ZESZYTY NAUKOWE POLITECHNIKI RZESZOWSKIEJ Nr 283
Budownictwo i Inżynieria Środowiska z. 59 (1/12/I) 2012

Renáta VARGOVÁ[1]
Technical University of Košice

EVALUATION OF THE EXPERIMENTS WITH RESPECT TO SIMULATION TECHNIQUES

The paper presented focuses on the comparison of the measurement results obtained in laboratory experiments on steel members subjected mostly to bending in the elastic-plastic stage. Special attention is paid to an evaluation of the elastic and plastic bending load capacity, with particular emphasis on the statistical processing of resistance and geometrical characteristics of steel members. Some results of the statistical study and experimental research are presented in the form of graphs and tables.

1. Introduction

The main concern in design of any steel construction is ensuring its high reliability and cost-effectiveness. As a general rule, the concept of structural reliability seems contradictory to the economy concept, leading to the opinion that the structure should not be unreasonably reliable at the cost of its economy and vice versa. Structural reliability depends on a number of random variables. Therefore, it is necessary to pay attention to such input variables that most affect structural resistance, i.e. to introduce into the calculations correct mechanical properties and geometrical proportions of a structure, to select the proper calculation models. If a structural system or component is to perform its required function reliably during the whole period of its service life, it is essential that these aspects are taken into consideration during the early design stages. The task can be solved using a statistical approach where a great deal of numerical data and experience must be collected and applied to demonstrate a more or less random character of dimensions, material and geometrical characteristics, structural arrangements, load etc.

For the purpose of verifying the influence of individual random variables acting on steel beams as well as verifying the elastic-plastic behaviour of steel members in bending, an experimental plan was set up in the laboratories. It

[1] renata.vargova@tuke.sk

helped the researchers understand the problem and build up a general picture of the real behaviour of a steel beam during its loading stage. The experiment included two groups of beams – N1 and N2.

2. Experimental programme

The experimental programme included a total of 24 simple steel beams with the identical cross-section of a rolled-steel joist IPE 160. With regard to the purpose of the research, various interior spans were designed for the individual beams – L (600, 800, 1 000 and 1 200 mm). Three identical beams were tested for each of the spans to verify the results obtained. Depending on the loading mode and the type of support the beams were divided into two basic groups. The loading diagrams for the individual beams are shown in Fig. 1 and Fig. 2.

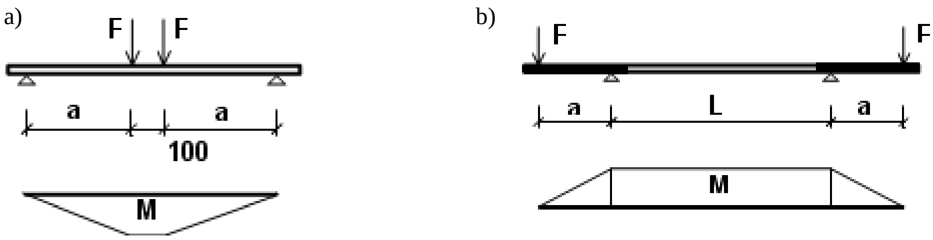

Fig. 1. Loading diagrams for: a) the first, b) the second group of test beams

Fig. 2. Test configuration: a) the first, b) the second group of test

Load on the beams was activated with a hydraulic device, one actuating cylinder, bearing plates 12 mm in width and, in the beams of group N2, also through a rigid secondary beam. The test beams were propped horizontally against lateral-torsional buckling only at their supports by means of special frames.

The testing procedure followed the theoretical calculations of the ultimate loads and the standard verifications of the beams in terms of buckling resistance. During the experiment attention was paid to the evaluation of statistics of material properties and geometrical data and to the examination of overall vertical deflections of the beam (v), lateral deflections (u) and strains (ε).

The acquired results and knowledge then created a necessary database for the evaluation of elasto-plastic resistance of the beams and was used for the calibration of theoretical models [1].

3. Evaluation of geometrical data and material properties using a probabilistic approach

In accordance with the purpose of this article, the acquired set of geometrical data for the examined beams was further used in a probabilistic-statistical analysis. The analysis consisted of the evaluation of all measured geometric dimensions: the depth of a cross section – h, its width – b, flange thickness – t_f and web thickness – t_w, and the specification of variations in the measured geometric dimensions (in comparison with those declared by the manufacturer). These variations were subsequently compared with the tolerances specified in the product standards. Permissible tolerances were considered in compliance with STN EN 10029+AC [2] and STN EN 1090-2 [3]. As indicated in the standards, all structural members – whether hot-rolled or cold-formed products of structural steels – must comply with the specific tolerances. Not only do the tolerances apply to structural elements, or components, but also to structural systems constructed from such sections, unless these standards are replaced by some stricter criteria stipulated in Annex D.1 to the standard STN EN 1090-2.

STN EN 1090-2 defines the following basic tolerances:

- depth

  $\Delta = h / 50 = 160/50 = 3{,}2$ mm,

- width

  $\Delta = B / 80 = 82/80 = 1{,}025$ mm.

In compliance with the standard STN EN 10029+AC:

- web thickness

  $\Delta = -0{,}4a + 1{,}1$ mm (A),

$\Delta = –0{,}3a + 1{,}2$ mm (B),

$\Delta = 0{,}0a + 1{,}5$ mm (C),

- flange thickness

$\Delta = –0{,}4a + 1{,}1$ mm (A),

$\Delta = –0{,}3a + 1{,}2$ mm (B),

$\Delta = 0{,}0a + 1{,}5$ mm (C).

For comparison, the dimensions declared by the producer for the individual cross-sections are shown in Tab. 1 and the real dimensions for all tested beams measured in the experiments in Tab. 2.

Table 1. Dimensions of cross-sections declared by the producer

Beam	**Section**	***h* [mm]**	***b* [mm]**	**t_w [mm]**	**t_f [mm]**
N1-600-1200, N2-600-1200	IPE 160	160	82	5	7,4

Table 2. Dimensions of the cross-sections measured in the experiments and their variations

Beam	***h* [mm]**	***b* [mm]**	**t_w [mm]**	**t_f [mm]**	**Δh [mm]**	**Δb [mm]**	**Δt_w [mm]**	**Δt_f [mm]**
N1-600-1	160,20	82,82	5,27	7,03	0,20	0,82	0,27	–0,37
N1-600-2	160,80	82,65	5,20	6,95	0,80	0,65	0,20	–0,45
N1-600-3	161,70	82,82	5,20	7,02	1,70	0,82	0,20	–0,38
N1-800-1	161,83	82,02	5,17	7,05	1,83	0,02	0,17	–0,35
N1-800-2	161,93	82,13	5,27	7,03	1,93	0,13	0,27	–0,37
N1-800-3	161,50	82,22	5,17	7,15	1,50	0,22	0,17	–0,25
N1-1000-1	161,67	82,68	5,27	6,97	1,67	0,68	0,27	–0,43
N1-1000-2	160,40	82,70	5,33	7,03	0,40	0,70	0,33	–0,37
N1-1000-3	160,87	82,45	5,53	7,03	0,87	0,45	0,53	–0,37
N1-1200-1	160,90	82,47	5,73	6,95	0,90	0,47	0,73	–0,45
N1-1200-2	161,00	82,58	5,60	6,95	1,00	0,58	0,60	–0,45
N1-1200-3	160,77	82,75	5,27	7,08	0,77	0,75	0,27	–0,32
N2-600-1	160,77	82,40	5,17	6,93	0,77	0,40	0,17	–0,47
N2-600-2	160,57	82,22	5,00	6,95	0,57	0,22	0,00	–0,45
N2-600-3	160,63	82,90	5,12	6,97	0,63	0,90	0,12	–0,43
N2-800-1	161,50	82,90	5,07	6,95	1,50	0,90	0,07	–0,45
N2-800-2	160,57	83,13	5,00	6,92	0,57	1,13	0,00	–0,48
N2-800-3	160,23	82,37	5,03	7,03	0,23	0,37	0,03	–0,37
N2-1000-1	160,07	82,38	5,00	6,93	0,07	0,38	0,00	–0,47
N2-1000-2	160,13	82,78	5,13	6,95	0,13	0,78	0,13	–0,45
N2-1000-3	161,07	82,08	5,03	6,98	1,07	0,08	0,03	–0,42
N2-1200-1	160,20	82,70	5,00	6,97	0,20	0,70	0,00	–0,43
N2-1200-2	161,13	82,87	5,03	6,93	1,13	0,87	0,03	–0,47
N2-1200-3	161,10	82,32	5,03	6,97	1,10	0,32	0,03	–0,43

The sign of all variations in the depths of the cross-sections in the experiments is positive. The maximum $\Delta\ h_{max}$ is +1,93 mm, which, when taking into account the absolute value, is within the standard tolerances. Similarly, the variations in the cross-sectional widths satisfied the standard criteria with the maximum permissible tolerance Δb_{max} = +0,9 mm. For the variations in the web thicknesses, a maximum of +0,73 mm was recorded, which, again, is within the standard tolerances for Class A and thus also in compliance with the standard requirements for Classes B and C. The variations observed in the flange thicknesses of the beams met the standard requirements set for Class A (negative variations). However, there was only one beam that met the criteria for Class B (a negative variation) and none of the beams complied with the standard requirements for Class C. All measured values were negative with a minimum of – 0,48 mm.

The dimensions of the cross-sections measured in the experiments allowed to determine the corresponding sectional properties. These were later compared with the properties declared by the producer. On the basis of the evaluated data statistical items for the individual sets of geometrical data were calculated: the cross-sectional area variation – φ_A, the variation in the moment of inertia – φ_I and the variations in the module of section – φ_W, φ_{Wpl}. The statistics of geometrical variations and yield stress f_y in the test beams are shown in Tab. 3.

Table 3. Statistics of geometrical variations and yield stress in the tested beams

	φ_A	φ_I	φ_W	φ_{Wpl}	f_y
Normal	0,98785987	0,983778151	0,97808556	0,98191	308,2916667
Median/Mean	0,98276827	0,979658627	0,97508385	0,978782	310
Standard deviation	0,01680388	0,01331343	0,01086728	0,012994	15,69679881
Population variance	0,00028237	0,000177247	0,0001181	0,000169	246,3894928
Kurtosis	–0,2956059	–1,29220915	–1,3767716	–1,40846	–1,99290361
Skewness	0,72706124	–0,08789393	–0,022755	0,081616	–0,10399985
Max–min difference	0,05965377	0,045158534	0,03623169	0,042218	39
Minimum	0,96586009	0,958967838	0,95856843	0,959941	285
Maximum	1,02551385	1,004126372	0,99480012	1,002159	324
Number of beams	24	24	24	24	24

Histograms for the individual sets of cross-sectional geometrical variations and yield stress were constructed. The relative values of geometrical variations were drawn in the range from 0,9 (0,95) to 1,1 (1,05). The graphs of the associated probability density functions were plotted using these data sets and then used in the particular histograms. Yield stress histograms were drawn using the absolute values ranging from 200 to 400 MPa. All histograms are shown in Fig. 3 and 4. Due to a small number of specimens it was impossible to make reliable interferences from the data concerning material characteristics so the evaluation of the material properties could not be considered representative of the set or population. Therefore, in the resistance evaluation of the individual structural elements and systems it is recommended to utilize yield stress histograms given by L. Rozlívka according to the class and type of steel used.

Histograms created for sectional properties (Fig. 4) confirmed the correctness of the application of Histogram N105, ranging from 0,95 to 1,05, defined in the software package MC SIMUL.

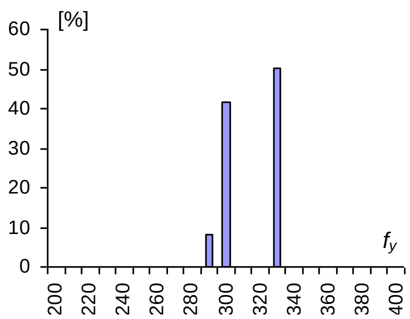

Fig. 3. Yield stress f_y histograms

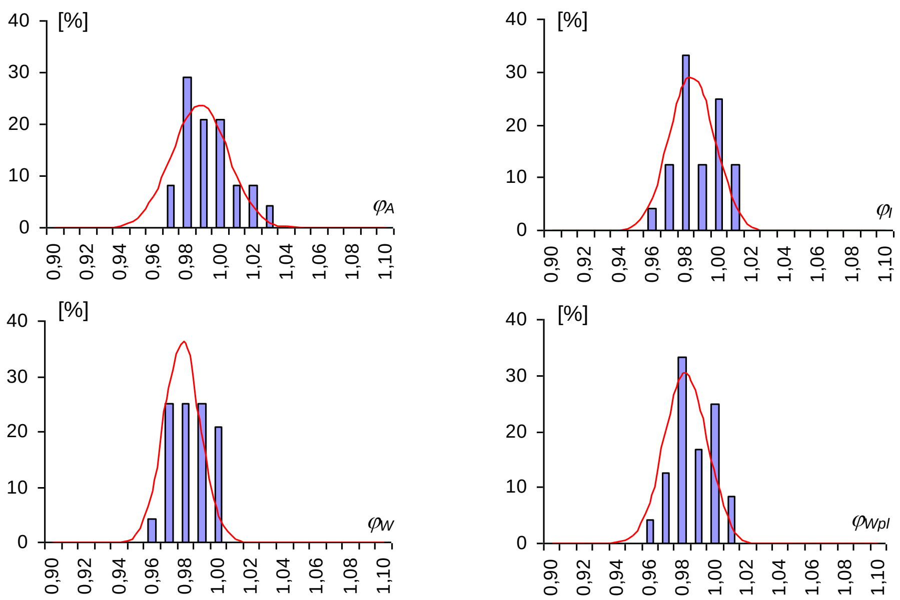

Fig. 4. Histograms for the variations in the geometrical properties of the sections φ_A, φ_I, φ_W, and φ_{Wpl}

In structural reliability assessment using a probabilistic approach it is essential to use real observed statistical sets of sectional properties as they can significantly contribute to the economical design of steel structures. However, the collection of such data is quite a challenging task for an ordinary structural designer. To solve the problem, a correlation analysis for the individual sectional properties was applied. Its purpose was to explore the possibilities of utilizing a single representative random variable for all types of load (action). To verify the assumption, the closeness of linear relation/dependence between the individual geometric data was examined. Correlation coefficients for assumed linear relations between the individual geometrical characteristics for the individual sections were determined and are given in Tab. 4.

Table 4. Correlation coefficients for linear relations

	A	***I***	***W***	***W**$_{pl}$*
A	1			
I	0,802358	1		
W	0,885632	0,976299	1	
***W**$_{pl}$*	0,942394	0,953646	0,988356	1

As Cohen has it [2, 4], the absolute value of correlation lower than 0,1 is trivial, between 0,1 and 0,3 it is small, between 0,3 and 0,5 it is considered medium and the one between 0,5 and 1,0 is great. The resulting correlations confirm that the observed relations are close. Therefore, it is possible to express the given characteristics applied in bending through the characteristics used for tension and/or compression, where:

$$\varphi_A \approx \varphi_I \approx \varphi_{Wel} \approx \varphi_{Wpl}.$$

The correlation between the relative values of the moment of inertia φ_I, the elastic section modulus φ_{Wel} and the plastic section modulus φ_{Wpl} on the relative sectional area of the member φ_A can be seen from Fig. 5. Regression curves were fitted to these data points defined by the equation:

$$\varphi_A = a_j \cdot \varphi_A + b_j,$$

where a_j, b_j are constants calculated by the method of least squares and $j = I$, W_{el}, W_{pl}.

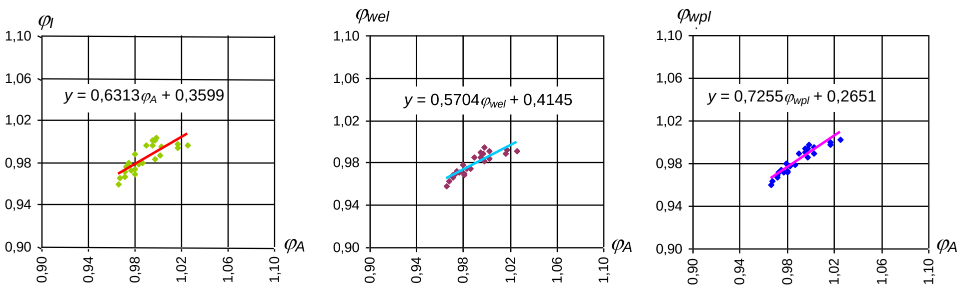

Fig. 5. Regression of sectional properties

Both the correlation and regression analyses confirmed the validity of the assumption to use the uniform histogram N105 generated for the normal distribution in the range between 0,95 and 1,05 for all sectional characteristics.

4. Conclusion

The measurement and the subsequent evaluation of geometrical variations is one of the most important tools in the assessment of accuracy of manufacture and quality of steel structures and elements. Possible variations in geometrical properties have an impact on cross-sectional characteristics and these in the upshot affect the overall structural resistance and deformations. Thus, geometrical variations should be fully in compliance with the set standard tolerances. In the experiment, all geometrical data for the beams were statistically analysed and the results obtained regarding the depth, width, and the web thickness were within the tolerances according to the recommendations given by the standards STN EN 10029 and STN EN 1090-2. The flange thickness was somewhat problematic – it satisfied only the standard criteria for Class A with a negative variation and there was a negative variation for Class B too; however, only one beam was able to fulfil the criteria. Unfortunately, there were no beams that met the required standard criteria for Class C. This implies that the verification of geometrical dimensions can be regarded as perfectly legitimate.

The results acquired, and the evaluation of the results as well, point to the fact that the verification of geometrical properties and all other above mentioned characteristics is quite justifiable and brings the knowledge that may contribute to the more economical design of steel structures.

The article was written within the frames of the project ITMS 26220220124 „Research into Filler – beam Deck Bridges with Encased Beams of Modified Sections”.

References

[1] Vargová R.: Probabilistic analysis of reliability and resistance of frames and members, [Dissertation] TU – Civil Engineering Faculty in Košice, Košice 2010.

[2] STN EN 10029+AC:1998: Hot-rolled steel plates 3 mm thick or above. Tolerances on dimensions and shape, SUTN, Bratislava 1998.

[2] STN EN 1090-2:2009: Execution of steel structures and aluminium structures. Part 2: Technical requirements for steel structures, SUTN, Bratislava 2009.

[4] Cohen J.: Statistical power analysis for the behavioral sciences, 2nd ed., Lawrence Erlbaum, New Jersey 1988.