

## ***Experimental investigation of joint with positive eccentricity in CFS truss***

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### **Keywords:**

stability, truss, eccentricities in the truss joints, experimental tests

1

### **Abstract**

1

2 Due to technological restrictions, in the case of trusses made of CFS open cross-sections,  
3 positive eccentricities in the truss joints have become very frequent. Therefore studies  
4 concerning the load-bearing capacity of truss joints with positive eccentricity were  
5 undertaken. It was assumed that the resistance of cold-formed steel (CFS) open cross-  
6 section truss joints located on compression chords and with positive eccentricity is greater  
7 than that which results from hitherto known methods of steel structure dimensioning. In order  
8 to confirm the hypothesis, experimental studies were conducted. A series of 5 full-scale  
9 research models was subjected to destructive tests to determine the deformation forms and  
10 strains of the hat-section walls in the area of the eccentric joint. Forms of truss chord stability  
11 loss under compression and bending were identified. The course of research and analysis of  
12 results were described in the article. Then comparative analysis of the outcomes of the  
13 experimental tests and the results of analytical calculations carried out according to  
14 Eurocode standards was done, and the obtained results confirmed the usefulness of the  
15 undertaken research. The obtained results did not allow for explicit confirmation of the  
16 research hypothesis at this stage, but constituted the basis for validation of the numerical  
17 model which has been elaborated. Currently, the authors carry out a variety of numerical  
18 analysis.

### **19 1. Introduction**

20 Cold-formed steel (CFS) members are used more and more commonly in industrial  
21 buildings, not only as secondary framing members but also as main bearing elements such

22 as roof trusses and columns. As a result, new research problems appear. Both individual  
23 structural elements such as beams and columns [1–4] and entire parts of the structure,  
24 including lattice girders, are subjected to testing [5–7].

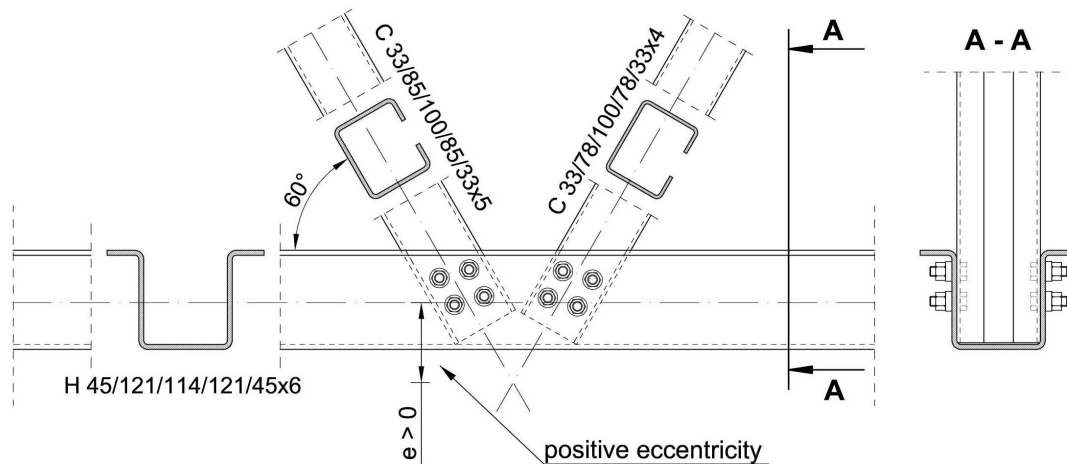
25 Modern lightweight technology used in the production of steel trusses affects their  
26 shape, particularly the construction of joints and, as a result, eccentricities frequently appear  
27 in the trusses' joints. This can induce additional local bending and shearing forces in the  
28 truss chords [8]. In the case of lattice structures composed of circular, square or rectangular  
29 hollow sections, the eccentricity influence on the static design resistances of the welded  
30 joints has been precisely investigated [9,10]. Detailed information about such joints can be  
31 found, among others in publications [9], [11], [12], [13] and in the standard [14]. The  
32 information included in these works shows that the load carrying capacity of joints with  
33 positive eccentricities is lower than that of joints without eccentricities ( $e=0$ ). On the other  
34 hand, in joints with negative eccentricities the opposite is true; their load carrying capacity is  
35 higher.

36 However, in the case of trusses made of CFS open cross-sections, diagonals are  
37 attached to chords by bolts. Trusses' chords are most often designed with hat-sections or  
38 members made of two channels, and diagonals are usually made of single channel sections.  
39 The use of the above mentioned open cross-sections affects the way joints are shaped.  
40 Diagonals are inserted into the cross-sections of the chords, as was done in [6,15], or as  
41 presented in Figure 1. The proper arrangement of mechanical connectors in the truss joints  
42 causes that brace members must be shifted outside, as shown in Figure 1. This results in the  
43 creation of positive eccentricities in the truss joints, and hence bending moments and shear  
44 forces in the chords of lattice girders. Those additional internal forces should be taken into  
45 account when truss members are designed.

46 In engineering practice, the dimensioning of truss chords made of CFS members  
47 taking into account positive eccentricities, in accordance with current European standards  
48 [14,16–18], is performed using a simplified method. However, taking into account the  
49 additional bending and shear between the points of intersection of the diagonals' axes

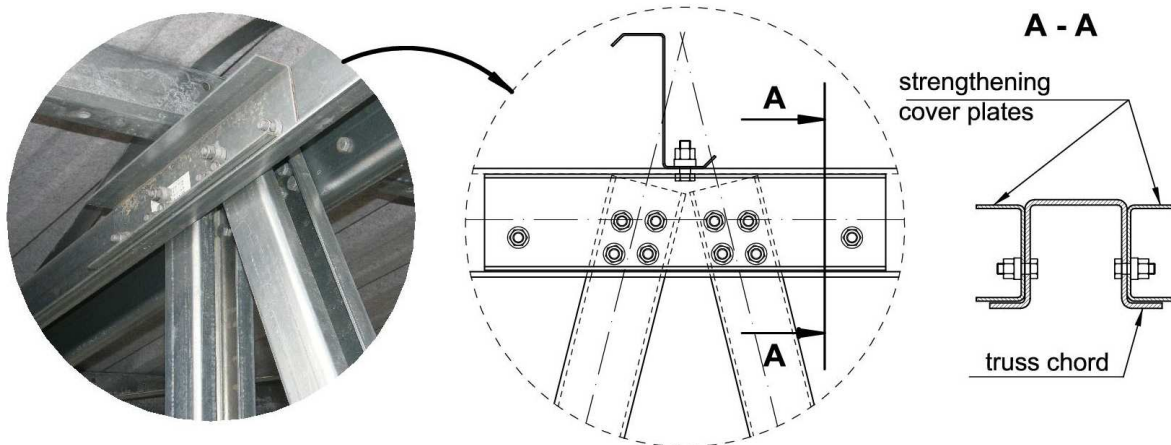


50 causes an increase in the strain of the bottom chord cross-section in the area of the joint. As  
 51 a result, the cross-section of the chord should be increased along its entire length or, as  
 52 presented in the work [19], strengthening cover plates should be made in the nodal zone  
 53 (Fig.2).



54

55 **Fig.1.** Positive eccentricity in the bolted truss joint made from CFS open cross-sections.



56

57 **Fig.2.** Joint with strengthening cover plates [8].

58 **2. State of the art**

59 Most of the studies of CFS members undertaken so far have been carried out on  
 60 single compressed [20–23] or bent structural elements [3,24–28]. The issue of wall stability  
 61 under the influence of shear and bending was investigated by Pham and Hancock [29,30],  
 62 and the results of experimental studies were used to validate the FEM model. Crisan,

63 Ungureanu, and Dubina, in turn, conducted full-scale stability tests of both perforated  
64 sections [31] and high storage racks composed of these [32].

65 Particularly noteworthy are the tests carried out on complex structural elements such  
66 as truss girders. Jankowska-Sandberg and Kołodziej [5] carried out a full-scale examination  
67 of the stability of trusses made out of hollow sections while Iwicki and Krajewski [33]  
68 examined the stability of trusses on models in a 1:4 scale. The top and bottom chords of the  
69 tested truss consisted of two angles and, additionally, battens were applied in the top chord,  
70 while the diagonals were made from closed cross-sections. The above-mentioned studies  
71 concerned the stability of the top and bottom truss chords at different transverse support  
72 conditions of both chords.

73 In fact, few experiments concern studies of trusses made of CFS open cross-  
74 sections. Dawe, Liu, and Li [7] conducted experimental research testing thirteen full-scale  
75 triangular truss specimens made of cold-formed C-section with edge stiffeners. Connections  
76 in the joints were made using self-drilling screws. Additionally, gusset plates were used in the  
77 supporting and ridge joints. Specimens were fabricated in three pitches: 4:12, 6:12, and 8:12.  
78 The aim of the experiments was to investigate the impact of the top chord inclination and the  
79 effectiveness of various methods of strengthening the heel connection on the increase in the  
80 load capacity of the truss. Analysis of the top chord inclination showed that an increase in  
81 truss pitch resulted in an increase in the load capacity of truss specimens. For comparative  
82 purposes, DSM (Direct Strength Method) analysis was also performed and calculations were  
83 carried out using the recommendations of Canadian standards CSA.

84 Dizdar, Baran and Topkaya [34] conducted experimental and numerical tests on 17  
85 full-scale floor trusses made of cold formed sigma-type cross-sections with edge stiffeners.  
86 Diagonals were attached to chords by making use of rivets or self-tapping screws. Both the  
87 rigidity and bearing load capacity of the trusses were tested depending on such parameters  
88 as the wall thickness of the section, the number of connection fasteners used in the joint, and  
89 the method of connecting the diagonals with the chords.

90 Reda et al. [35] performed experimental tests on 12 triangular roof trusses made of  
91 CFS C-sections. Experimental studies allowed for a numerical model to show the actual  
92 behavior of the structure. The aim of the study was to investigate the overall behavior of CFS  
93 trusses using finite element analysis to predict the location of the truss failure and its cause  
94 by calculating the demand to capacity (D/C) ratio using the Direct Strength Method (DSM) for  
95 each component of the truss and comparing this with the results of FE analysis. The  
96 presented procedure for calculating the D/C ratio can contribute to simplifying the  
97 engineering design process by omitting the time-consuming FEM analysis.

98 In turn, Song et al. [36] performed tests on 5 trusses made of CFS sections joined by  
99 means of a self-piercing riveted (SPR) connection. The trusses were subjected to point  
100 bending. The influence of the trusses' height and span on their flexural stiffness and ultimate  
101 bearing capacity was investigated. As a result of parametric analyses, a theoretical formula  
102 was proposed to calculate the equivalent flexural stiffness of the trusses with SPR  
103 connections. As the formula agreed with the experiment's results, it was concluded that the  
104 proposed theoretical formula is valid and can be applied to practical engineering. Not only  
105 was the flexural stiffness and load capacity the subject of the carried out research but also  
106 lateral buckling of chords. Konkong et al. [37] studied the lateral buckling of a CFS cantilever  
107 truss. The object of their interest was expressions for the buckling length factor which are  
108 presented in AISI [38] and the Eurocode 3 [17] specification. Good conformity between  
109 experimental and analytical results was stated. It had been shown that the values of the K-  
110 factor by AISI and Eurocode 3 were too conservative for compression truss.

111 Zaharia and Dubina presented comprehensive studies on the rigidity of bolted joints  
112 in truss joints made of CFS members in [39]. Experimental studies carried out over several  
113 years were aimed at assessing the actual behavior of screw joints in trusses made of CFS  
114 sections. Based on tests performed on single joints and on truss fragments, a theoretical  
115 model of joint stiffness was proposed. The joint rigidity formula, which determines the  
116 buckling length of the lattice members, was also checked by testing on a full-scale structure.

117 Much more experimental research on the joints and connections of the CFS truss  
118 members was undertaken. Lim and Nethercot [40] investigated the stiffness of the shear  
119 connection in the ridge joint of a portal frame made of CFS channel sections. The results of  
120 experimental studies were compared with the results of the numerical analysis. The  
121 dependence of the connection stiffness on the number of bolts and the bolt spacing in the  
122 connection was proved. These studies were continued by Lim et al. [41]. Dubina [42]  
123 described the full-scale tests of lap joints in portal frame joints made of CFS channel  
124 sections.

125 Mathieson et al. [43,44] presented an innovative pin-joint connector called Howick  
126 Rivet Connector (HRC). The research program began with experimental tests on T-joints.  
127 Then, HRC connectors were proposed for the pinned joints in the trusses made of CFS  
128 sections. Tests were carried out on a number of trusses made of channel-section under static  
129 and cyclic loading. Test results indicated that due to the shape of connectors considered,  
130 they can have a positive effect on the joint stiffness.

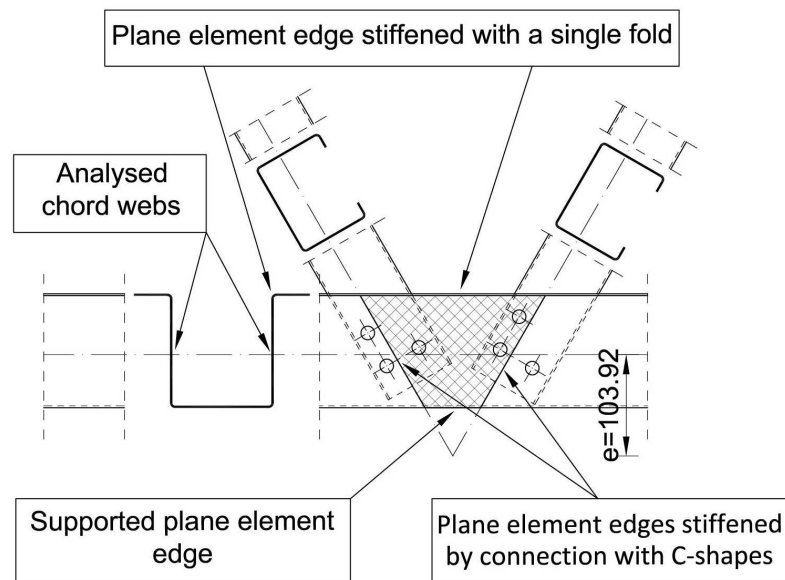
131 Thus, eccentricities in the joints of modern truss structures made from CFS open  
132 cross-sections are inevitable. However, the eccentricity influence on joint resistance has not  
133 been studied so far. The knowledge on sectional instability modes local or distortional  
134 buckling of individual plate elements at complex connections is still challenging and requires  
135 more effort and attention from designers as they are not entirely covered by design code  
136 procedures. Zaharina and Dubina [39] have already drawn attention to the need for such  
137 research due to the lack of standard regulations in this area.

### 138 **3. Experimental investigation**

139 The currently used methods of shaping joints by introducing diagonals inside truss  
140 chords (Fig. 1) cause that the joint area on the truss chord is stiffened by the walls of the  
141 truss members. The support conditions in the part of the hat-section web in the area of the  
142 analyzed truss chord joint are indicated in Figure 3.

143 Therefore, the following research hypothesis was assumed:

144 The resistance of truss joints made from CFS open cross-sections that are located on  
 145 compression chords and have positive eccentricity is greater than the resistance that results  
 146 from hitherto known methods of steel structure dimensioning. The increase in resistance in  
 147 the joint originates from the local support conditions of the plane elements of the hat-section  
 148 chord in the area of the truss joint.



149  
 150 **Fig.3.** Support conditions of the hat-shaped section plane element in the area of the truss  
 151 joint.

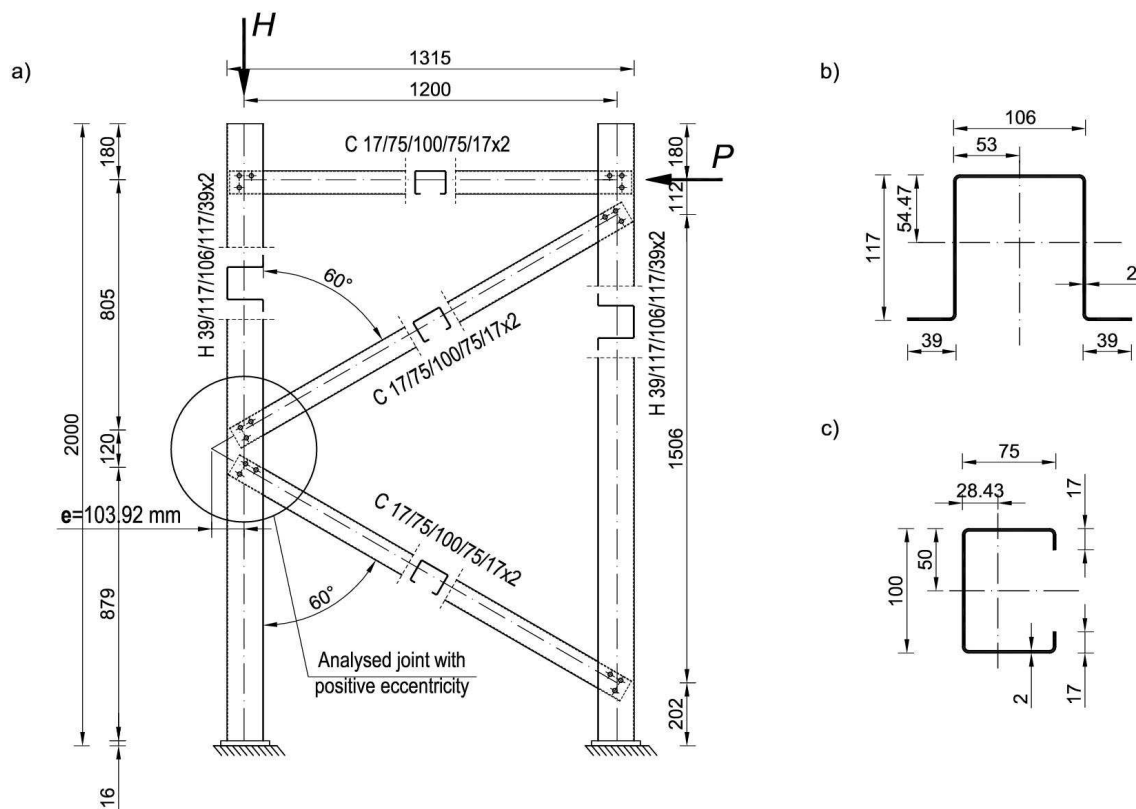
152 In order to prove the above hypothesis, experimental studies of a typical joint used by one of  
 153 the well-known companies involved in the design and manufacturing of steel structures from  
 154 CFS sections were undertaken at Gdańsk University of Technology. Due to the high cost of  
 155 such studies, the experiment was carried out only for one wall thickness of sections, and the  
 156 obtained results allowed the development of a correct numerical model.

### 157 3.1. Experiment arrangement

#### 158 3.1.1. Research model

159 Experimental studies were carried out in a laboratory of Gdańsk University of  
 160 Technology, Poland. A full-scale research model of the truss fragment made of CFS open  
 161 cross-sections was designed. The geometry of the truss adopted for testing is shown in

162 Figure 4a. Chords were made from the hat-section H 39/117/106/117/39x2, while diagonals  
 163 were made from the channel-section C 17/75/100/75/17x2 (Fig. 4b, c). Due to the shape of  
 164 the cross-sections used for the lattice member profiles (open cross-sections), the diagonals  
 165 were inserted into introduced inside the chords, as was done in [15], and [6]. With such a  
 166 method of joint shaping, screw connectors are most often used. In the investigated truss  
 167 segment, 6 bolts M12, class 8.8 were used in each joint (3 bolts on each web), as shown in  
 168 Figure 4a).  
 169 The joint with the value of the eccentricity  $e = 103.92$  mm located on the compression chord  
 170 was tested, as only walls of compressed members are subjected to the local loss of stability.



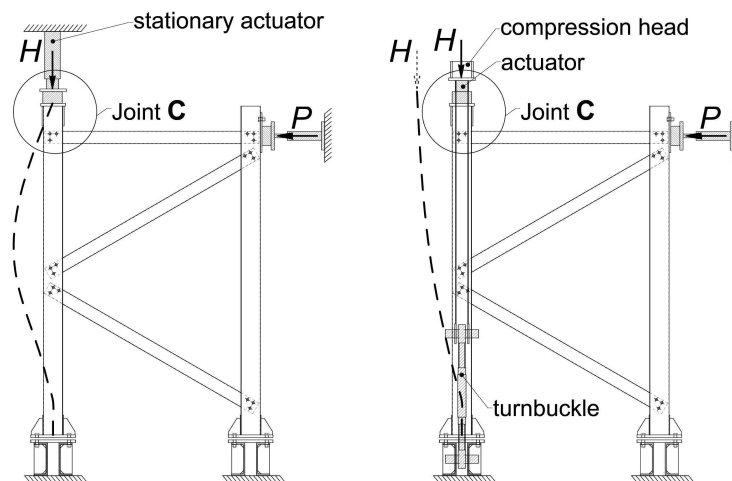
171  
 172 **Fig. 4.** a) research model; static scheme, dimensions of sections used in experimental tests  
 173 for: b) chords, c) diagonals. (All dimensions in mm)

### 174 3.1.2. Research stage

175 The static scheme of the research model is shown in Figure 4. The model was loaded  
 176 with two point forces  $H$  and  $P$ . Force  $H$  causes compression of the bottom chord, while force

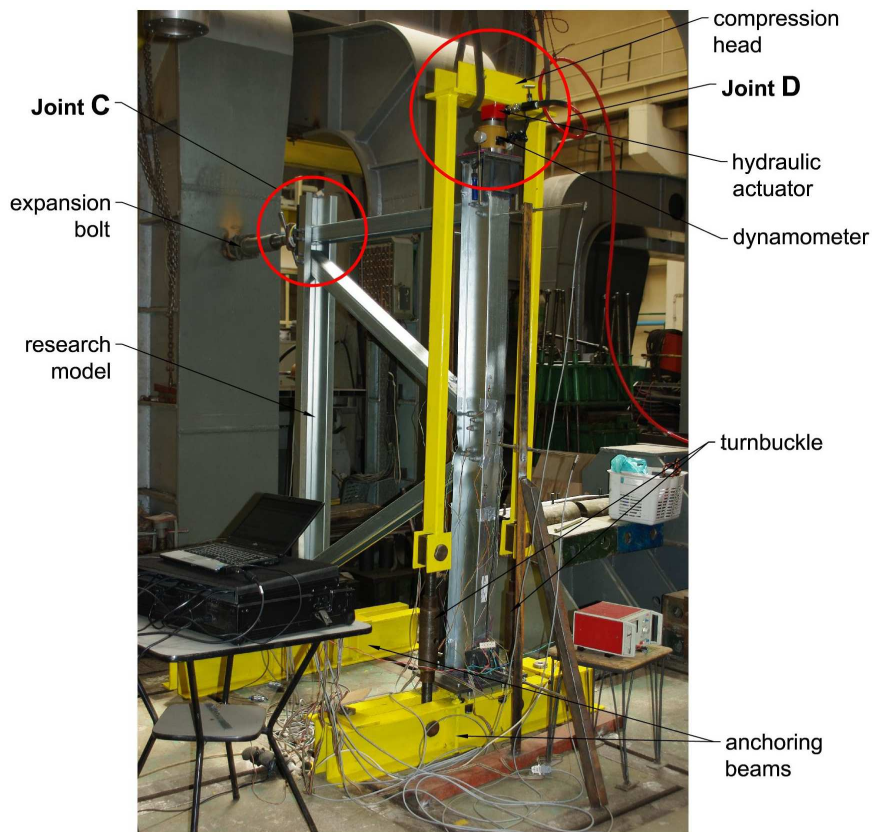


177 P causes, due to the natural distribution of internal forces in the eccentric joint, local  
 178 shearing/bending of the bottom chord in the area of the joint. The use of a stationary actuator  
 179 to introduce force H (Fig. 5a) would change the static scheme of the cantilever mounted  
 180 tested model by immobilizing the free end of the truss in joint C. Therefore, a special  
 181 compression head was applied to introduce force H via two turnbuckles. This allowed for the  
 182 free displacement of joint C under the action of force P (Fig. 5b).



183  
 184 **Fig. 5.** Research model scheme of work when using: a) stationary actuator, b) compression  
 185 head [8].

186 One of the tested models fixed on the research station is shown in Figure 6. The stand was  
 187 developed after a series of preliminary numerical analyses and after the preliminary tests  
 188 described in [8,19]. The support and auxiliary elements of the experimental station were  
 189 painted yellow.



190

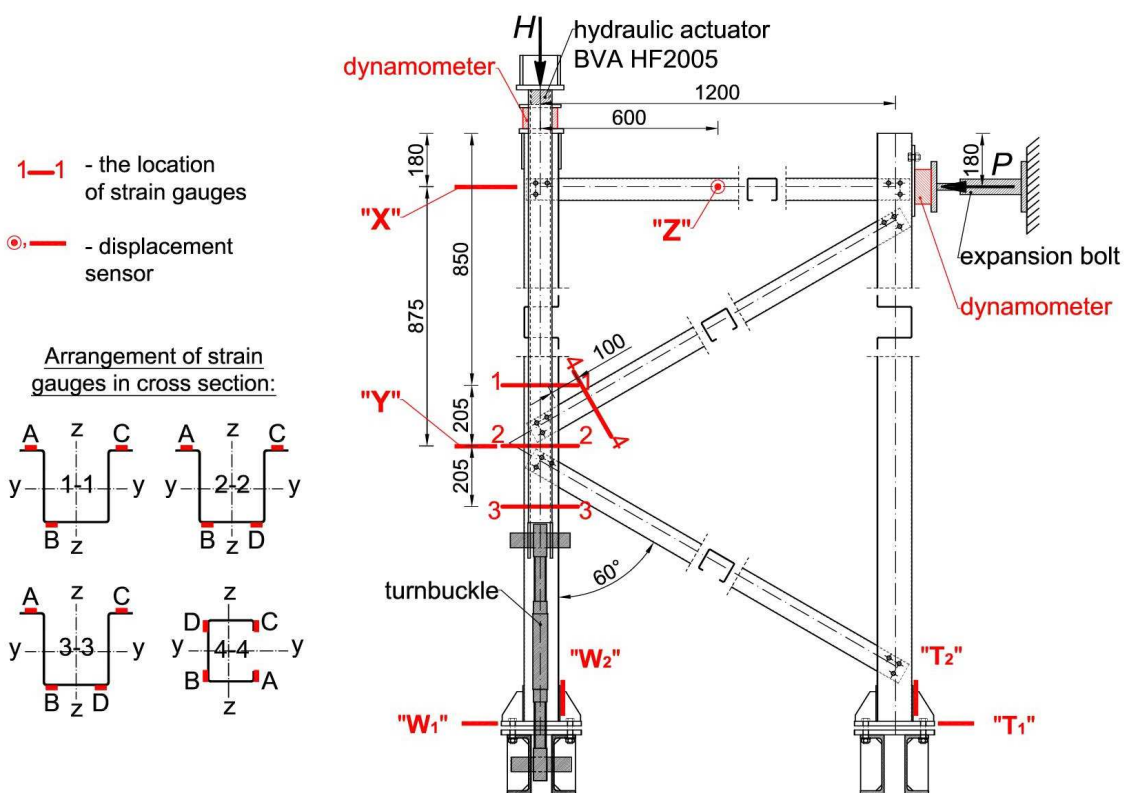
191 **Fig. 6.** Truss segment on the research station.

192 **3.1.3. Applied instruments**

193 The following measuring equipment was used during the tests. Seven inductive  
 194 displacement sensors with a measuring range of  $\pm 100$  mm were applied and arranged on the  
 195 structure according to the diagram in Figure 7. The inductive sensors X and Y measured  
 196 displacements of the research model in the bending plane, and sensor Z measured  
 197 displacements out of the bending plane. Displacement sensors W1, W2, T1, and T2 were  
 198 used to control possible horizontal and vertical displacements of support joints. For strain  
 199 measurements, linear electrical resistance strain gauges LY11-6 / 120A were applied in four  
 200 measuring cross-sections. The three cross-sections were placed on the compression chord  
 201 (hat-section), respectively: 1-1 above the examined joint, 2-2 on the axis of the joint and 3-3  
 202 below the joint, and one additional cross-section 4-4 on the compression diagonal. The  
 203 layout of the strain gauges in each of the cross-sections is shown in Figure 7. In total, 16  
 204 strain gauges were used, including one compensating strain gauge. Four strain gauges were

205 applied at every cross-section. Symmetrical about both local axes (z-z and y-y) strain gauges  
 206 layout assured monitoring of the strains and type of stresses. The same values of the  
 207 readings of the strain gauges A, B, C, and D indicated axial compression. In the case of  
 208 bending about axis y-y, the readings were expected to be the same but only on two strain  
 209 gauges at each side of the bending axis (A=C and B=D). When bending about axis z-z the  
 210 equal readings were on the opposite sides of the bending axis (A=B and C=D).

211 During the tests, two point forces P and H were applied to the structure respectively  
 212 by means of the expansion bolt and the actuator BVA HF2005 with a lifting capacity of 20  
 213 tons. The value of force P was registered using a dynamometer, N.B.C. Elettronica, with a  
 214 nominal range of  $\pm 100$  kN located between the expansion bolt that was causing force P and  
 215 the tensioned chord of the structure, whereas the value of force H was measured by a CZAH  
 216 CT 25 dynamometer with a rated load of 250 kN, located below the compression head and at  
 217 the top of the compression chord of the research model.



218  
 219 **Fig. 7.** Schematic layout of the measuring equipment on the research model.

## 220 **3.2. The course and results of experimental studies**

### 221 **3.2.1. The course of experimental research**

222 Five research models with subsequent numbers 1, 2, 3, 4 and 5 were subjected to  
223 testing. Procedures for preparing and conducting the described tests were carried out in  
224 accordance with the standard EN 1993-1-3 [17]. The destructive tests of the models were  
225 preceded by uniaxial tensile tests, which were the basis for identification of material  
226 constants and determination of the relation  $\sigma$ - $\epsilon$  of the steel material from which the members  
227 of research models were made. The tests were executed on samples cut from sections used  
228 for truss models build. 18 samples were gained from the hat section and 12 samples from  
229 the channel-section. Based on the obtained test results, the following average values of  
230 material properties were determined:

- 231 • hat-section  $E=210$  GPa,  $f_y=398$  MPa,  $f_u=489$  MPa,
- 232 • channel-section  $E=199$  GPa,  $f_y=366$  MPa,  $f_u=445$  MPa.

233 Load application to research models with turnbuckles was preceded by control and  
234 equalization of the tension force values in the tensioning elements of the compression head.

235 Application of load to the structure was carried out at two stages:

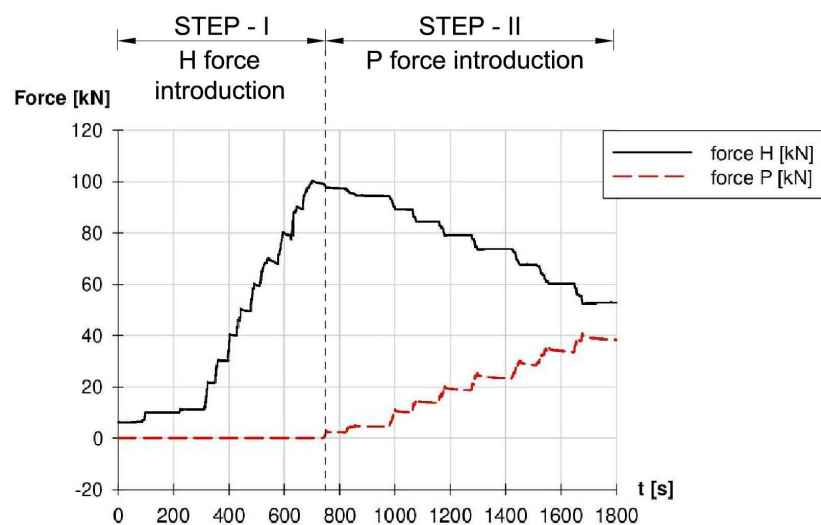
- 236 • Stage I - applying force H,
- 237 • Stage II - applying force P.

238 The forces were put on the structure by gradually increasing their value. Force H  
239 (compression) was introduced in 10 kN increments until the value of 100 kN was reached.

240 From that moment, the value of force H was not changed as its task was only to achieve a  
241 certain level of normal stress in the analyzed cross-section. Next, the second stage of  
242 loading proceeded, i.e. the introduction of force P, with a step of 5 kN. There was a local loss  
243 of stability of member walls in the area of the analyzed joint (Fig. 11a) in all five models,  
244 already within the range of force P values from 5 kN to 10 kN. However, the bending load P  
245 was still increased up to a value of about 40 kN in order to determine the mechanisms of

246 destruction of the bending joint area (Fig. 11b). The course of introduced loads at time “t”  
247 recorded during destructive tests is shown in Figure 8 as illustrated in the results obtained  
248 during the testing of model No. 4. It should be noted that the time course of the load was  
249 analogous in all five models.

250 Based on the analysis of the course of the introduced load, it can be seen that an  
251 increase in the value of force P causes a simultaneous decrease in the value of force H (Fig.  
252 8). It should be pointed out that after starting the introduction of load P, it was not possible to  
253 maintain the value of force H at a constant level due to the instrumentation used. At the  
254 same time, it should be noted that this fact was not significant due to the purpose of the  
255 study, which was to lead to a possible destruction of the compression chord under the  
256 influence of bending moment load in the area of the analyzed joint. However, the values of  
257 both forces were accurately recorded (Fig. 8).

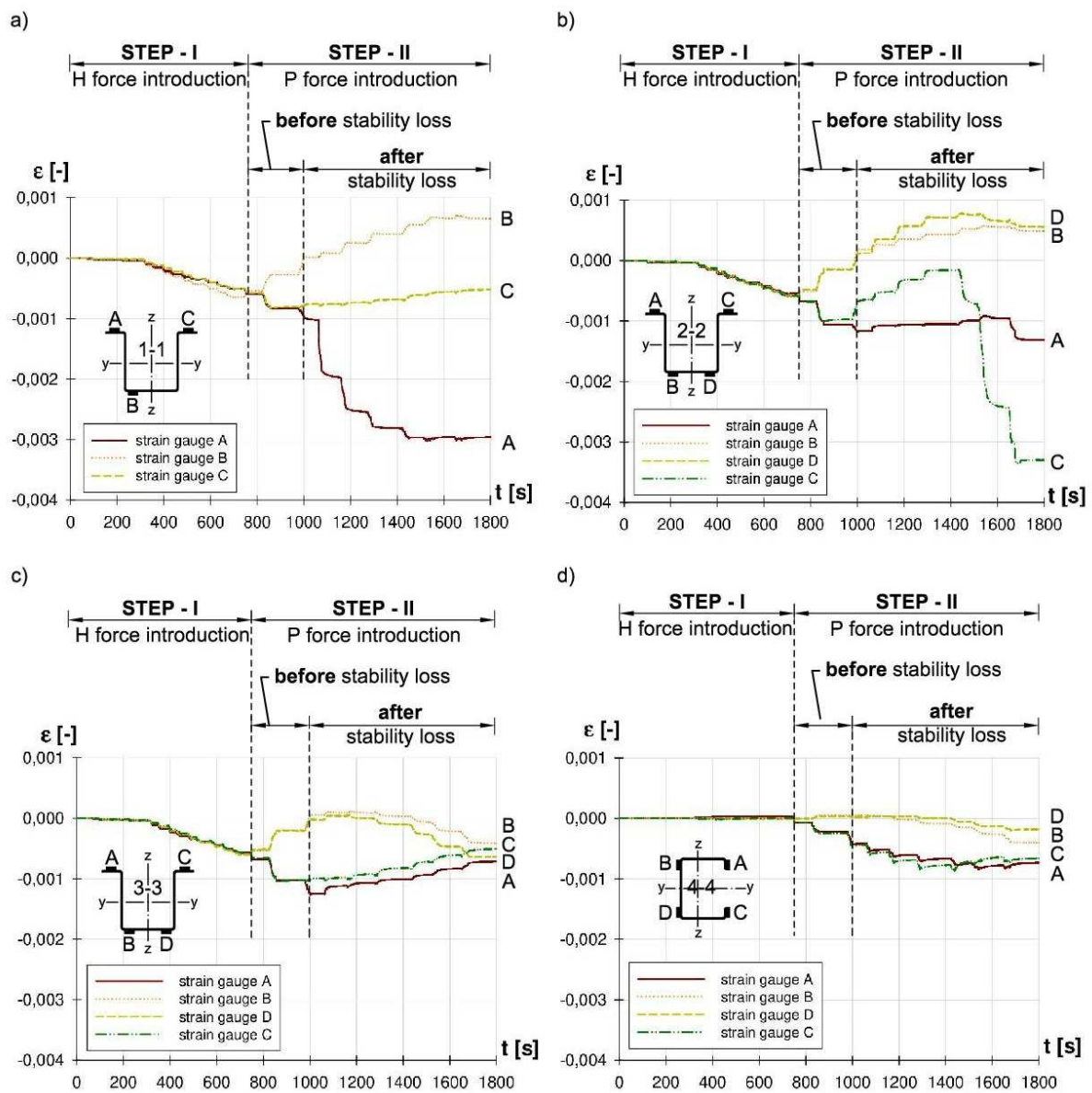


258  
259 **Fig. 8.** Load history in time “t” registered during tests, as illustrated in based on the example  
260 of model No. 4.

### 261 3.2.2. Results of research

262 Due to the purpose of the study, which was to determine the forms of deformation of  
263 the hat-section walls and strains near the analyzed joint while applying the load, the strain  
264 values were registered with strain gauges in four measuring cross-sections (Fig. 7). The

265 courses of strains as a function of time, specified in all four measurement cross-sections and  
 266 registered during tests on model No. 4 are shown in Fig.9.

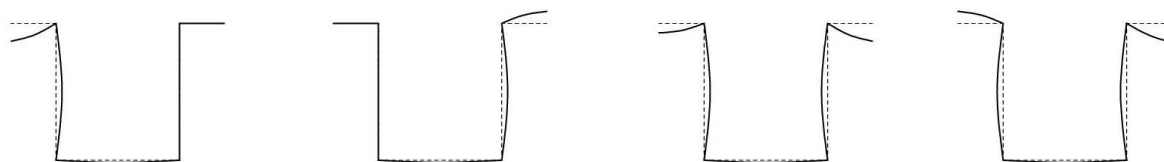


267  
 268 **Fig. 9.** The course of strains as a function of time in cross-sections: a) 1-1, b) 2-2, c) 3-3, d)  
 269 4-4, as illustrated in model No. 4 (description in the text).

270 Based on the charts presented in Figure 9, it was observed that at the first stage of the  
 271 loading, i.e. while introducing force H, there is compliance between the readings of the strain  
 272 gauges A, B, C and D in all four measuring cross-sections, which indicates an even  
 273 distribution of stress in the compressed cross-section. In addition, it should be pointed out  
 274 that at the first stage of loading no forms of deformations of the research model members

275 were observed. During the second stage of loading, in the range before local loss of stability,  
276 symmetrical increases (A, C strain gauges) or reductions (B, D strain gauges) of cross-  
277 sectional strains were observed relative to the local y-y axis ("clean" bending). Nevertheless,  
278 in the range after the loss of stability, it was observed that the deformation shape loses its  
279 symmetrical form. The cantilever walls of the hat-sections were characterized by different  
280 directions of strains (measured with A and C strain gauges). The lack of symmetry relative to  
281 the local z-z axis during deformation is caused by the existence of various imperfections.

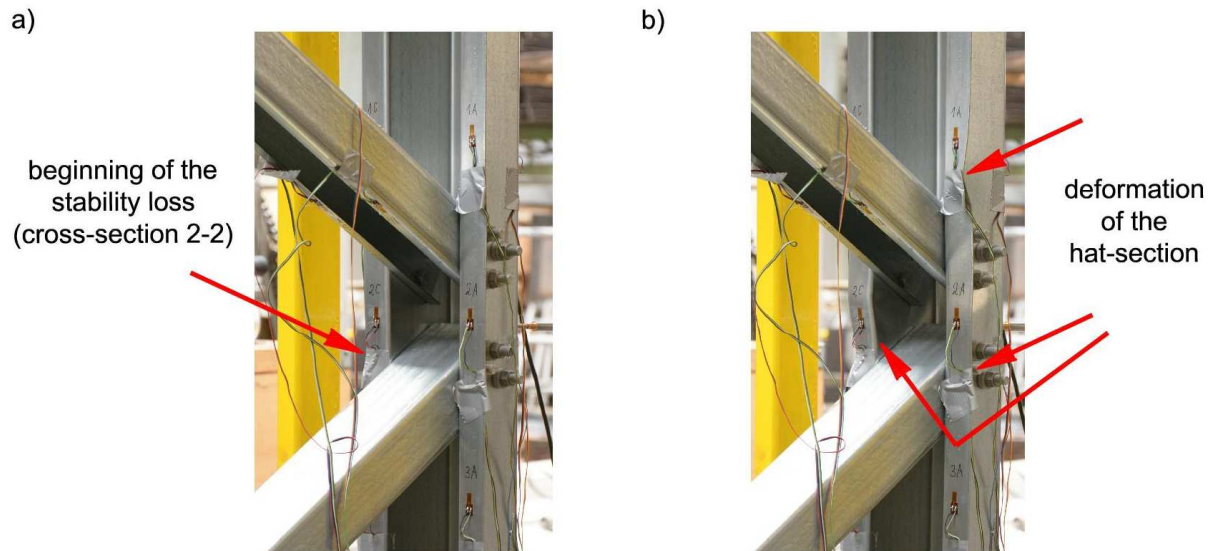
282 Forms of truss chord stability loss under compression and bending identified on the  
283 basis of the destructive tests of five research models are presented in Figure 10. The loss of  
284 stability always started in the measuring cross-section 2-2 and, as the load increased,  
285 deformations increased gradually. In all examined cases only a local form of loss of stability  
286 occurred, what is visible in Figures 10, 11a and 11b. Typical plastic mechanism of failure  
287 when columns build from open cross-sections are subjected to eccentric compression [45,46]  
288 had been observed. It was CF1 when the flange is an outstanding compression element and  
289 CW1 when the web of the cross-section is an internal compression element (flip disc type).



290

291 **Fig. 10.** Forms of truss chord local stability loss.

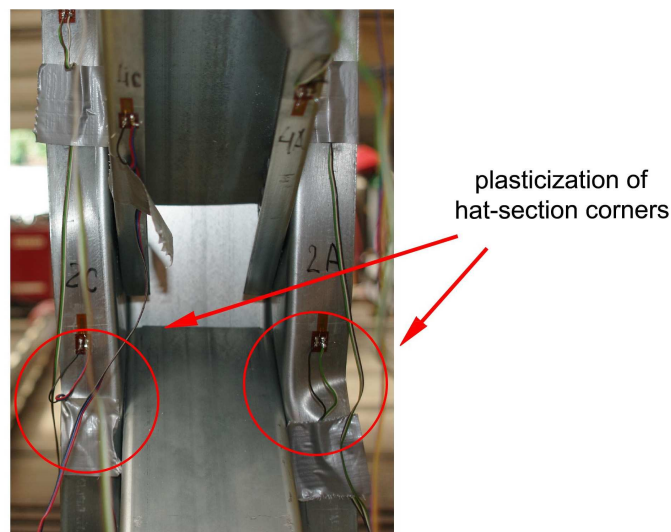
292 Deformation forms of the hat-section walls in the area of the analyzed joint at the  
293 beginning of the stability loss are presented in Figure 11a, while Figure 11b presents the  
294 deformation form of the hat-section observed during the experiment for the final stage of  
295 loading ( $P=40$  kN,  $H_{cor}=55$  kN).



296

297 **Fig. 11.** a) beginning of the hat-section stability loss for the load value  $P=5$  kN,  $H_{cor}=95$  kN,  
 298 b) deformation of the hat-section chord at the final stage of the load.

299 The values of the external load of the research model as a set of forces,  $P$  and the  
 300 corresponding force  $H$ , are summarized in Table 1. These are the values at which the  
 301 beginning of the loss of local stability of the member walls in the vicinity of the analyzed joint  
 302 was observed. It was also observed that in all the research models in cross-section 2-2 or  
 303 directly in its vicinity, permanent deformations of the hat-section corners occurred. This  
 304 indicates that the material yield point was reached in this section area (Fig. 12). The yielding  
 305 state of the corners in the area of the analyzed joint was considered the limit state of the load  
 306 capacity of the analyzed cross-section.



307



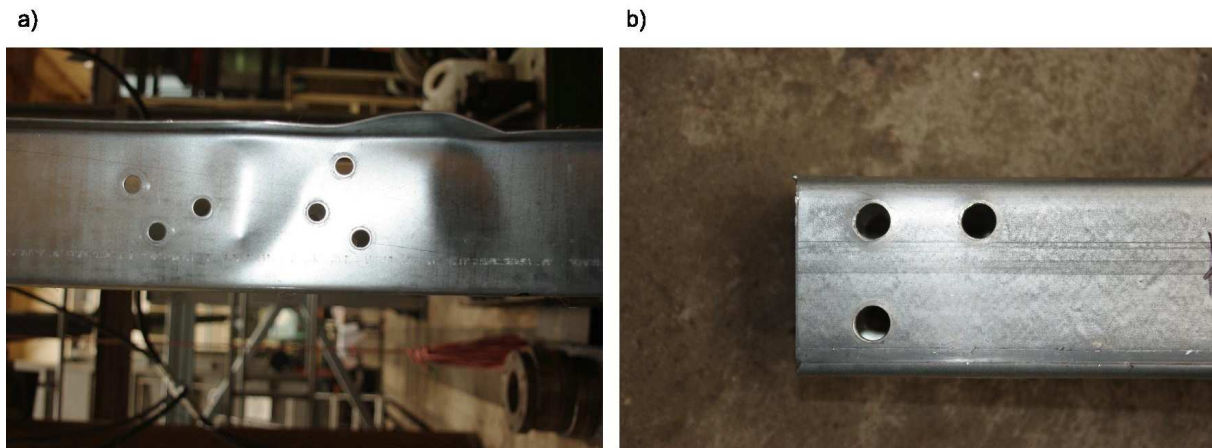
308 **Fig. 12.** Yielding of the hat-section corners in the measuring cross-section 2-2 (based on the  
 309 example of research model no. 2).

310 Considering the above, the beginning of the stability loss observed during the tests can be  
 311 qualified as the attainment of the critical state of the cross-section, and the load values given  
 312 in Table 1 as the critical load of the research model (in the form of a set of P and H forces).  
 313 When the above-mentioned load was reached, it was considered that the structure was  
 314 working in a supercritical state until it reached the limit state. The observed variation of the  
 315 forces P and  $H_{cor}$  is resulting from the "eye method" applied for identification of the stability  
 316 loss moment during the conducted experiments. Received values of the forces were  
 317 verified with the use of elaborated diagrams presented in Figure 9. The variation of force  
 318 P ranging from 5 to 8,5 kN is acceptable when statistical analysis is done. The test results  
 319 presented in Table 1 showed good convergence of results for all models. All results are  
 320 within +/- 2 standard deviations, which indicates good repeatability of the experiment.

321 **Table 1.** Listing of corresponding values of P and  $H_{cor}$  forces at which the local stability loss  
 322 of the compressed chord of the truss model occurred.

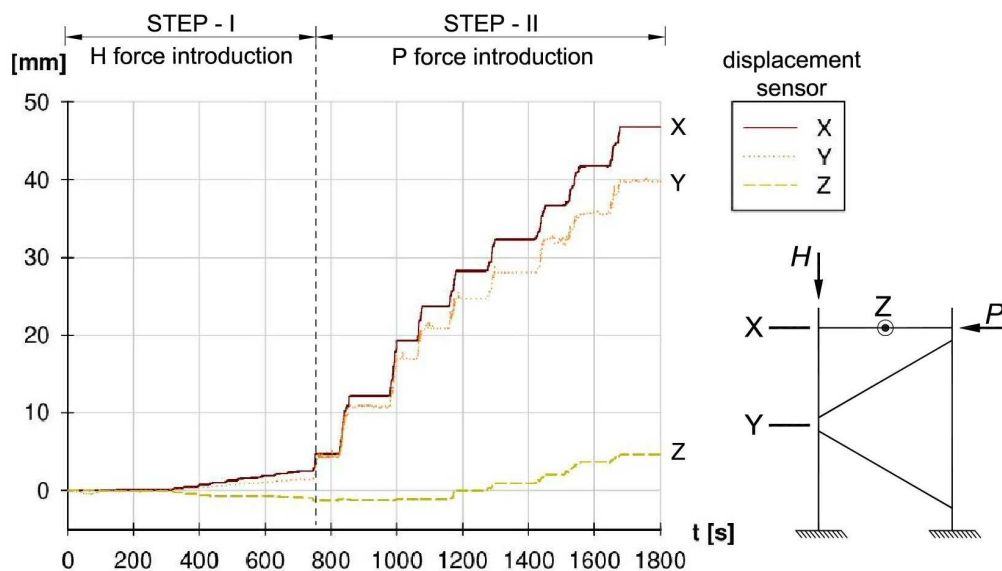
Research model number	Force P [kN]	Average value of force P $P_{av}$ [kN]	Standard deviation $s_P$ [-]	Corresponding force H $H_{cor}$ [kN]	Average value of force H $H_{cor,av}$ [kN]	Standard deviation $s_H$ [-]
Model 1	5.0	6.58	1.58	106	94.6	7.43
Model 2	8.5			96		
Model 3	7.7			89		
Model 4	5.0			95		
Model 5	6.7			87		

323 After carrying out destructive tests, the models were unscrewed to check for damage to  
 324 fasteners and holes. It is visible in Figure 13 that there was no damage to the holes in the  
 325 walls of the hat or channel-shaped sections. Bolts M12 class 8.8 used in the connection were  
 326 also not damaged.



327  
 328 **Fig. 13.** View of bolt holes in deformed truss members during the experiment: a) hat-section,  
 329 b) channel-section.

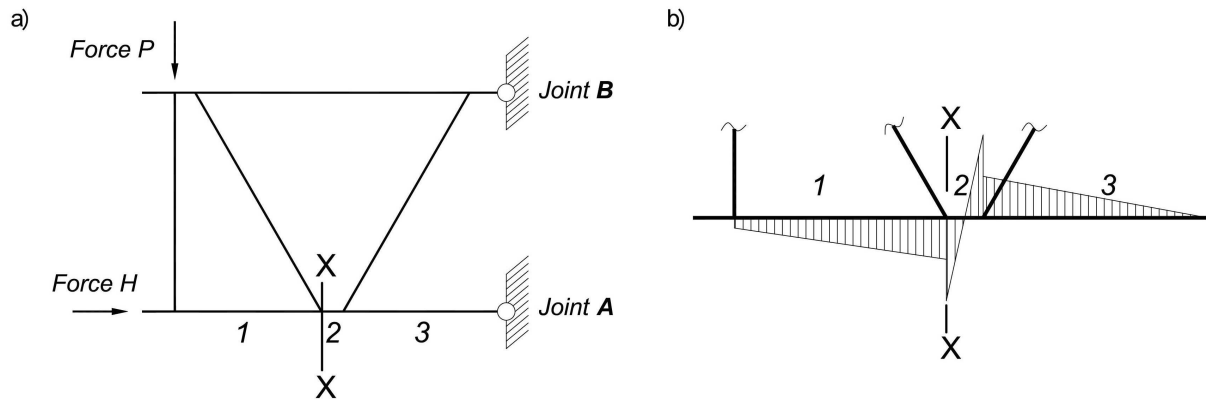
330 Model displacements at X, Y and Z measurement points are shown in Figure 14. The  
 331 lateral displacement values (in the plane perpendicular to the plane of the truss) resulting  
 332 from the "Z" sensor and which ranged from -2.28 mm to 4.65 mm, were considered  
 333 insignificant. In turn, the control of model support displacements showed minimal rotation  
 334 and support displacement.



335  
 336 **Fig. 14.** Model displacements at X, Y, Z measurement points – illustrated in model No. 4.

337 **4. Eurocode 3 design procedure.**

338 For comparative purposes, the stress intensity of cross-sections 1-1, 2-2 and 3-3 in the  
 339 research truss joint was checked using analytical design procedures covered by European  
 340 standards [14,16–18].



341  
 342 **Fig. 15.** a) Static scheme of the analyzed truss with the numbering of the compression chord  
 343 members, b) bending moment diagram at the compression truss chord.

344 In the analyzed case (Fig. 15), in the cross-section X-X (no 2), due to the positive  
 345 eccentricity that occurs, in addition to normal forces shear forces and bending moments also  
 346 occur. Taking into account the forces in member No 2 and in members No 1 and 3,  
 347 respectively, the effort of the cross-section was calculated on the basis of the standard  
 348 formula (1) [17]. The analysis was carried out in the elastic range. The calculations were  
 349 performed for an eccentricity with value  $e = 103.92$  mm and sections with a wall thickness of  
 350 2 mm. The value of the eccentricity  $e = 103.92$  mm results from the geometry of the truss  
 351 (angle of inclination of the diagonals), dimensions of the connected members, and the  
 352 connectors used.

$$353 \quad N_{Ed}/N_{c,Rd} + (M_{y,Ed} + \Delta M_{y,Ed})/M_{cy,Rd,com} \quad (1)$$

354 where:

355  $N_{Ed}$  – design value of the compression force,

356  $M_{y,Ed}$  – design value of the bending moment (resulting from the eccentricity at the  
 357 joint),

358  $\Delta M_{y,Ed}$  – additional bending moment (due to a shift of the centroidal axis of the  
 359 effective cross-section relative to the centroidal axis of the gross cross-section  
 360 (for class 4),  
 361  $N_{c,Rd}$  – design resistance of a cross-section for uniform compression,  
 362  $M_{cy,Rd,com}$  – moment resistance for the maximum compressive stress in a cross-section  
 363 that is subjected only to moment about y-y axis.

364 It should be noted that, in accordance with the Eurocode procedure, for the calculations of  
 365 the degree of the cross-section effort to have value in engineering terms, the random nature  
 366 of both the loads and the research model should be taken into account. Loads in  
 367 experimental studies are deterministic. In order to obtain random values, 84% quantile of H  
 368 and P load values corresponding to mean values of H and P force increased by one standard  
 369 deviation respectively were used in calculations. Table 2 presents effort values in three  
 370 analysed cross-sections for four selected load cases (set of forces H and P).

371 **Table 2.** Effort of cross-sections of truss members Nos. 1, 2, 3 (according to Fig. 15)

e=103.92 mm					
quantiles of forces		Member no. / cross-section	Internal forces		Effort of the compression and bending cross-section [%]
H [kN]	P [kN]		$N_{Ed}$ [kN]	$M_{Ed}$ [kNm]	2.0 mm
108	0	1 (1-1)	107.79	0.13	70.58
		2 (2-2)	107.96	0	69.02
		3 (3-3)	108	0	69.05
102	6	1 (1-1)	101.94	0.38	70.05
		2 (2-2)	106.55	0.55	75.19
		3 (3-3)	111.10	0	71.03
85	23	1 (1-1)	85.35	1.24	70.50
		2 (2-2)	102.32	1.78	88.29
		3 (3-3)	119.88	0.39	81.65
57	47	1 (1-1)	57.95	2.56	69.94
		2 (2-2)	92.46	3.64	105.88
		3 (3-3)	128.28	0.77	91.91

372 When force  $P$  is equal zero, there is only axial compression and the effort of every one cross-  
373 section was comparable. Introduction even a small value of the force  $P$  caused cross-section  
374 bending due to the eccentric loading of the joint. This led to an increase in effort in the  
375 measuring cross-section 2-2 in member 2 in comparison with the cross-sections 1-1 and 3-3  
376 located on members 1 and 3. In summary, the introduction of force  $P$  resulted in the  
377 following: The effort of the cross-section in member No. 2 (in the joint) is always greater than  
378 the effort of the cross-section in members Nos. 1 and 3, which of course is associated with  
379 the occurrence of shear forces and bending moments (a consequence of positive  
380 eccentricity). At the final stage of the research model loading we are dealing with a special  
381 case when the nodal cross-section is overloaded (as was confirmed by experimental tests)  
382 while maintaining the resistance condition outside the joint. Therefore, it is logical that in a  
383 design situation, exceeding the load capacity in the joint is already a signal for the designer  
384 to increase the cross-section of the entire truss chord or make reinforcing cover plates (Fig.  
385 2). So, according to the Eurocode procedure, the cross-sectional load capacity of the  
386 eccentric joint is decisive when the truss chord cross-section is selected.

## 387 **5. Summary and conclusions**

388 The conducted experimental tests were aimed at obtaining information on the  
389 behaviour of trusses made of CFS with positive eccentricity at joints, as well as obtaining the  
390 research results in the form of a register of strains and displacements of both the model and  
391 the joint. The calculations of the analyzed truss segment carried out in accordance with  
392 Eurocode procedures coincide with the results of experimental tests for a truss made of 2  
393 mm thick sections. Based on the experimental research the following conclusions were  
394 made:

- 395 • Local buckling of the hat-section chord was observed.
- 396 • The loss of stability in all research models started in the middle of the joint, in the  
397 vicinity of the measuring cross-section 2-2.

- 398       • Plastic mechanisms of failure occurred only in truss members.  
399       • Neither bolts nor holes for fasteners were destroyed.

400       Due to the high cost of experimental research, destructive tests were limited to one  
401 wall thickness, so it was not possible to explicitly confirm the research hypothesis formulated  
402 at this stage of work. In order to be able to answer the hypothesis, numerical analyses are  
403 planned for the wall thicknesses: 1.0 mm, 1.5 mm, and 4.0 mm and for different eccentric  
404 values. What is more the results of experimental tests will constitute the basis for validation  
405 of the numerical models.

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