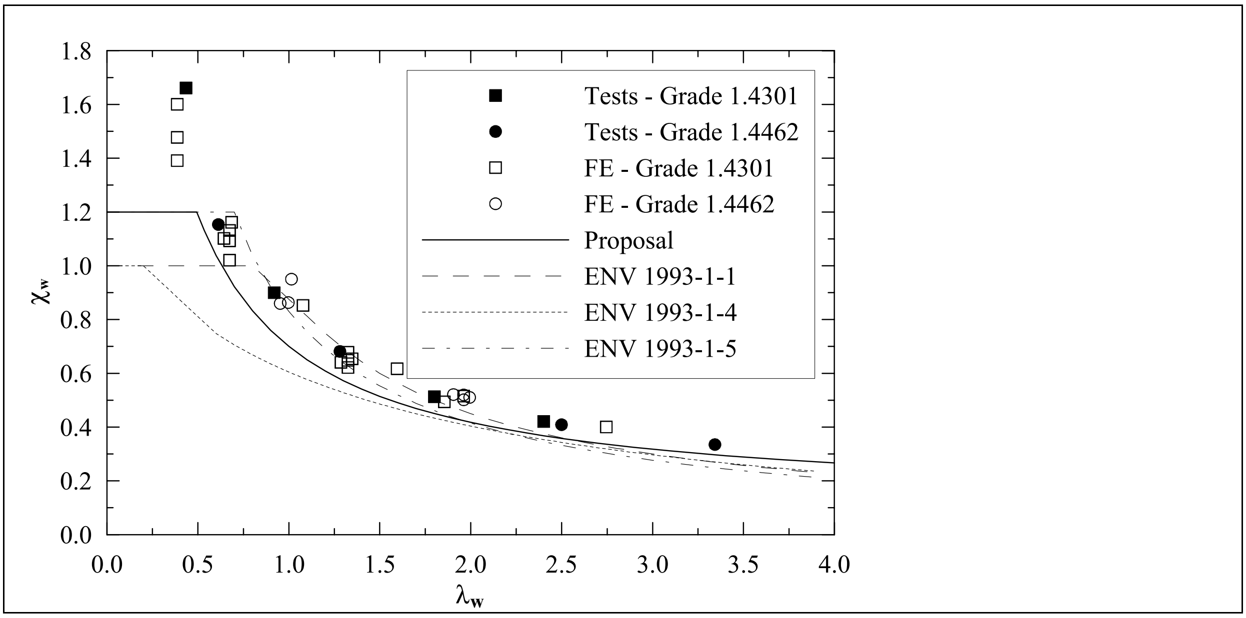

Figure 5 Test and numerical results reduced with the predicted contribution from the flanges compared with the proposed design curve. Design curves according to ENV 1993-1-1:1992 [7], ENV 1993-1-4:1996 [4] and ENV 1993-1-5:1997 [8] are given as references.

5 DISCUSSION AND CONCLUSIONS

The experimental and numerical studies on patch loading support the use of ENV 1993-1-5:1997 [8] instead of ENV 1993-1-1:1992 [7] concerning the resistance to concentrated forces for the case when the load is applied far from an unstiffened girder end. The parametric study showed that the design procedure in ENV 1993-1-5:1997 [8] well considers the flange thickness, the length of the loading plate and the width of the web panel. Still, the effect of the web slenderness on the resistance remains to be improved.

When girders are subjected to a concentrated load and a simultaneous in-plane bending moment, the interaction between the two load effects has to be considered. The results from this study showed that the formula proposed in ENV 1993-1-5:1997 [8] well describes the effect of a combination of patch loading and in-plane bending moment.

For the shear resistance of stainless steel girders, it was found that the hitherto available design procedure underestimated the ultimate resistance obtained in the tests performed. A comparison between the obtained test results and design codes for structural steel resulted in the conclusion that the mechanical properties of stainless steel are such that a specific design procedure with respect to shear resistance is needed for stainless steel. The FE-study performed to calibrate and extrapolate the test results was taken as satisfactory, even though it (with the exception of the stocky girders) gave a small over prediction of the shear resistance.

The design procedure derived from the obtained results is similar to the one in ENV 1993-1-5:1997 [8]. The features of the proposed procedure that differ from the one currently in ENV 1993-1-5:1997 [8] are the reduction function and the contribution from the flanges. It is concluded that the proposed tentative design curve with respect to the shear buckling resistance is a large improvement compared with the one currently in ENV 1993-1-4:1996 [4]. Furthermore it is concluded that the contribution of the flanges is a matter that needs further research.

Conclusions drawn from these studies are:

- Test results from, and numerical simulations of, girders subjected to concentrated loads show that ENV 1993-1-5:1997 gives a good prediction of the resistance and therefore, instead of ENV 1993-1-1:1992, should be used as design procedure.
- The interaction formula for concentrated loads and a simultaneous bending moment in ENV 1993-1-5:1997 accounts for the influence of the bending moment.
- Test results from, and numerical simulations of, girders subjected to shear show that the hitherto available design procedure is too conservative. The design procedure with respect to shear resistance presented is a large improvement compared with the currently available design procedure. It can be used to predict the shear resistance of stainless steel girders.

6 REFERENCES

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