

Experimental investigation of joint with positive eccentricity in CFS truss

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1

Abstract

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

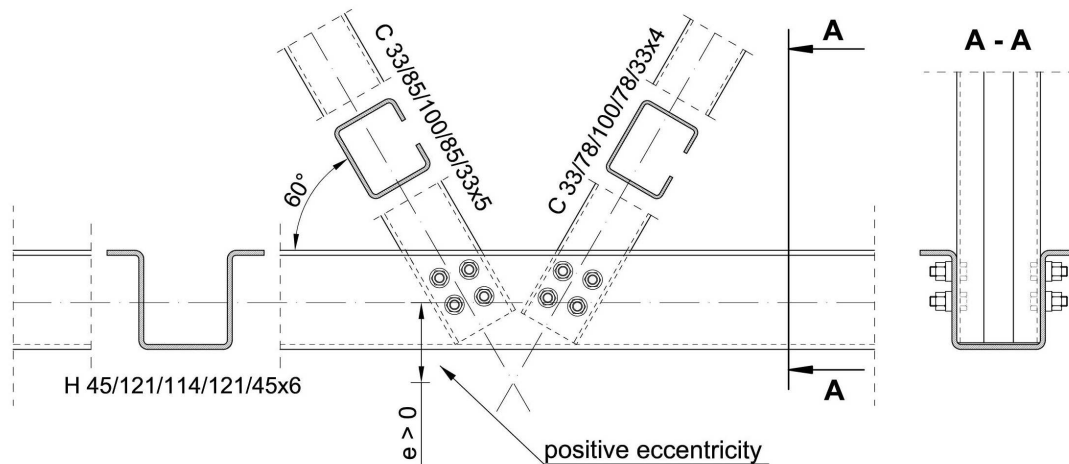
22 as roof trusses and columns. As a result, new research problems appear. Both individual
23 structural elements such as beams and columns [1–4] and entire parts of the structure,
24 including lattice girders, are subjected to testing [5–7].

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.

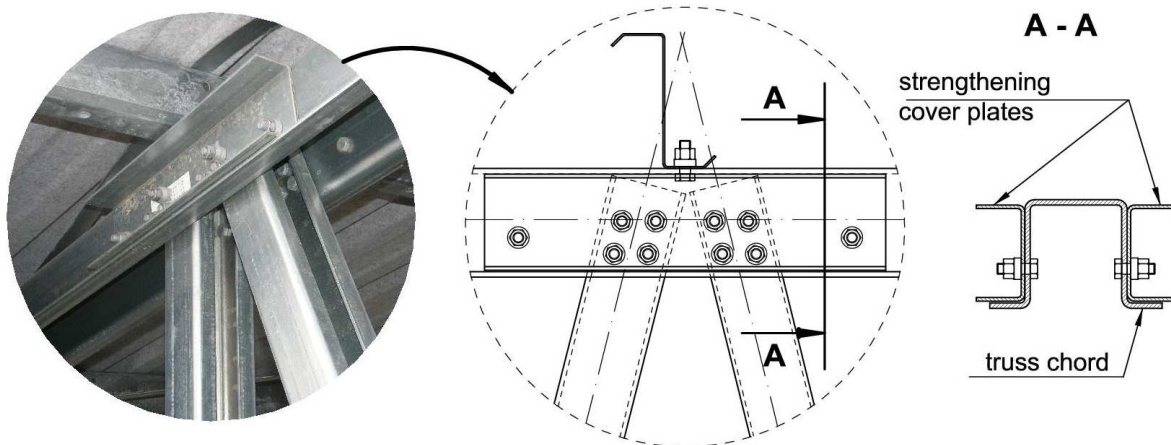
46 In engineering practice, the dimensioning of truss chords made of CFS members
47 taking into account positive eccentricities, in accordance with current European standards
48 [14,16–18], is performed using a simplified method. However, taking into account the
49 additional bending and shear between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

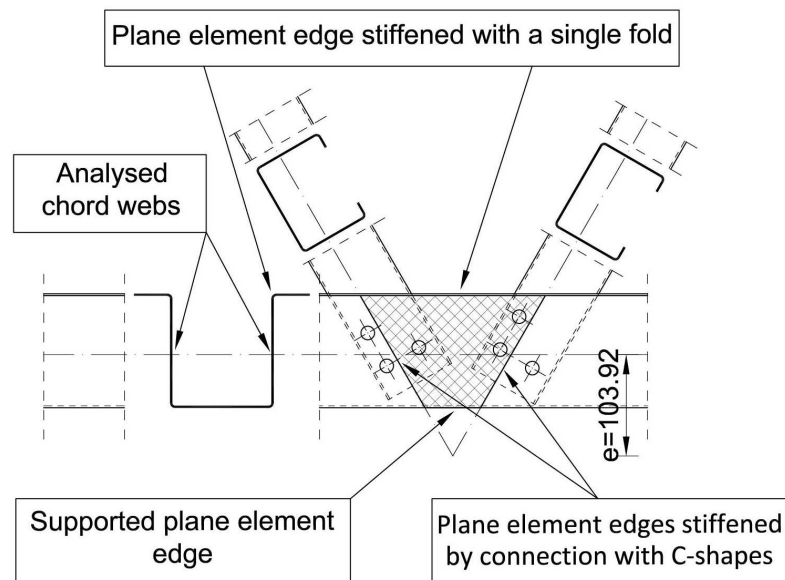
131 Thus, eccentricities in the joints of modern truss structures made from CFS open
132 cross-sections are inevitable. However, the eccentricity influence on joint resistance has not
133 been studied so far. The knowledge on sectional instability modes local or distortional
134 buckling of individual 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.

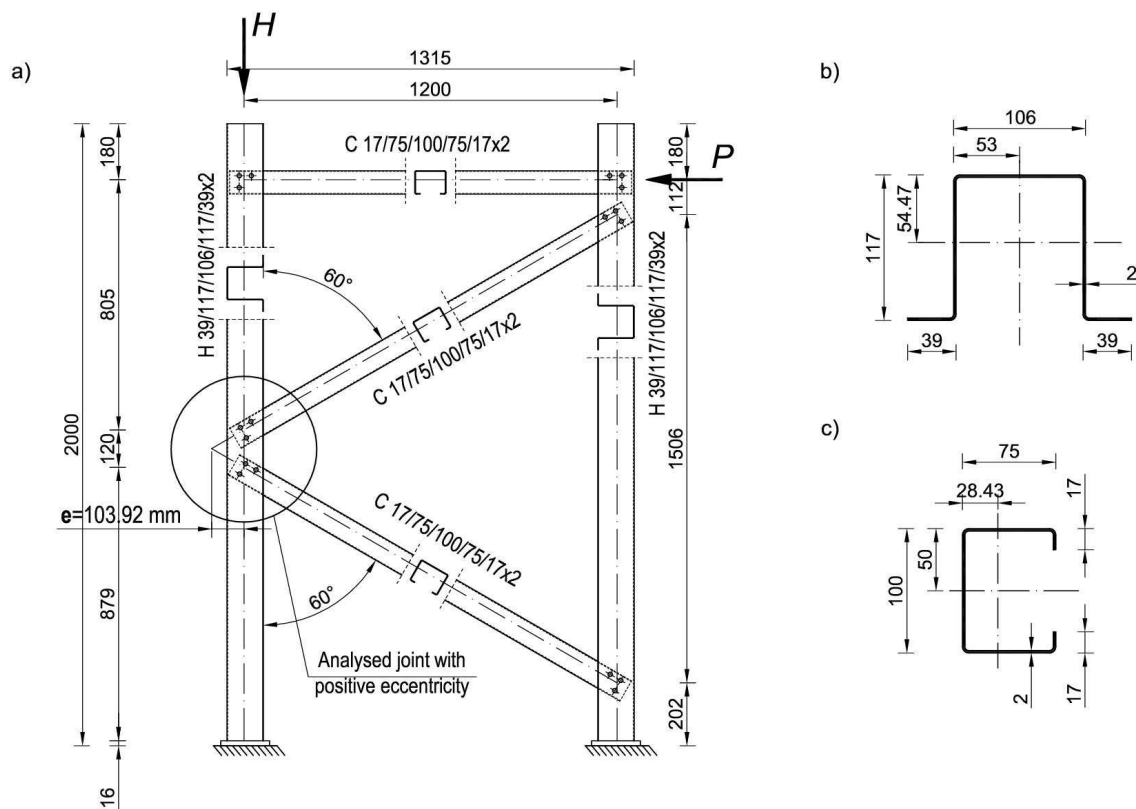
152 In order to prove the above hypothesis, experimental studies of a typical joint used by one of
 153 the well-known companies involved in the design and manufacturing of steel structures from
 154 CFS sections were undertaken at Gdańsk University of Technology. Due to the high cost of
 155 such studies, 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.

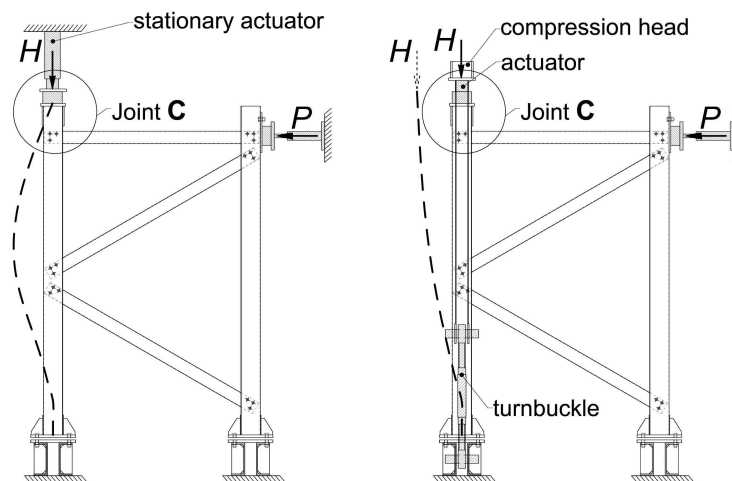


171
 172 **Fig. 4.** a) research model; static scheme, dimensions of sections used in experimental tests
 173 for: b) chords, c) diagonals. (All dimensions in mm)

174 3.1.2. Research stage

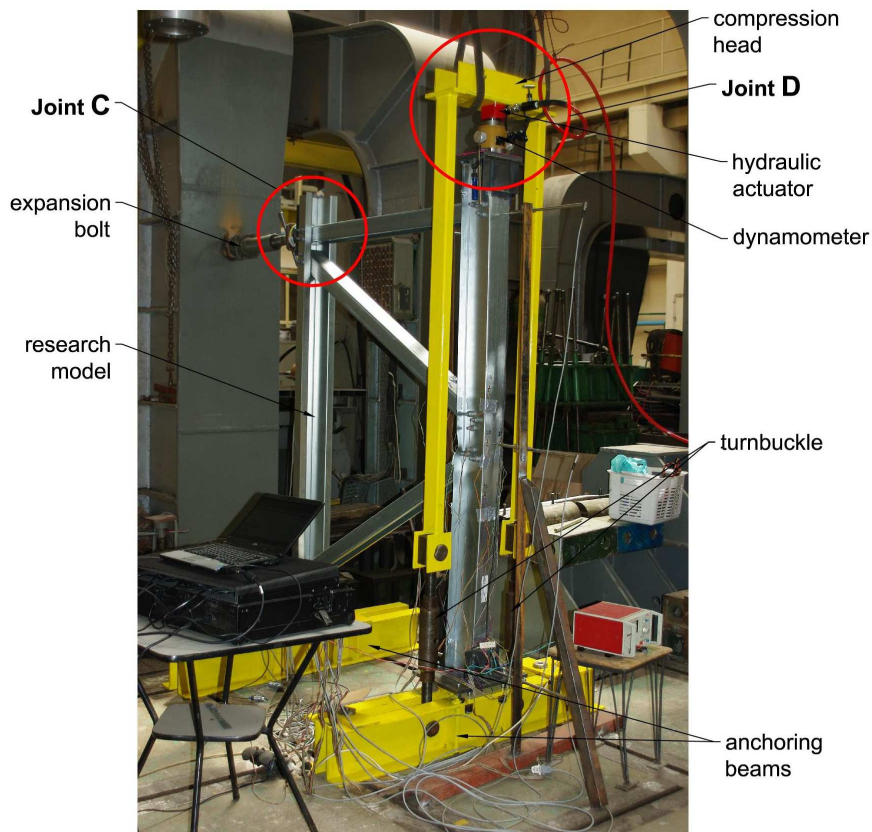
175 The static scheme of the research model is shown in Figure 4. The model was loaded
 176 with two point forces H and P . Force H causes compression of the bottom chord, while force

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

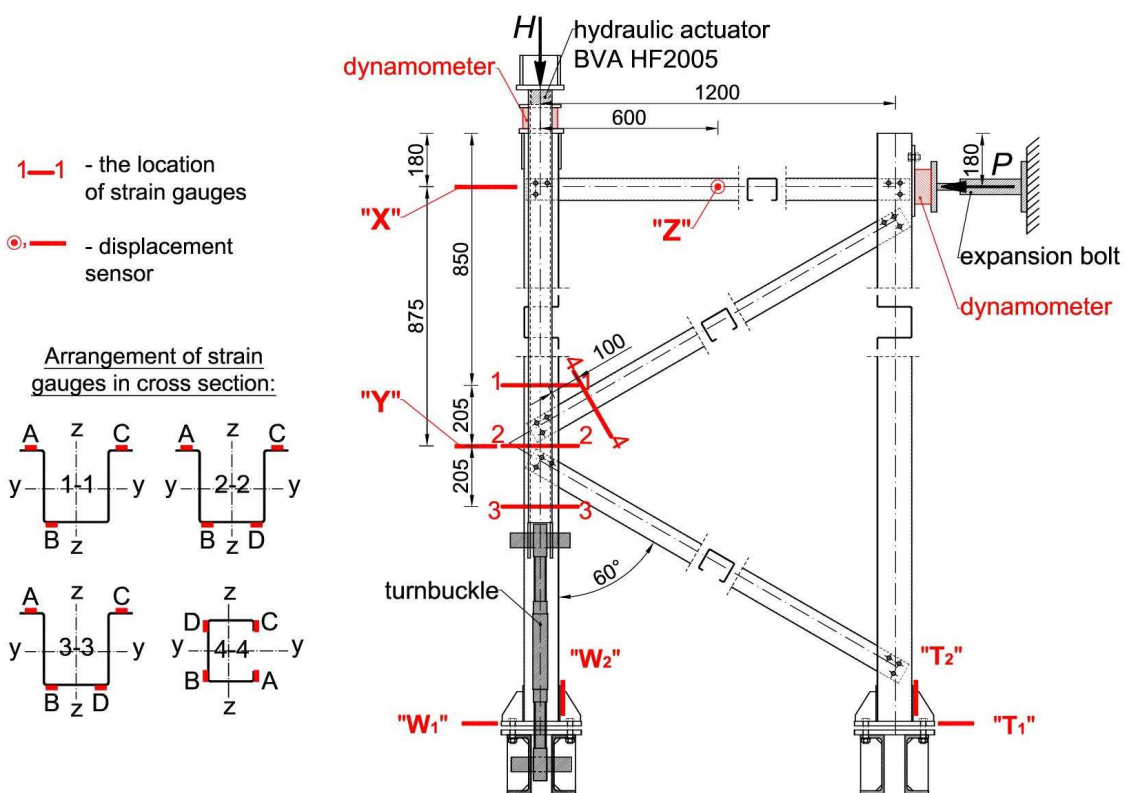
191 **Fig. 6.** Truss segment on the research station.

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 ± 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 ± 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 σ - ϵ 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:

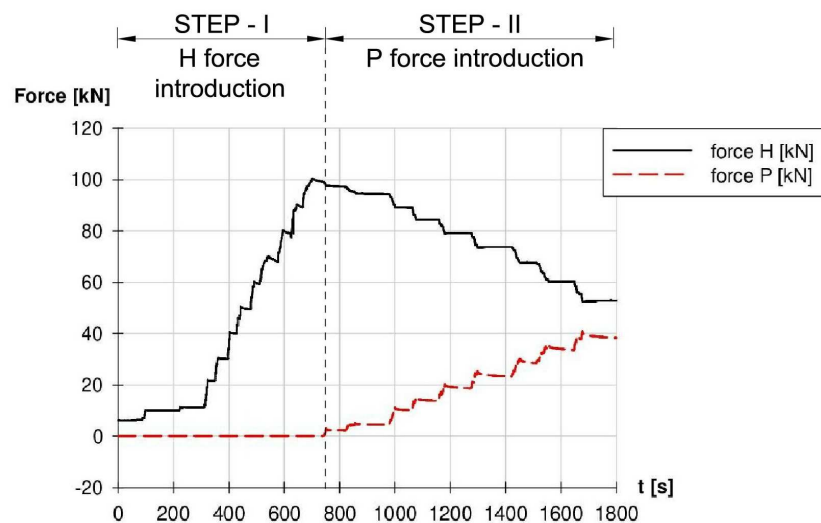
- 236 • Stage I - applying force H,
- 237 • Stage II - applying force P.

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).

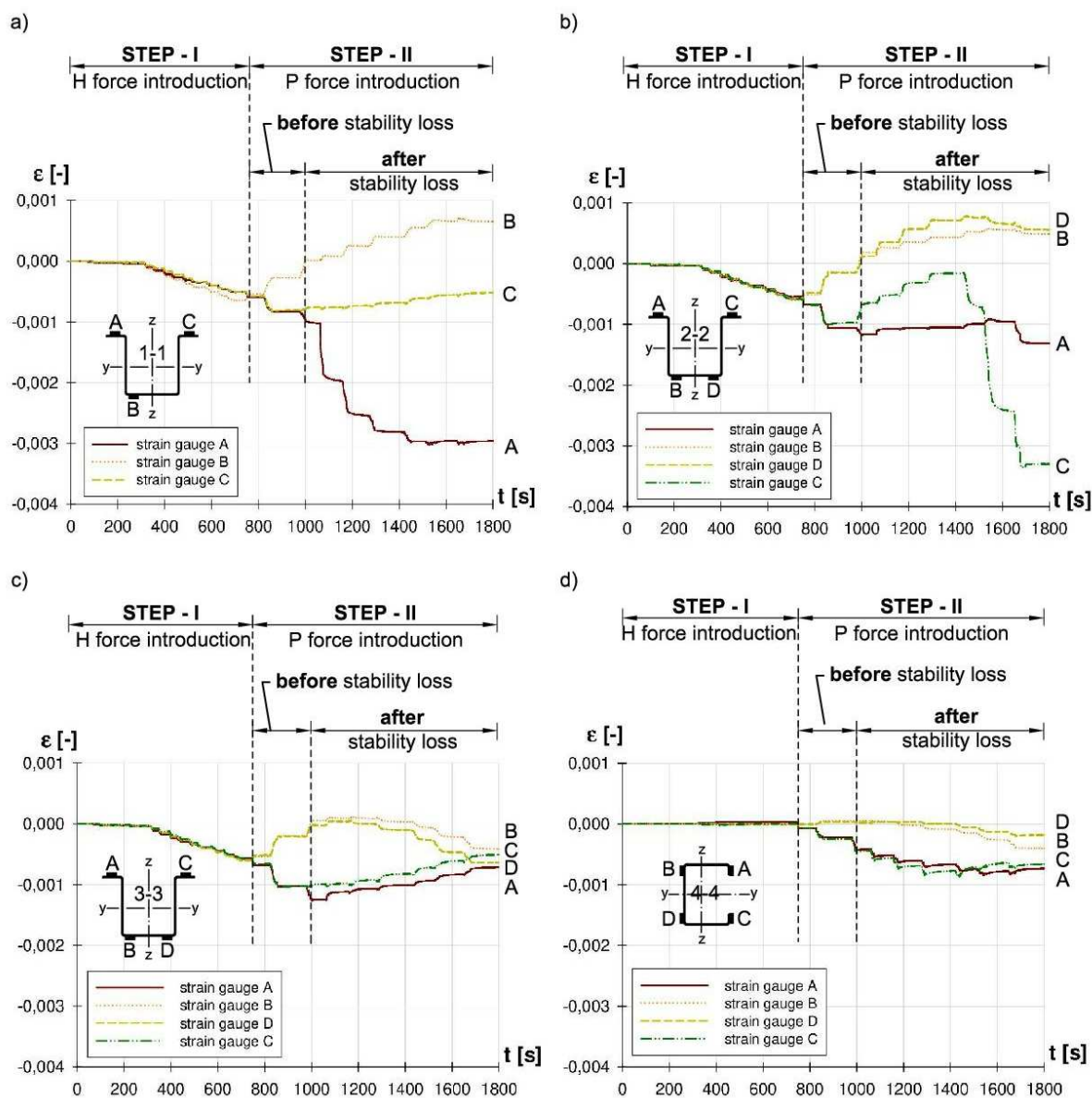


258
259 **Fig. 8.** Load history in time “t” registered during tests, as illustrated in based on the example
260 of model No. 4.

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.

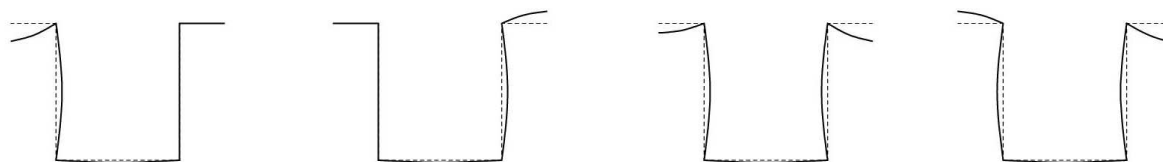


267
 268 **Fig. 9.** The course of strains as a function of time in cross-sections: a) 1-1, b) 2-2, c) 3-3, d)
 269 4-4, as illustrated in model No. 4 (description in the text).

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.

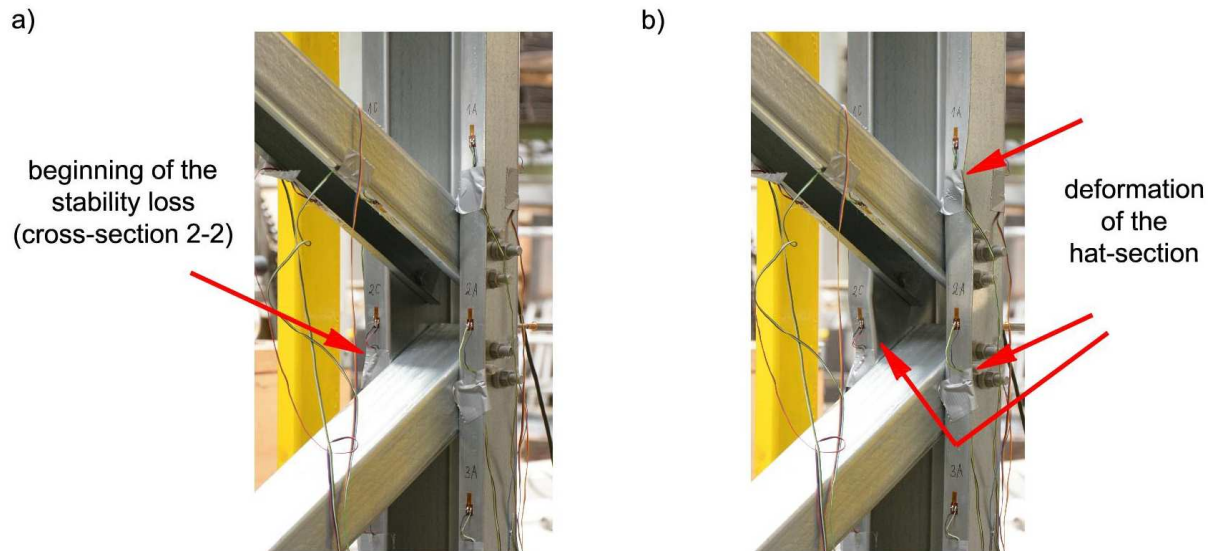
282 Forms of truss chord stability loss under compression and bending identified on the
283 basis of the destructive tests of five research models are presented in Figure 10. The loss of
284 stability always started in the measuring cross-section 2-2 and, as the load increased,
285 deformations increased 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.

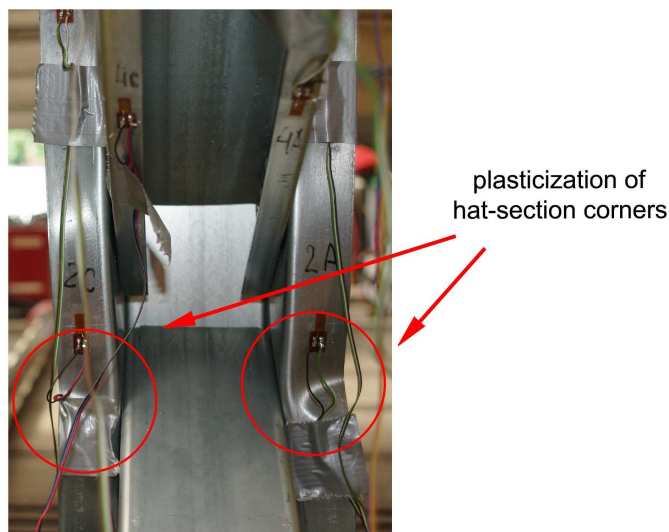
292 Deformation forms of the hat-section walls in the area of the analyzed joint at the
293 beginning of the stability loss are presented in Figure 11a, while Figure 11b presents the
294 deformation form of the hat-section observed during the experiment for the final stage of
295 loading ($P=40$ kN,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

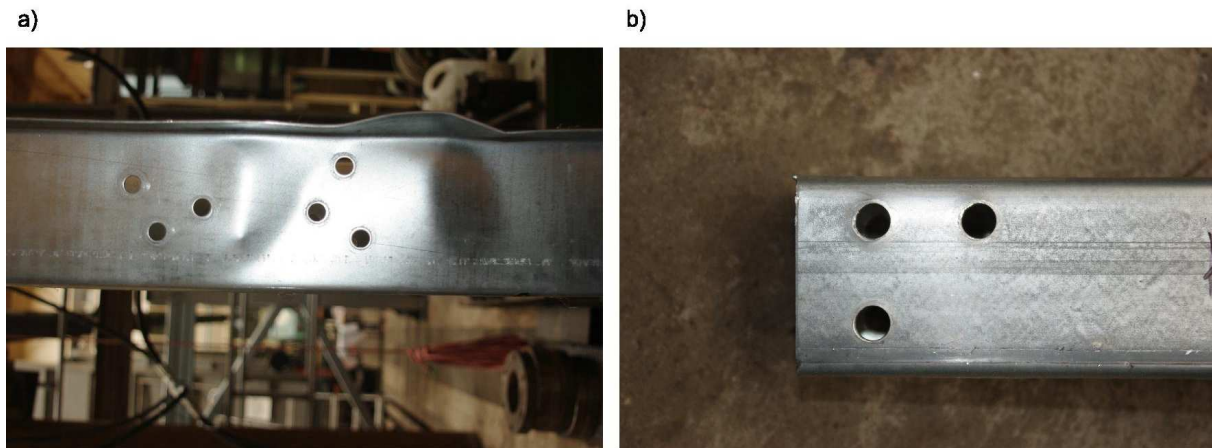
308 **Fig. 12.** Yielding of the hat-section corners in the measuring cross-section 2-2 (based on the
 309 example of research model no. 2).

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.

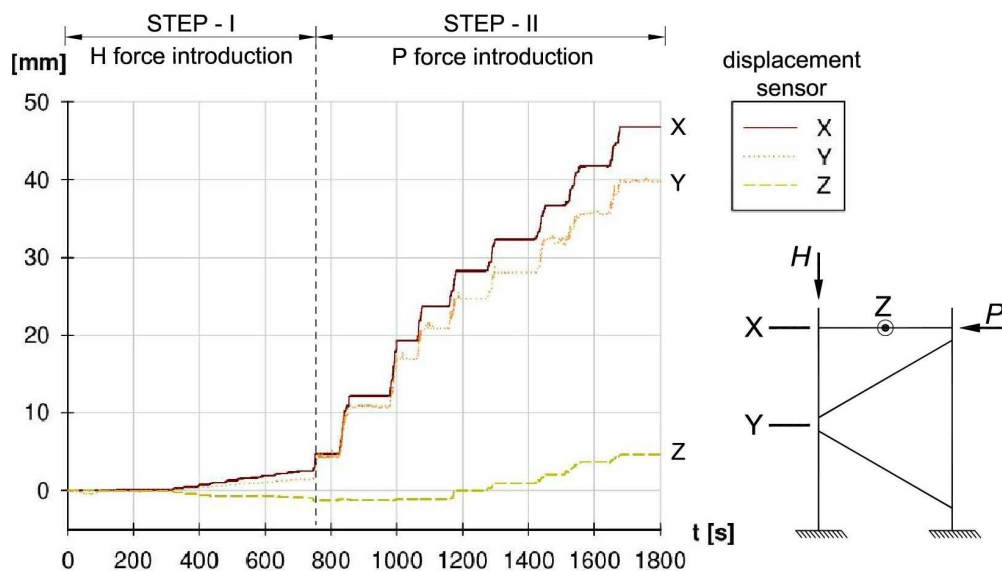
Research model number	Force P [kN]	Average value of force P P_{av} [kN]	Standard deviation s_P [-]	Corresponding force H H_{cor} [kN]	Average value of force H $H_{cor,av}$ [kN]	Standard deviation s_H [-]
Model 1	5.0	6.58	1.58	106	94.6	7.43
Model 2	8.5			96		
Model 3	7.7			89		
Model 4	5.0			95		
Model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

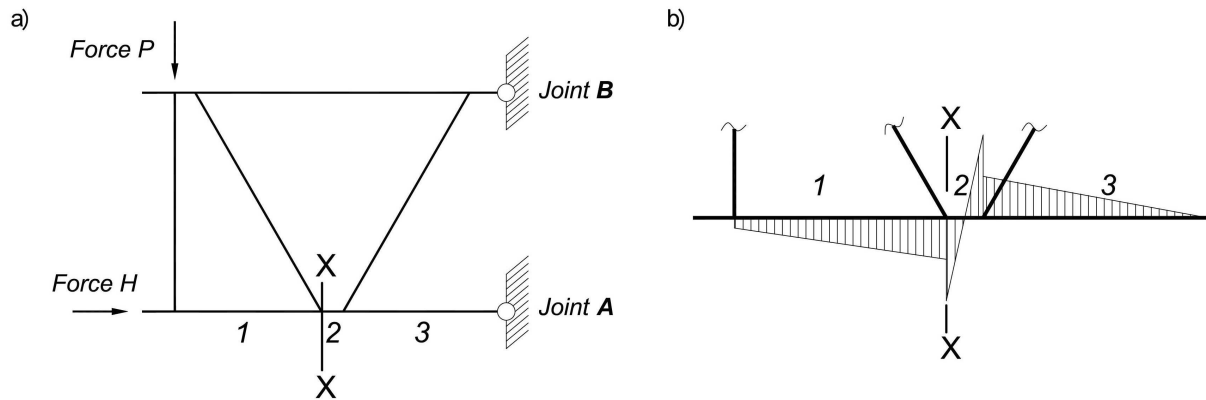
330 Model displacements at X, Y and Z measurement points are shown in Figure 14. The
 331 lateral displacement values (in the plane perpendicular to the plane of the truss) resulting
 332 from the "Z" sensor and which ranged from -2.28 mm to 4.65 mm, were considered
 333 insignificant. In turn, the control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

406 **6. References**

- 407 [1] P. Deniziak, K. Winkelmann, TS-based RSM-aided design of cold-formed steel
408 stiffened C-sectional columns susceptible to buckling, *Shell Struct. Theory Apl.* (2018) 533–
409 536.
- 410 [2] A. Crisan, V. Ungureanu, D. Dubina, Behaviour of cold-formed steel perforated
411 sections in compression: Part 2—numerical investigations and design considerations, *Thin-*
412 *Walled Struct.* 61 (2012) 97–105. <https://doi.org/10.1016/j.tws.2012.07.013>.
- 413 [3] D. Dubina, V. Ungureanu, Behaviour of multi-span cold-formed Z-purlins with bolted
414 lapped connections, *Thin-Walled Struct.* 48 (2010) 866–871.
415 <https://doi.org/10.1016/j.tws.2010.04.003>.
- 416 [4] P. Paczos, K. Magnucki, Local buckling of cold-formed thin-walled channel beams with
417 drop flange, *Adv. Trends Struct. Mech. Comput.* - Zingoni Ed. (2010).
- 418 [5] J. Jankowska-Sandberg, J. Kołodziej, Experimental study of steel truss lateral–
419 torsional buckling, *Eng. Struct.* 46 (2013) 165–172.
420 <https://doi.org/10.1016/j.engstruct.2012.07.033>.
- 421 [6] D. Visy, S. Adany, A.L. Joo, Finite element studies on the global buckling of cold-
422 formed trusses., *Eighth Int. Conf. Adv. Steel Struct.* (2015) 1–18.
- 423 [7] J.L. Dawe, Y. Liu, J.Y. Li, Strength and behaviour of cold-formed steel offset trusses,
424 *J. Constr. Steel Res.* 66 (2010) 556–565. <https://doi.org/10.1016/j.jcsr.2009.10.015>.



- 425 [8] M. Gordziej-Zagórska, E. Urbańska-Galewska, P. Deniziak, Ł. Pyrzowski, Truss
426 joint with positive eccentricities – experimental research, *Civ. Environ. Eng. Rep.* (2017)
427 107–123.
- 428 [9] J. Bródka, M. Broniewicz, *Konstrukcje stalowe z rur*, Arkady, Warszawa, 2001.
- 429 [10] J. Bródka, M. Broniewicz, *Konstrukcje stalowe z kształtowników zamkniętych*, Polskie
430 Wydawnictwo Techniczne, Rzeszów, 2016.
- 431 [11] J.A. Packer, J. Wardenier, Y. Kurobane, D. Dutta, N. Yeomans, *Design guide for*
432 *rectangular hollow (RHS) joints under predominantly static loading.*, CIDECT Construction
433 with hollow steel sections, Koln, 1992.
- 434 [12] J. Wardenier, Y. Kurobane, J.A. Packer, D. Dutta, N. Yeomans, *Design guide for*
435 *circular hollow section (CHS) joints under predominantly static loading.*, CIDECT
436 Construction with hollow steel sections, Koln, 1991.
- 437 [13] Y. Chen, J. Yang, K. Hu, Parametric study and formulae of SCFs for positive large
438 eccentricity CHS N-joints, *J. Constr. Steel Res.* 120 (2016) 117–131.
439 <https://doi.org/10.1016/j.jcsr.2016.01.005>.
- 440 [14] EN 1993-1-8, Eurocode 3: Design of steel structures - Part 1-8: Design of joints, Eur.
441 Comm. Stand. (2005).
- 442 [15] C. Basaglia, D. Camotim, GBT-based buckling analysis of cold-formed steel trusses,
443 *Proc. Sixth Int. Conf. Thin Walled Struct. Timisoara Rom.* (2011) 149–156.
- 444 [16] EN 1993-1-1, Eurocode 3: Design of steel structures - Part 1-1: General rules and
445 rules for buildings, Eur. Comm. Stand. (2005).
- 446 [17] EN 1993-1-3, Eurocode 3: Design of steel structures - Part 1-3: General rules -
447 Supplementary rules for cold-formed members and sheeting, Eur. Comm. Stand. (2006).
- 448 [18] EN 1993-1-5, Eurocode 3: Design of steel structures - Part 1-5: General rules - Plated
449 structural elements, Eur. Comm. Stand. (2006).
- 450 [19] M. Gordziej-Zagórska, E. Urbańska-Galewska, Ł. Pyrzowski, P. Deniziak, A.
451 Łukowicz, Preliminary experimental research on stability of truss joint with positive



- eccentricity., *Recent Prog. Steel Compos. Struct. Proc. 13th Int. Conf. Met. Struct. Zielona Góra Pol.* (2016) 425–432.
- [20] J. Ye, S.M. Mojtabaei, I. Hajirasouliha, Local-flexural interactive buckling of standard and optimised cold-formed steel columns, *J. Constr. Steel Res.* 144 (2018) 106–118. <https://doi.org/10.1016/j.jcsr.2018.01.012>.
- [21] J. Ye, I. Hajirasouliha, J. Becque, Experimental investigation of local-flexural interactive buckling of cold-formed steel channel columns, *Thin-Walled Struct.* 125 (2018) 245–258. <https://doi.org/10.1016/j.tws.2018.01.020>.
- [22] A. Łukowicz, E. Urbańska-Galewska, Deformations of innovative cold-formed GEB section., *7th Eur. Conf. Steel Compos. Struct. EUROSTEEL.* (2014) 1–6.
- [23] M.S.H.M. Sani, F. Muftah, C.S. Tan, Experimental Analysis of Cold-Formed Steel C-Sections with the Notch Subjected to Axial Compression, *KSCE J. Civ. Eng.* 24 (2020) 1228–1239. <https://doi.org/10.1007/s12205-020-2340-z>.
- [24] M.R. Haidarali, D.A. Nethercot, Finite element modelling of cold-formed steel beams under local buckling or combined local/distortional buckling, *Thin-Walled Struct.* 49 (2011) 1554–1562. <https://doi.org/10.1016/j.tws.2011.08.003>.
- [25] D. Dubina, V. Ungureanu, L. Gîlia, Experimental investigations of cold-formed steel beams of corrugated web and built-up section for flanges, *Thin-Walled Struct.* 90 (2015) 159–170. <https://doi.org/10.1016/j.tws.2015.01.018>.
- [26] M. Kotełko, Load-capacity estimation and collapse analysis of thin-walled beams and columns—recent advances, *Thin-Walled Struct.* 42 (2004) 153–175. [https://doi.org/10.1016/S0263-8231\(03\)00055-7](https://doi.org/10.1016/S0263-8231(03)00055-7).
- [27] C. Yu, B.W. Schafer, Simulation of cold-formed steel beams in local and distortional buckling with applications to the direct strength method, *J. Constr. Steel Res.* 63 (2007) 581–590. <https://doi.org/10.1016/j.jcsr.2006.07.008>.
- [28] A. Łukowicz, E. Urbańska-Galewska, M. Gordziej-Zagórska, Experimental testing of innovative cold-formed GEB section., *Civ. Environ. Eng. Rep.* (2015) 129–140.



- 479 [29] C.H. Pham, G.J. Hancock, Tension field action for cold-formed sections in shear, J.
480 Constr. Steel Res. 72 (2012) 168–178. <https://doi.org/10.1016/j.jcsr.2011.12.001>.
- 481 [30] C.H. Pham, G.J. Hancock, Numerical investigation of longitudinally stiffened web
482 channels predominantly in shear, Thin-Walled Struct. 86 (2015) 47–55.
483 <https://doi.org/10.1016/j.tws.2014.09.005>.
- 484 [31] A. Crisan, V. Ungureanu, D. Dubina, Behaviour of cold-formed steel perforated
485 sections in compression. Part 1—Experimental investigations, Thin-Walled Struct. 61 (2012)
486 86–96. <https://doi.org/10.1016/j.tws.2012.07.016>.
- 487 [32] A. Crisan, V. Ungureanu, D. Dubina, Influence of web members on the in-plane and
488 out-of-plane capacities of steel storage upright frames, Thin-Walled Struct. 81 (2014) 175–
489 184. <https://doi.org/10.1016/j.tws.2013.10.024>.
- 490 [33] M. Krajewski, P. Iwicki, Stability and load bearing capacity of a braced truss under
491 upward wind loading, Eurosteel. (2017).
- 492 [34] C. Dizdar, E. Baran, C. Topkaya, Strength and stiffness of floor trusses fabricated
493 from cold-formed steel lipped channels, Eng. Struct. 181 (2019) 437–457.
494 <https://doi.org/10.1016/j.engstruct.2018.12.041>.
- 495 [35] M. Reda, T. Sharaf, A. ElSabbagh, M. ElGhandour, Behavior and design for
496 component and system of cold-formed steel roof trusses, Thin-Walled Struct. 135 (2019) 21–
497 32. <https://doi.org/10.1016/j.tws.2018.10.038>.
- 498 [36] L. Song, W. Yan, C. Yu, Z. Xie, Q. Tan, Flexural behavior investigation of the CFS
499 truss beams with self-piercing riveted connection, J. Constr. Steel Res. 156 (2019) 28–45.
500 <https://doi.org/10.1016/j.jcsr.2019.01.014>.
- 501 [37] N. Konkong, T. Aramraks, K. Phuvoravan, Buckling length analysis for compression
502 chord in cold-formed steel cantilever truss, Int. J. Steel Struct. 17 (2017) 775–787.
503 <https://doi.org/10.1007/s13296-017-6031-7>.
- 504 [38] AISI, North American specification for the design of cold-formed steel structural
505 members., Am. Iron Steel Inst. (2012).



- 506 [39] R. Zaharia, D. Dubina, Stiffness of joints in bolted connected cold-formed steel
507 trusses, *J. Constr. Steel Res.* 62 (2006) 240–249. <https://doi.org/10.1016/j.jcsr.2005.07.002>.
- 508 [40] J.B.P. Lim, D.A. Nethercot, Stiffness prediction for bolted moment-connections
509 between cold-formed steel members, *J. Constr. Steel Res.* 60 (2004) 85–107.
510 [https://doi.org/10.1016/S0143-974X\(03\)00105-6](https://doi.org/10.1016/S0143-974X(03)00105-6).
- 511 [41] J.B.P. Lim, G.J. Hancock, G. Charles Clifton, C.H. Pham, R. Das, DSM for ultimate
512 strength of bolted moment-connections between cold-formed steel channel members, *J.*
513 *Constr. Steel Res.* 117 (2016) 196–203. <https://doi.org/10.1016/j.jcsr.2015.10.005>.
- 514 [42] D. Dubina, Structural analysis and design assisted by testing of cold-formed steel
515 structures, *Thin-Walled Struct.* 46 (2008) 741–764. <https://doi.org/10.1016/j.tws.2008.01.030>.
- 516 [43] C. Mathieson, K. Roy, G.C. Clifton, A. Ahmadi, J.B.P. Lim, Failure mechanism and
517 bearing capacity of cold-formed steel trusses with HRC connectors, *Eng. Struct.* 201 (2019)
518 109741. <https://doi.org/10.1016/j.engstruct.2019.109741>.
- 519 [44] C. Mathieson, K. Roy, G.C. Clifton, A. Ahmadi, M. Rehan, J.B.P. Lim, Novel pin
520 jointed moment connection for cold-formed steel trusses, *Steel Compos. Struct.* (2019) 453–
521 467.
- 522 [45] V. Ungureanu, M. Kotelko, J. Grudziecki, Plastic mechanisms for thin-walled cold-
523 formed steel members in eccentric compression, (2016).
- 524 [46] M. Kotelko, *Nośność i mechanizmy zniszczenia konstrukcji cienkościennych.*,
525 Wydawnictwo WNT, Warszawa, 2011.

526