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LATERAL-TORSIONAL BUCKLING OF WELDED SLENDER STAINLESS STEEL I-BEAMS

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Abstract: *The paper is focused on welded slender stainless steel beams subjected to major bending moment. Difference in behaviour of stainless steel and common carbon steel is generally known, but design of stainless steel members is established mainly for hollow sections (CHS, SHS/RHS) as these are typical stainless steel profiles. Currently, open sections are also being used in structures and the design rules for both local buckling of very slender section as well as lateral-torsional buckling curves are based on very limited experimental and numerical research especially cross section Class 4. For this reason, six beams made of two types of stainless steel with different slenderness for lateral-torsional buckling were tested. Additionally, numerical GMNIA models of these beams are successfully validated. A parametric study based on the validated models will be used for development of a new accurate and safe design procedure.*

Keywords: Stainless steel, Lateral-torsional buckling, Local buckling, Slender beam

1. Introduction

Over the last several decades, popularity in use of stainless steel for structures increases. This material is a specific type of steel which is highly alloyed containing more than 10,5% of chromium. Austenitic, ferritic and duplex stainless steel are the most used in structural applications. These groups of materials are different with respect to strength, ductility, weldability, toughness and the ability to resist corrosive environment as result of using various alloying elements in varying amount. The main reason for limited use is the initial cost of the material, which differs significantly for each group of stainless steel and is much higher than for common carbon steel.

The main difference from common carbon steel is in behaviour during loading which is evident from stress-strain diagram. Instead of typically linear behavior up to a visible yield strength for carbon steel, the stress-strain curve for stainless steel has a more rounded response with no clearly defined yield point. The material non-linearity is the reason for other design procedures for stainless steel structures as the stiffness is reduced by yielding below the 0.2 % (yield) proof strength and strain hardening is usually much higher.

Generally, open cross-sections subjected to bending around the major axis with unrestrained or partly restrained compressed flange or compressed web tend to fail with influence of lateral torsional (global) buckling. Whereas slender cross section resistance may be governed by plate (local) buckling. Both phenomena have strong influence on resistance of steel beams.

Specifically, the resistance of beam subjected to lateral-torsional buckling should be determined according to EN 1993-1-4, where supplementary rules are given for structural stainless steel. The design requirements are the same as for carbon steel, only an imperfection factor α_{LT} for determining the reduction factor χ_{LT} for lateral torsion buckling is different. However, there are no experimental data supporting the codified value of the factor α_{LT} for slender welded open section.

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2. Experiments

The prepared experimental program consisted of 6 four point bending tests. There was selected only one cross section (Class 4) made of two stainless steel grades and with three different lengths (2.4, 3.8 and 5.4 m span). Summary of beams is given in Fig. 1, where the section geometry, section flange/web classification, section and beam slenderness are specified.

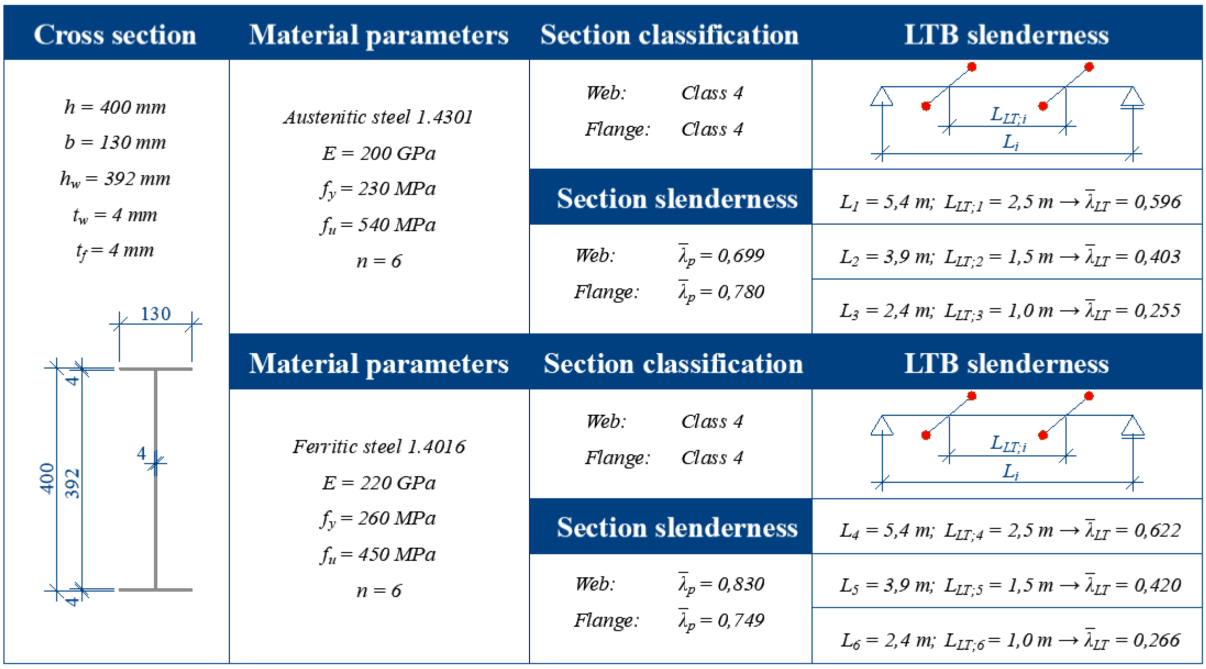

Fig. 1: Experimental program overview.

It can be seen that the test arrangement was identical for all beams, only the beam span and the distance between lateral restraints was different. The beam was supported at its ends under the lower flange. Lateral restraints were used at ends and points of loading for both the upper and lower flange (Fig. 2).

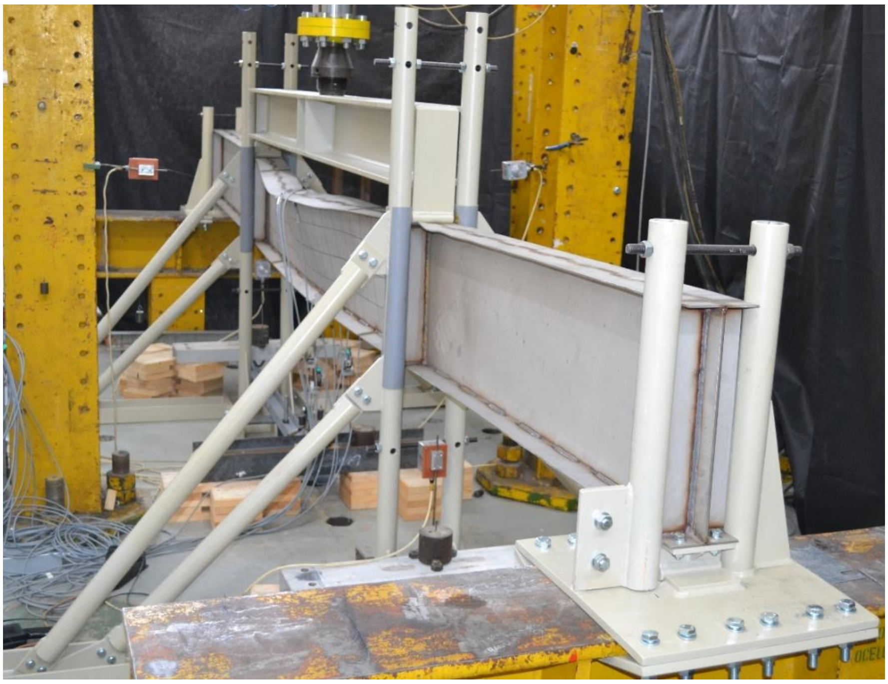

Fig. 2: Test arrangement and beam failure mode (test 1).

Before each test, the initial local and global imperfections were measured. Several material tests were also carried out, measured material properties are shown in Tab. 1. A two-stage Ramberg-Osgood material diagram was used for description of the stress-strain characteristic and used for validation of numerical models.

Tab. 1: Material properties of the tested specimen.

Type of steel	Yield strength [MPa]		Ultimate strength [MPa]		Modulus of elasticity [GPa]		R-O parameter [-]		Parameter $n`_{0.2.1.0}$ [-]	
1.4016	307.016		430.698		201.371		8.10		1.80	
1.4016	-	306.83	-	429.36	-	202.86	-	7.90	-	1.79
1.4016	306.642		428.029		204.343		7.70		1.77	
1.4301	297.787		637.413		195.693		5.60		2.30	
1.4301	297.345	297.99	622.003	632.50	197.339	196.07	5.50	5.63	2.35	2.34
1.4301	298.834		638.079		195.188		5.80		2.37	

3. FE model validation

The geometrically and materially nonlinear analysis with imperfection (GMNIA) was created in software Abaqus using shell element S4R (four node shell element with reduced integration). Section resp. member imperfections were assumed by the lowest elastic buckling eigenmode for local resp. global buckling. The amplitudes of imperfections were taken as measured for each specimen. Residual stresses due to welding were also considered in the numerical models, but showed very small influence on the beam resistance (Fig. 4).

In Tab. 2 is shown the comparison of the beam response from experiment, FE model and from the procedure according to EN1993-1-4 with using measured material properties. Failure modes for test 1 and test 3 are shown in Fig. 3 as well.

Tab. 2: The beam resistance comparison.

Test [-]	Material [-]	L_i [m]	$L_{LT,i}$ [m]	$M_{b,Rd}$ [kNm]				
				Test	FE model		EN 1993-1-4	
1	1.4301	5400	2500	76.1	77.0	1.2%	49.0	35.6%
2	1.4301	3800	1500	89.8	75.8	15.7%	62.2	30.8%
3	1.4301	2400	1000	90.8	77.8	14.4%	69.3	23.7%
4	1.4016	5400	2500	83.3	79.4	4.6%	50.6	39.3%
5	1.4016	3800	1500	98.7	79.9	19.0%	64.2	35.0%
6	1.4016	2400	1000	97.4	81.9	15.8%	71.5	26.6%

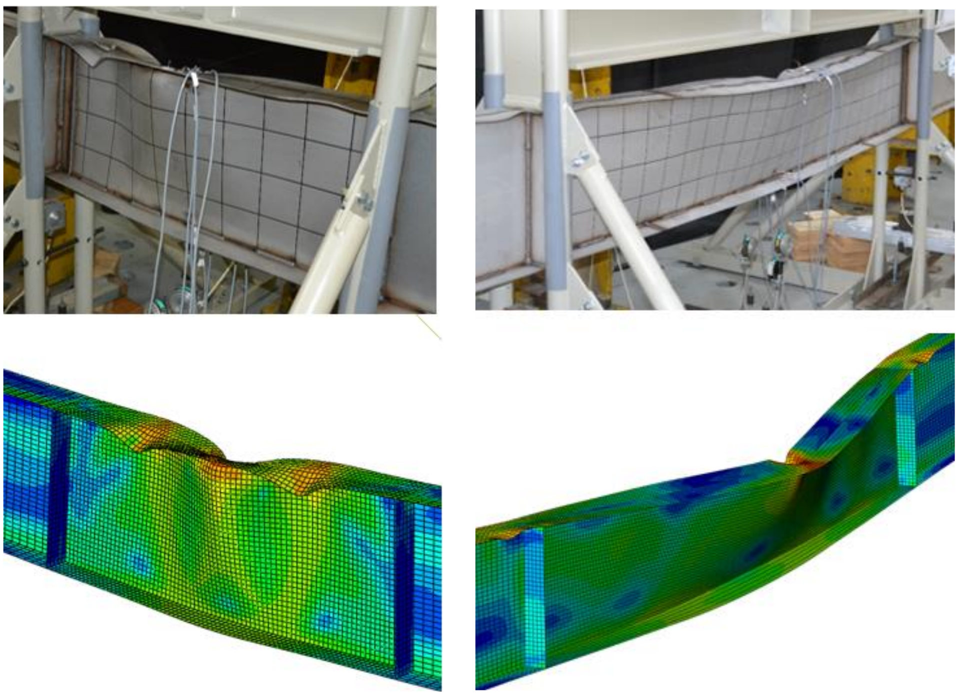

Fig. 3: Test and FE model failure mode (test 3 and test 1).

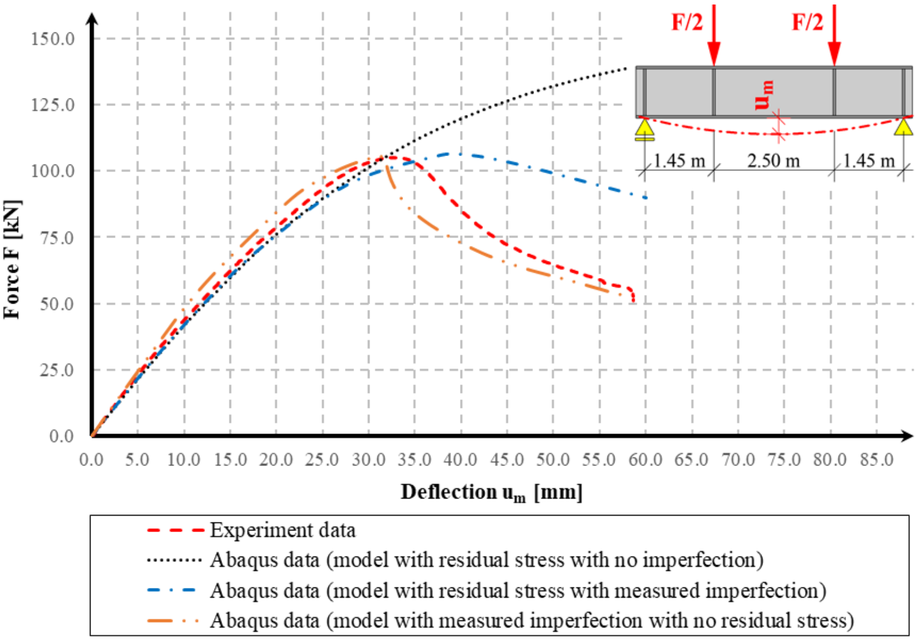

Fig. 4: Validation of the test 1(stainless steel 1.4301; L= 5.4 m).

Generally, the numerical model allow to predict the real behavior well as the predicted resistance is in average 11,8% lower. In case of the longer beams, the prediction was also very accurate. For shorter beams, where the resistance is more influenced by local imperfections, the differences between experimental and predicted curves are bigger. The scatter is contributed to the simplification in the local imperfection shape being considered by the first elastic buckling eigenmode. The real shape of imperfection is more favourable for the beam resistance. More precise imperfection modelling is planned in the future.

An analysis of the results indicated on differences between values of bending moment followed in the test, FEA and standards. It seems that the experiment is not trivial and requires improvement for further examination of the beam type behaviour.

4. Conclusions

This paper is focused on behaviour of slender open section stainless steel beams loaded by major bending moment. Numerical models created in software Abaqus using GMNIA were successfully validated based on the experimental data. Furthermore, the comparison of results obtained according to existing design procedure was made. In the next step, the procedure of cross section resistance in bending as well as the lateral-torsional buckling curves will be investigated in a numerical parametric study.

Acknowledgement

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