ON DEVELOPMENT OF NUMERICAL RESISTANCE MODELS OF THIN-WEB STEEL GIRDERS

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DOI: 10.35181/tces-2023-0003

Abstract. *Thin-web steel girders are attractive to be used because of their efficiency in bending. For such girders, the issue of ensuring local buckling becomes very relevant. Calculation formulas in most cases are complex and have a limited scope of application. The calculations based on numerical models make it possible to consider all the specifics of the designed element more universally. The article deals with the calculation of the buckling of the web girder under the combination of patch and shear loading by finite element modelling. Numerical models have been created, and a comparative analysis with experimental results has been carried out. The presented principles for constructing FE models (mesh size, material model, etc.) are recommended to follow when analysing the resistance and behaviour of beams with a thin web. Sensitivity analysis of the FE model with respect to the input parameters revealed the most important parameters (yield strength of steel, web thickness, geometry), the uncertainty of which needs to be taken into account when creating FE models. The convergence of the results supports the use of the finite element method in the design of steel beams for a qualitative and quantitative assessment of resistance. However, further development of unified principles for creating FE models and their verification on a larger amount of experimental data is required, as well as the determination of partial factors considering the variability and uncertainties in the obtained results and specified reliability level.*

Keywords

Web panel buckling, critical force, discretization, finite element method, imperfections, modelling, numerical model.

1. Introduction

Thin-web steel beams attract with their efficiency when subjected to bending. However, the issue of ensuring local buckling becomes very relevant for such beams. Many theoretical and experimental works study the resistance of such beams, but in most cases, they are devoted to the investigation of the behaviour of such elements under the influence of individual force factors (load effects), such as bending, shear, and patch loading. Nevertheless, in practice, the application of a combined shear and patch loading to beams with a thin web is often encountered. An example of such a resistance model is a steel bridge girder in the process of sliding (launch) onto supports or a crane girder for an overhead crane, in which the web of the crane girder is not only locally loaded but also subjected to a shear force.

This combination of actions on welded I-beams with a thin web was experimentally investigated by Roberts T. M. and Shahabian F. [1, 2], followed by Braun B. [3]. These experiments showed a significant interaction between shear force and patch loading. As a result, the interaction formula and the modification of the reduced stress method and the effective width method described in EN 1993-1-5 under combined loading were proposed [4, 5]. However, the use of analytical formulas is limited by the complexity and area for which they have been experimentally confirmed. As a rule, due to the complexity of the deformation process of a steel beam taking into account the postcritical behaviour in the case of loss of local buckling of the web, the models for ultimate resistance are conservative [6-8]. The use of numerical methods, in particular the finite element method, is effective for complex resistance models. According to [4], numerical methods may lead to a more accurate description of behaviour and estimates of ultimate resistance but manuals for specification of the parameters of the models and interpretation of the results are missing. The last decade revealed a growing interest in the use of computer (numerical) models for the analysis of structural resistance. While several studies [9-14] showed a good correspondence between experiments and FE models, widely accepted principles for creating FE models are missing; this is an obstacle for their practical applications. As a consequence, it is of interest to analyse the behaviour of steel beams based on numerical modelling and compare the results with experiments. The application of FE models involves three basic steps:

- development and unification of principles and parameters for FE models – the main focus of this study,

- evaluation of the accuracy of results based on comparison with experimental data – discussed briefly [15],
- determination of reliability parameters such as partial and sensitivity factors [16].

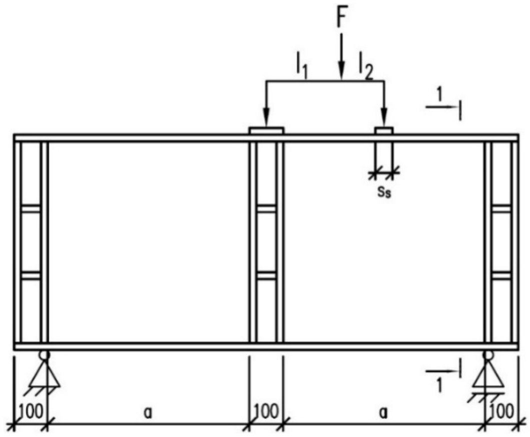
The article presents an overview of the principles and main parameters for constructing FE models (such as FE mesh size, material model, etc.), as well as an analysis of their influence on the result of the FE model, based on which recommendations were made on the methods of setting and parameter values of FE models for thin-web steel structures. Sensitivity analysis of the FE model to the input parameters (mechanical properties of steel, geometric dimensions) determined the set of parameters that most significantly affect the result of the calculation and for which further research is needed. Comparison with experimental data shows the efficiency and accuracy of calculations based on FE models.

2. FE Models

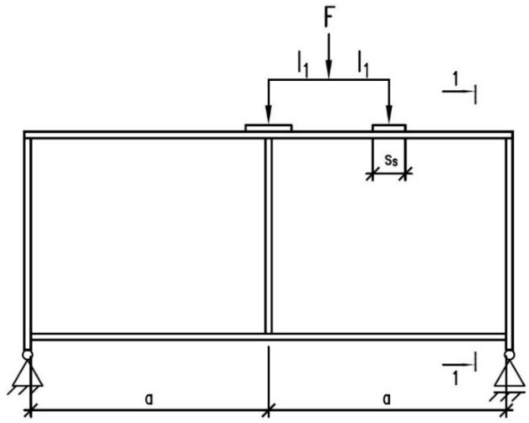
At present, numerical modelling is often used because of the lack of the possibility of carrying out a full-scale experiment or for a parametric study of the effects of various parameters. This makes it possible to evaluate the ultimate resistance of completely new structural solutions for which there are no calculation methods (models). In this work, numerical modelling is done by the finite element method using the software Abaqus. The creation and study of the accuracy of the application of FE models for ultimate resistance of thin-web welded beams is based on the results of experimental studies, which were published by Roberts T. M. and Shahabian F. [1] and Braun B. [3]. The beams marked PG1-2 and PG4-2 from [1] and the beams SP600 and SP1200 from [3] are adopted (the original designations are kept in this study). Schemes of the modelled types of beams are provided in Fig. 1.

In order to eliminate possible influence of experimental error, the test cases were taken from different sources. Since spacing of stiffeners is assumed to have a dominant effect on the behavior of the beams and influences a failure mode, the beams are selected so as to consider different cases of the ratio of the step of the stiffeners a to the height of the web, h_w. For beams SP600 and SP1200 the distance between the stiffeners is more than the height of the web, and for beams PG1-2 and PG4-2 - less than h_w.

FE models of the beams were created from measurements of the ***dimensions*** and ***mechanical*** characteristics of the tested beams. Table 1 provides the geometric parameters and the mechanical properties of the steel of the beam elements. The thickness of the stiffeners and base plates for the beams PG1-2 and PG4-2 are 10 mm. Yield strength values are obtained from averaged uniaxial tensile test data for each beam (three samples for the web and two for the flanges). Ultimate strength of steel was not measured in [1]. This value is taken as equal to $f_u = 1.35\,f_y$ according to [17]. Due to the lack of data, generally accepted values are considered for elastic modulus, $E = 210$ GPa, and Poisson's ratio, $\nu = 0.3$.



(a)



(b)

Fig. 1: (a) Scheme of beams PG1-2 and PG4-2, (b) Scheme of beams SP600 and SP1200.

Stiffeners and base plates for beams SP600 and SP1200 have the same width and thickness as the flanges. Two tests are made from the same batch of steel. Mechanical properties of steel were obtained from the averaged data of uniaxial tensile tests of specimens (three tests each for flanges and webs). The elastic modulus for the web is equal to $E_w = 177$ GPa, and for the flanges and stiffeners is equal to $E_f = 186$ GPa. Poisson's ratio is $\nu = 0.3$.

Tab.1: Geometrical parameters and mechanical properties of steel elements.

Beams	SP600	SP1200	PG1-2	PG4-2
a, mm	2390	2390	600	500
h_w, mm	600	1200	600	1000
t_w, mm	6	6	4.1	1.9
b_f, mm	450	450	200	200
t_f, mm	20	20	12.3	9.8
s_s, mm	200	200	50	50
f_{yw}, MPa	383	383	339	247
f_{uw}, MPa	543	543	-	-
f_{yf}, MPa	354	354	250	313
f_{uf}, MPa	519	519	-	-
E_w, GPa	177	177	210	210
E_f, GPa	186	186	210	210

The strength and deformation properties of steel taken into account by a *stress-strain curve* have a dominant effect on the ultimate resistance of steel structures. The resistance of thin-web elements can be affected by the strain hardening of steel as in the plate, secondary bending stresses occur in addition to the primary stresses in the membrane. Considering the linear part of material model only may lead to the loss of bending stiffness of the plates. The following material models were considered in the analysis of the observed effects: elastic-plastic without strain hardening, elastic-plastic with linear strain hardening, and the quadrilinear relationship according to Swedish standard BSK07 [18] (Fig. 2).

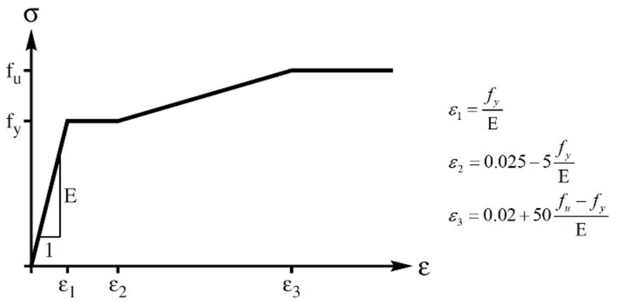


Fig. 2: Stress-strain curve according to BSK07.

Figure 3 shows a comparison of the load-displacement curves for the SP600 and PG1-2 beams depending on the selected material model. It can be seen that a lower value of the ultimate load is reached in the stress-strain curve without strain hardening. However, the difference between the results of the FE models is about 1%, which corresponds to the fact that all material models apply to solve this problem. In the following analysis, the quadrilinear relationship according to BSK07 is applied.

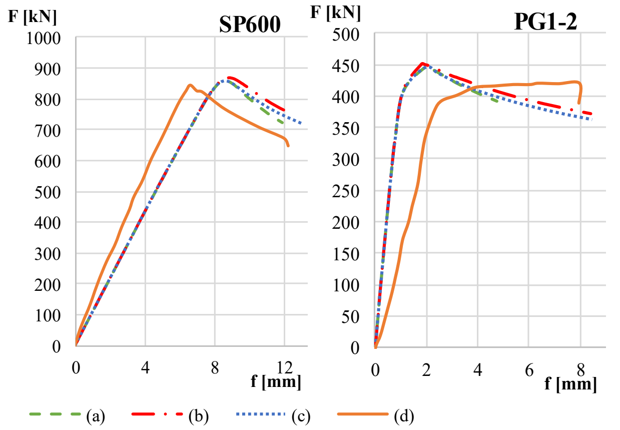


Fig. 3: Comparison of load-displacement curves depending on selected material model (a) Elastic-plastic without strain hardening, (b) Elastic-plastic with linear strain hardening, (c) Quadrilinear relationship according to BSK07, (d) Experimental data.

The loading plates are modelled by the "rigid" element. The interaction between the loading plates and the top flange is made through a «surface-to-surface» type of contact with the presence of friction. Loads are applied as static loads, loading is performed by a concentrated load through the reference point to the loading plate. The boundary conditions of the base plates are simply specified as a pinned and roller support.

Discretization (finite element size) was performed by the preliminary analysis for the choice of the finite element size. Figures 4 and 5 show the graphs of the influence of the size of finite-element on the value of the critical buckling force (F_{cr}) and the ultimate force (F_u), taking into account geometric and physical nonlinearity for the SP600 and PG4-2 beams. As a result, the most optimal size from the point of view of the solution accuracy and the use of computing power was adopted a finite element whose dimensions are five beam web thicknesses (20 mm for the PG1-2, 10 mm for the PG4-2, and 30 mm for beams SP600 and SP1200).

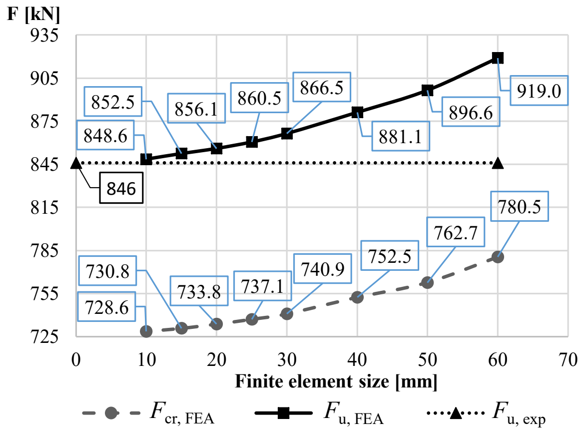


Fig. 4: Analysis of the finite element size for the SP600 beam

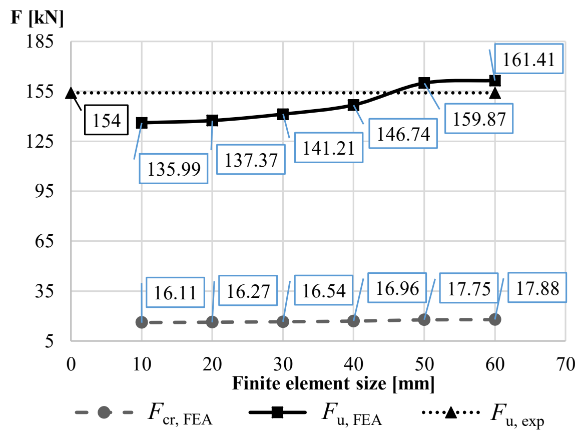


Fig. 5: Analysis of the finite element size for the PG4-2 beam.

In the manufacture and construction of real structures, ***imperfections*** are inevitable. In general, geometric and structural imperfections are distinguished. Geometric imperfections include, for example, initial deflections, curvatures, eccentricities, and tolerances of deviations from nominal geometric values. Structural imperfections include, for example, residual stresses caused by the manufacturing process.

Since the amplitude and distribution of residual stresses significantly depend on the manufacturing method and the shape of the cross-section, the assignment of these imperfections is difficult to automate, while there is not enough experimental data in which welding stresses were determined and their effect on the behaviour and resistance of the element was analyzed; therefore, the use of ***equivalent geometric imperfections*** can be recommended to account for welding stresses. This approach is used to calculate the stability of the common elements according to EN 1993-1-1 [4], a similar approach is used for the resistance models under consideration. The choice of this approach is also supported by research [19-21], in which the insignificant influence of welding stresses is observed on the results of calculations using FE models.

The most difficult task in specifying imperfections is choosing the shape, and amplitude (value). Based on recommendations [22-26] two options for taking into account imperfections are considered – based on eigenforms of elastic buckling and based on equivalent geometric imperfections (Eigenmode-affine imperfections and Manually defined imperfections). The shape of equivalent imperfections is assumed to be half sinusoidal with a deflection value equal to min (a/200, h_w/200) [4], so that for the beams SP600, PG1-2, PG4-2 the deflection was 3 mm, and for the beam SP1200 it was 6 mm.

The methodology of imperfections modelling based on eigenforms of buckling is very simple: it is necessary to perform a linear calculation of buckling of one model to determine the eigenforms of buckling, then write down the results of displacement of it and use them as the initial imperfections of the second model of the same beam when analysing its geometrically and physically non-linear work. The value of imperfections is taken as equal to that of equivalent imperfections.

Figure 6 shows a comparison of the load-displacement curves for the SP600 and PG1-2 beams depending on the method of defining imperfections, where it can be seen that the two methods apply to the task at hand. The difference between the results of the ultimate load was approximately 3%.

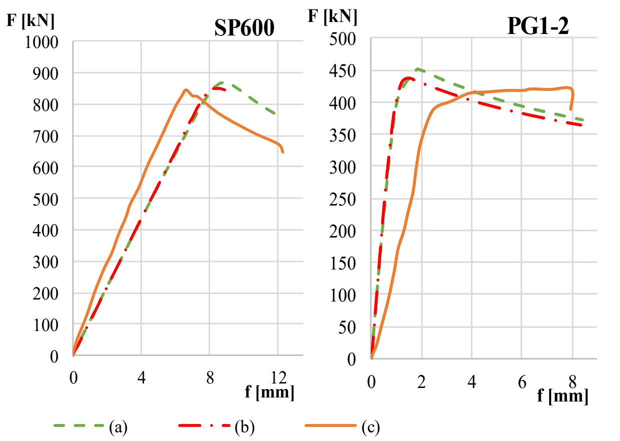


Fig. 6: Comparison of load-displacement curves depending on the method of defining imperfections (a) Eigenmode imperfections, (b) Manually defined imperfections, (c) Experimental data.

Common to all cross-sectional elements of numerical models is that the dimensions are large in two directions and small in the third direction. Thus, the geometry can be idealised as an average surface, which is then subdivided into shell elements. When calculating thin plates, in addition to the non-linearity of the material, large deformations contribute to the occurrence of geometrically non-linear effects. As the basic element for modelling the section of the beams, four-node shell finite elements with a bilinear shape function, denoted as S4R, were selected.

A solid twenty-node element, designated as C3D20R, was selected for the comparative analysis, and similar FE models were created. Figure 7 shows a comparison of the load-displacement curves for the S600 and PG1-2 beams depending on the selected element type. It can be seen from the two plots that the best convergence of the results of the FE model with the experimental data is achieved when using a solid FE. It was found that the difference between the results of the ultimate load between the FE models was about 2%. So it is more appropriate to use the shell S4R element, than an element of the solid C3D20R type, because modelling is more laborious and more computing power is used.

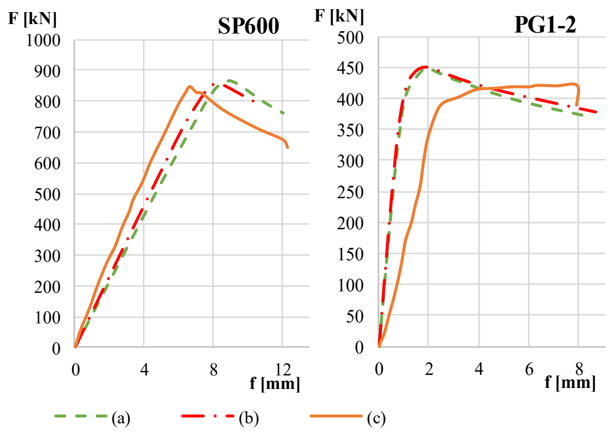


Fig. 7: Comparison of load-displacement curves depending on the selected finite element type (a) Shell (b) Solid (c) Experimental data.

The following parameters are taken for the basic FE model apllied in the following numerical studies:

- quadrilinear relationship according to BSK07,
- mesh size equal to five-times web thickness ($5t_w$),
- eigenmode imperfections and
- shell type of finite elements.

3. Results and Discussion

An important phase of the design based on FE models is the sensitivity analysis of the model to the values of the basic variables (yield strength, elastic modulus, web thickness, imperfections, etc.) and model parameters (mesh size, type of FE element, etc.) on the results (output variables) of the model [27]. It is necessary to establish at what scatter of input data the validity of the main conclusions drawn from the modelling results is preserved.

Sensitivity analysis should be performed taking into account the actual variability of the variables (input parameters). As a rule, it is most correct to consider changes in the variable that accounts for the mean value and the standard deviation. However, for this pilot study, a change of all variables by plus or minus 10% is accepted for simplification.

When performing a sensitivity analysis of the **mechanical properties** of steel, the yield strength, ultimate strength, and elastic modulus values obtained from experimental data were increased and decreased by 10%. For example, the value of the yield strength for the SP600 beam after an increase of 10% is: for the web f_{yw} = 422 MPa, and for the flanges and stiffeners, f_{yf} = 389 MPa. Table 2 presents the results of the beam calculations depending on the value of the yield strength. This parameter significantly affects the value of the ultimate load, but the nature of the deformation remains unchanged.

Tab.2: Yield strength sensitivity analysis.

Beams	f_y	F_{exp}, kN	F_{FEA}, kN	F_{exp} / F_{FEA}
SP600	+10%	846	920	0.92
	exp.		868	0.97
	-10%		810	1.04
SP1200	+10%	1030	1086	0.94
	exp.		1020	1.01
	-10%		954	1.08
PG1-2	+10%	412	489	0.84
	exp.		450	0.92
	-10%		423	0.97
PG4-2	+10%	154	146	1.05
	exp.		138	1.12
	-10%		125	1.23

The value of the ultimate strength for the SP600 beam after an increase of 10% for the web is f_{uw} = 597 MPa, and for the flanges and the stiffeners - f_{uf} = 571 MPa. This parameter does not affect the value of the ultimate load with such a change.

The value of the elastic modulus for the SP600 beam after increasing for the web is E = 194.6 GPa, and for flanges and stiffeners – E = 204.7 GPa. In Fig. 8, a comparison of the load-displacement curves of the SP600 beam is presented depending on the value of the elastic modulus. This parameter, with such a change, does not significantly affect the value of the maximum load, at which the difference between the FE models is 3%. The changes in the angle of inclination of the initial linear section of the curves can be seen.

When performing **sensitivity analysis of the geometry** of the FE models, the thickness of the web and flanges taken from experimental data increased and decreased by 10%. The web thickness for the SP600 beam after an increase of 10% is 6.6 mm. In Fig. 9, you can see a comparison of the load-displacement curves of the SP600 beam depending on the value of the web thickness. Table 3 shows the results of beam calculations after changing the thickness of the web. This parameter significantly affects the maximum load value, but the nature of the deformation remains unchanged.

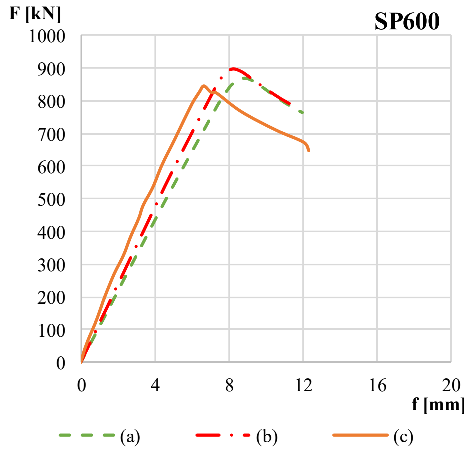


Fig. 8: Comparison of the load-displacement curves depending on the value of the elastic modulus (a) Basic FE model, (b) The value of the elastic modulus after an increase of 10%, (c) Experimental data.

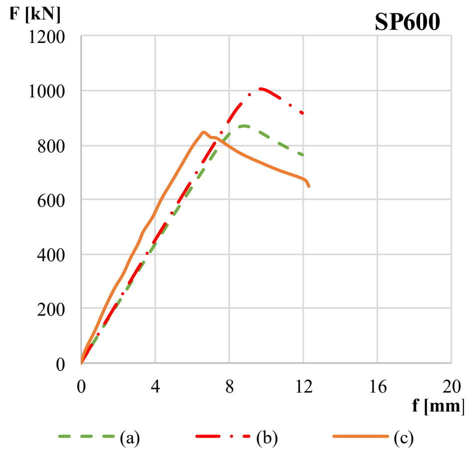


Fig. 9: Comparison of the load-displacement curves depending on the value of the web thickness (a) Basic FE model (t_w = 6 mm), (b) FE model (t_w = 6.6 mm), (c) Experimental data (t_w = 6 mm).

Tab.3: Web thickness sensitivity analysis.

Beam	t_w	F_{exp}, kN	F_{FEA}, kN	F_{exp} / F_{FEA}
SP600	exp.	846	868	0.97
	+10%		1004	0.84
	-10%		730	1.16
SP1200	exp.	1030	1020	1.01
	+10%		1184	0.87
	-10%		866	1.19
PG1-2	exp.	412	450	0.92
	+10%		520	0.79
	-10%		392	1.05
PG4-2	exp.	154	138	1.12
	+10%		155	0.99
	-10%		123	1.25

The thickness of the flanges for the SP600 beam after an increase of 10% is 22 mm. Fig. 10 shows a comparison of the load-displacement curves of the SP600 beam depending on the value of the thickness of the flanges. This parameter, with such a change, does not significantly affect the value of the maximum load, at which the difference between the two models is about 2% and the nature of the deformation remains unchanged.

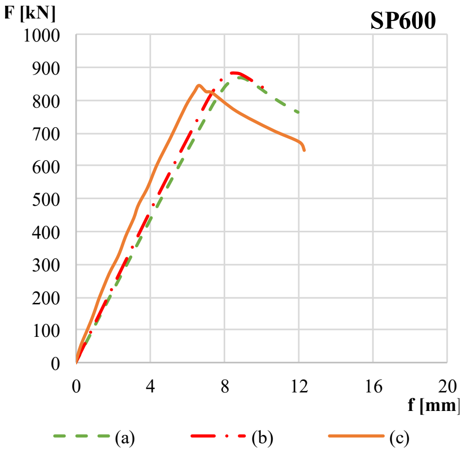


Fig. 10: Comparison of the load-displacement curves depending on the value of the thickness of the flanges (a) Basic FE model (t_f = 20 mm), (b) FE model (t_f = 22 mm), (c) Experimental data (t_f = 20 mm).

Sensitivity analysis of the **initial imperfections** was performed for the first and second eigenforms and the combination of eigenforms. It can be noted that for these beams, the choice between the first, second, and combined eigenforms of loss of stability does not significantly affect the value of the maximum load and the nature of deformation. Table 4 provides the results of sensitivity analysis focused on the effect of the **shape and magnitude of equivalent imperfections** on ultimate model resistance.

Tab.4: Sensitivity of ultimate model resistance to initial imperfections.

Beam	Eigenforms	Value, mm	F_{exp}, kN	F_{FEA}, kN	F_{exp}/F_{FEA}
SP600	1- st	3	846	868	0.97
		6		852	0.99
	2- st	3		886	0.95
	Comb.	3		882	0.96
SP1200	1- st	6	1030	1020	1.01
		12		1014	1.02
	2- st	6		1010	1.02
	Comb.	6		1040	0.99
PG1-2	1- st	3	412	450	0.91
		6		445	0.93
	2- st	3		418	0.98
	Comb.	3		418	0.98
PG4-2	1- st	2.5	154	138	1.11
		5		137	1.12
	2- st	2.5		123	1.25
	Comb.	2.5		121	1.27

As a result, FE models were constructed and data were obtained for comparison with the results of the experiments. Graphs of vertical movement of beams under the combined action of local and shear forces are shown in Fig.11. The results of the experimental and the FE models are summarized in Tab. 5. The models showed close numerical convergence with the experimental results.

Tab.5: Results of experimental data and FE models

Beam	$F_{exp.}$, kN	$F_{FEA.}$, kN	F_{exp} / F_{FEA}
SP600	846	868	0.97
SP1200	1030	1020	1.01
PG1-2	412	450	0.92
PG4-2	154	137	1.12

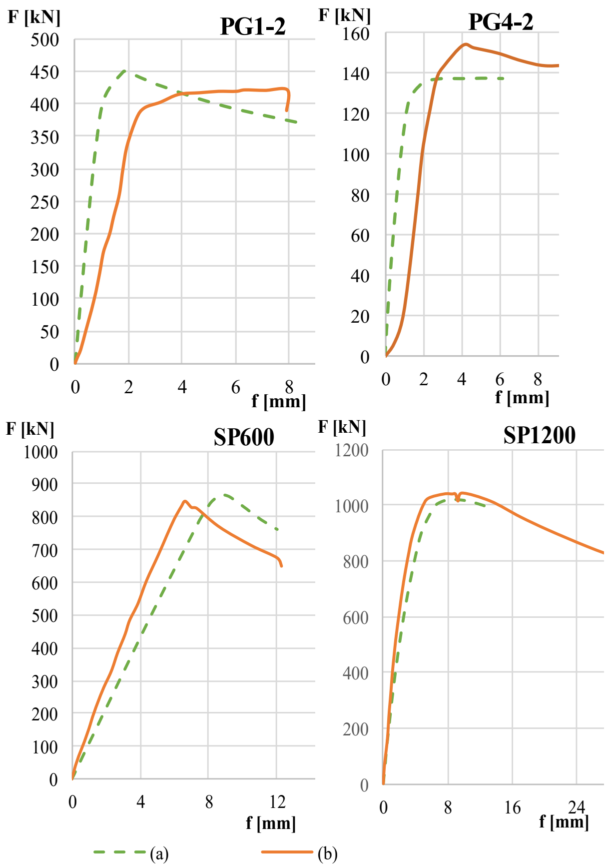


Fig. 11: Graphs of the vertical movement of the beams under the combined action of local and shear forces (a) Basic FE model, (b) Experimental data.

The purpose of this article is to apply a generic FE model, and not to develop the best-validated model. In this light, a difference of 10% may seem reasonable. The main reasons for the differences between the FE models and experimental data may be the shape and magnitude of imperfections. Moreover, also inaccuracies in experimental measurements of yield strength, elastic modulus and web thickness may play a role. Additional assumptions and approximations adopted here and possibly affecting model predictions are listed below:

- Only some mechanical properties of steel were specified In [1],, therefore, the tensile strength is assumed to be f_u = 1.35 f_y according to [17], the modulus of elasticity and the Poisson's ratio is assumed to be E = 210 GPa and ν = 0.3.
- This study is limited regarding a number of samples

to verify the results. The obtained findings thus need to be verified by further comparisons.

- Analysis covers only three material models.
- No sensitivity analysis with respect to the eccentricity of loading application is made. It is assumed that this eccentricity has a significant effect for a patch load; additional investigations based on measurements are needed.

4. Conclusions

This article investigates the importance of selection of parameters for FE models, which should be considered when assessing the resistance of steel elements with a flexible thin web. The main parameters affecting the result of modelling a thin-walled element include the selection of the stress-strain curve, specification of material properties, type and size of the finite elements, and shape and magnitude of initial imperfections. After analysing the models for materials, it is proposed to use a quadrilinear relationship with yielding and strain hardening. It appears that a lower value of the ultimate load is reached when the load-displacement curve without strain hardening is applied. However, the difference between the results of the FE models may be very small (about 1% in the cases under investigation). The analysis of the mesh size shows that the most optimal size is about five web thicknesses of the element. The use of solid elements does not lead to a significant increase in the accuracy of the model; therefore, it is recommended to use shell elements. The imperfections are recommended to be based on the first eigenforms of buckling.

The study indicates that the finite element method is perfectly suitable for solving problems related to the stability of beam web under combined load, makes it possible to take into account a wide range of factors and validate against full-scale experimental data.

The sensitivity analysis reveals that dominating is the variability of the thickness of the web (with a change of ± 10%, the load-bearing capacity changes by ± 16%) and the strength of the yield (with a change of ± 10%, the load-bearing capacity changes by ± 7%). These variables should thus be controlled during manufacturing and taken into account when calibrating partial factors.

It should be noted that, in addition to research in the field of the principles of constructing FE models and their verification with experimental data, it is necessary to develop criteria for limit states and safety format for FE applications [28].

5. Acknowledgements

This study has been supported by the Technology Agency of the Czech Republic under Grant CK03000125 and by the Ministry of Education, Youth and Sports of the Czech Republic under Grant CZ.02.2.69/0.0/0.0/18_053/0016980.

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