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EXPERIMENTAL AND NUMERICAL INVESTIGATION OF TWO-CHORD STEEL COLUMNS FILLED WITH CONCRETE

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Abstract

The paper presents experimental and numerical results for selected two-chord steel columns filled with concrete. The laboratory tests were accompanied by numerical analysis carried out using general purpose, finite element program ABAQUS. Based on heretofore research presented in this paper, one may assume that the application of dual chord steel – concrete columns is economically very profitable. A considerable growth of load bearing capacity (42-55%) and rigidity of those members in relation to steel columns is obtained with insignificant increase of the cost.
The study was conducted as a part of the research project nr 4TO7E01528, funded by the Polish Ministry of Science and Higher Education.

Streszczenie

W artykule przedstawiono wyniki badań dwugałęziowych słupów stalowych wypełnionych betonem. Dla wybranych elementów badanych doświadczalnie, przeprowadzono analizę numeryczną metodą elementów skończonych, przy zastosowaniu programu ABAQUS.
Z dotychczasowych badań wynika, że stosowanie tego typu słupów jest ekonomicznie bardzo uzasadnione. Wypełnienie betonem znacznie zwiększyło zarówno ich nośność (o 42-55%), jak i sztywność, w stosunku do słupów stalowych.
Przedstawione badania dwugałęziowych słupów wypełnionych betonem zostały wykonane w ramach projektu badawczego nr 4TO7E01528 finansowanego przez Ministerstwo Nauki i Informatyzacji.

Keywords: **ABAQUS; Columns filled with concrete; Composite columns; Load carrying capacity; Two-chord columns.**

1. INTRODUCTION

The existing building standards for steel, concrete, and composite structures, for example Eurocode 4 [8], do not provide any design guidance for the two-chord steel columns filled with concrete. However, today's construction practice shows more common usage of such structural elements. As it follows from an engineering practice, columns of that type would be broadly used in construction, in the floor method of the underground storey building, and in raising buildings with the use of "up&down" method, i.e. Filling up with concrete can also be applied in strengthening of steel columns in existing structures

The studies on increasing loading capacity and stiffness of two-chord steel columns, using concrete fill, were conducted already in 1935 [1]. The research covered only columns built of two C channels connected through batten plates. The studies presented in this paper are focused on columns built of two connected wide flange I beams.

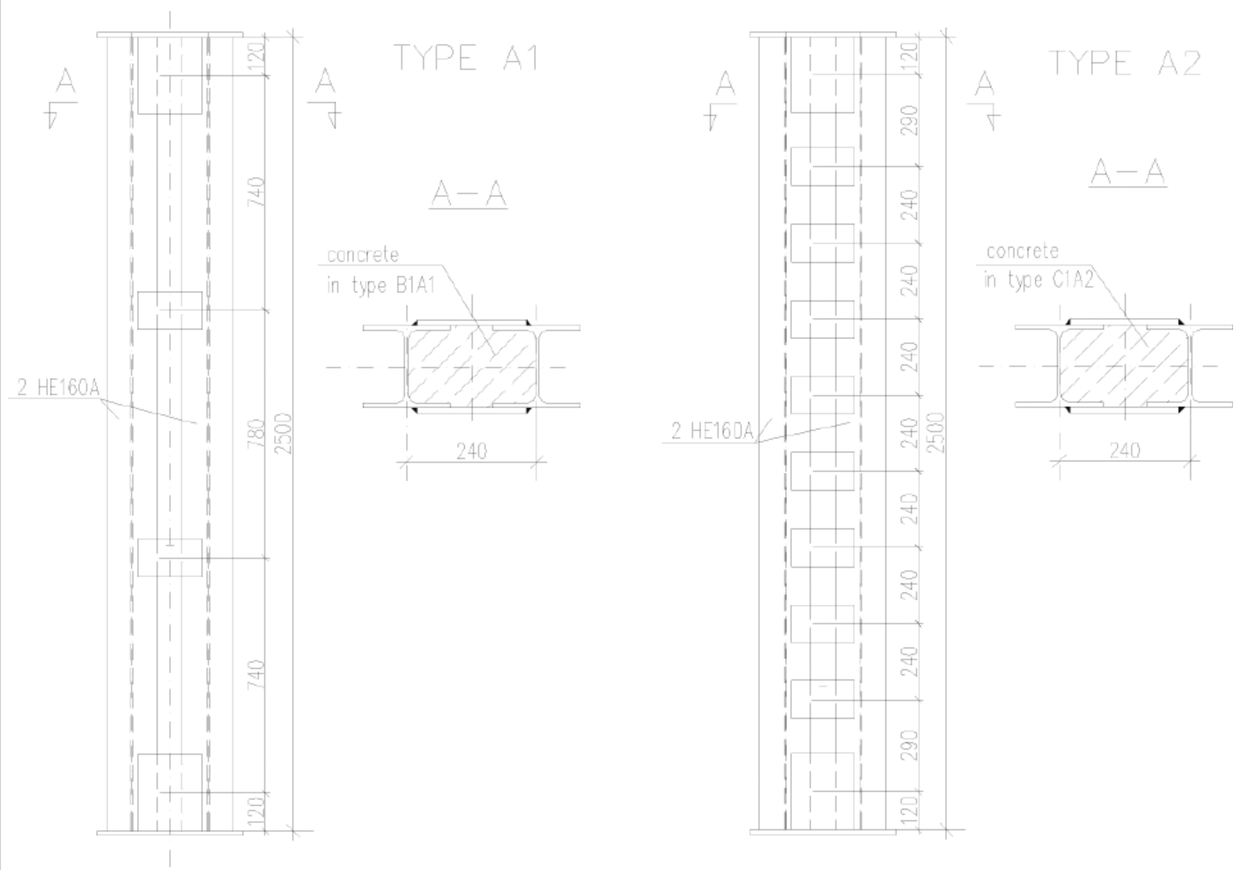

Figure 1.
The tested models for columns A1, B1A1 and A2, C1A2

The main objectives of the study are to determine how the concrete fill affects the structural behavior of such columns and to estimate their loading carrying capacity.

The tests were conducted on the steel and steel-concrete elements in the natural scale.

Finally, the degree of accuracy of the proposed model is investigated by means of the comparison between the experimental results and the numerical predictions. Such comparison deals with ultimate resistance and deformation of selected columns.

The related question is, if and under what conditions such columns can be treated as composite structural elements.

2. DESCRIPTION OF THE EXPERIMENTAL MODEL

So far twenty 2500 mm high columns have been tested, two steel and eighteen filled with concrete. The columns consisted of two wide flange I-beams HE160A, made of steel S235JRG2. Different spacing of buttenings (780 mm and 240 mm) and different concrete strength classes (C20/25 or C50/60) were applied. Figure 1 shows the setup for the tested columns.

The members were tested in the DB 600 first precision class hydraulic press. For all tested cases the top end of a column was loaded through a hinge while the base was fixed.

After the vertical position of the element in the press it has been corrected.

Inductive gauges for the measurement of the both planes horizontal deflection of the column as well as the vertical shortening, have been mounted.

During the laboratory tests the specimens were axially loaded up to the failure. The force was applied to the entire transversal cross-section and was growing from zero, every 200 kN to the maximum value. The recorded test data consisted of strains in concrete and steel, vertical contraction and horizontal deflections. Location of the inductive gauges on the tested columns is shown in Fig. 2.

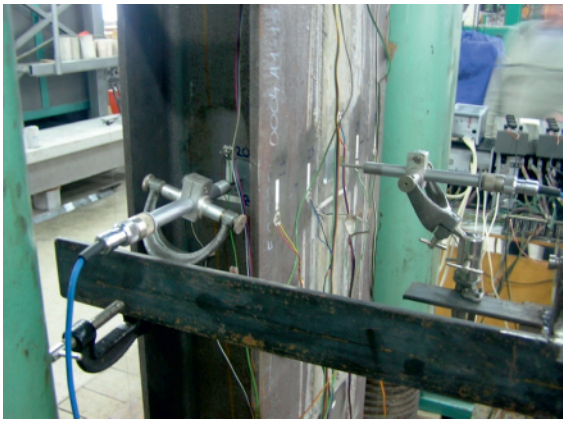

Figure 2.
Location of gauges on the middle part of the tested column

The selected tested columns are described in Table 1.

Table 1 presents only data for the columns which were also analyzed numerically.

Table 1.
Columns tested experimentally and analyzed numerically

Specimen	Materials	Spacing of lacings [mm]
A1	Steel	780
A2	Steel	240
B1A1*	Steel filled with concrete C20/25	780
C1A2*	Steel filled with concrete C20/25	240

* three elements of each type of columns were tested

3. RESULTS OF EXPERIMENTAL TESTS

When testing the steel columns A1 and A2 there was no deflection detected until critical loading. Within the second phase of the tests, after reaching the ultimate loading level, the substantial deflection was registered, accompanied by local buckling at the flanges. As it is shown in Table 2 the ultimate loading is almost the same for both steel columns. The largest horizontal displacement, along the direction parallel to the webs, was measured for column A1 at the distance of 650 mm from the upper end. At the middle height the deflection was 2.5 mm. For column A2, maximum horizontal displacement was located at larger distance of 950 mm. In the direction perpendicular to the webs displacements were several times smaller. Maximum shortening registered during the tests for columns A1 and A2 was 7 and 11 mm respectively. The largest plastic strains are located at the internal flanges, connected through the buttenings.

Table 2.
Experimentally determined ultimate loading

Specimen	Ultimate loading [kN]	Ultimate loading – mean value [kN]
A1	2160	**2160**
A2	2140	**2140**
B1A1-1	3200	
B1A1-2	3000	**3060**
B1A1-3	2980	
C1A2-1	3440	
C1A2-2	3200	**3333**
C1A2-3	3360	

For the composite columns type B1A1, made of steel and concrete, the first vertical cracks appeared after the loading reached 2200 kN. They were localized at the connections between the flanges and the concrete cores, at the areas between the lacings. The ultimate loading was accompanied mainly by crashed concrete at the upper segment between the lacings and local buckling of the external flanges at the upper part of the column [3], [4]. When the loading reached the ultimate level, the maximum horizontal displacement in the middle of the column was 2 mm and the total contraction was 9 mm.

The behavior of the composite columns type C1A2 (see Table 1) was analogues to the columns type B1A1. However, there were no vertical cracks along the connection between the steel flanges and the concrete core. It was also determined that the depth and extension of the crashed zone was much smaller. Concrete was crashed and delaminated in the zone covering three upper areas between the lacings. For the columns type C1A2 the average horizontal deflection (in the direction parallel to the webs) caused by the ultimate loading was around 5 mm. The larges deflection, measured at the end of the test, was for the point located 650 mm below the upper end of the column (in the direction parallel to the webs). The total contraction was equal to 10 mm.

Example, load-strain curves for the composite columns, for the steel and concrete parts, are presented in Figure 3. Presented strains were measured at the upper segment of the columns.

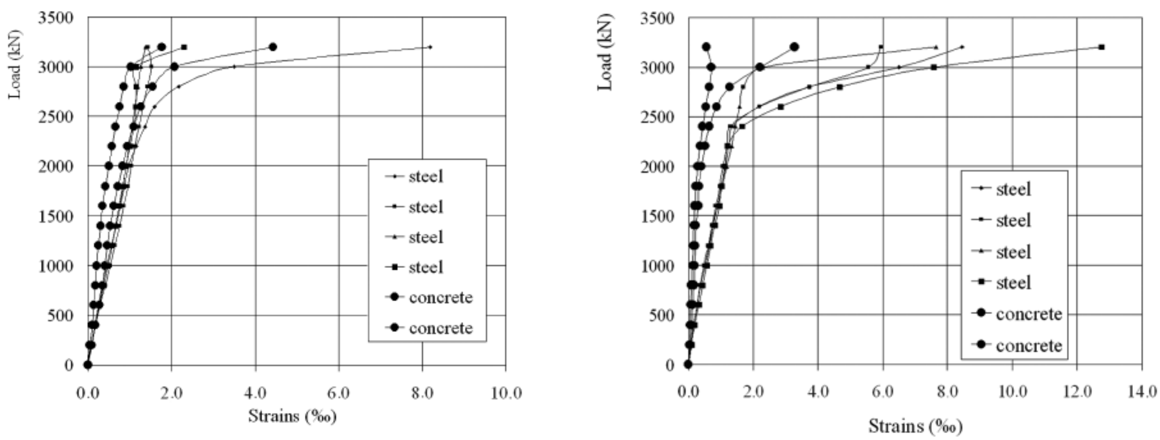

Figure 3.
Example load-strain curves for the composite columns, for the steel and concrete parts in type of columns: a) B1A1 and b) C1A2

4. FINITE ELEMENT ANALYSIS

The nonlinear analyses, carried beyond the critical point, were conducted using general purpose, finite element program ABAQUS/Standard Version 6.6-1 [6]. The FE models of the columns were developed using geometric data shown in Figure 1. All steel components are modeled with four node shell elements with nodes located in the mid planes. Additionally, there is a rigid plate build of solid elements attached to the top end plate. The rigid plate is used in the FE models to transmit uniformly loading subject to the top end of a column. This part is defined in the FE model as a rigid body accompanied with the reference point, located close to the mid point on the top surface. Imperfection, in a form of load eccentricity, can be applied by moving the reference point from mid point location. The loading is applied through the reference point as a prescribed vertical displacement with horizontal movements constrained. The vertical reaction and displacement recorded for the reference points are used to formulate equilibrium paths (see Figure 4). After preliminary calculations and comparison of the results for different options such as merged nodes and contact, this transmission was realized in the FE model by merging coinciding nodes.

The concrete core is modeled with solid elements. The mesh of the concrete core is divided into elements in such a way that external nodes coincide with nodes representing steel components. All possible interactions can be traced by defining contact between concrete and steel components or by merging nodes.

To cover the post buckling behavior of the columns it is necessary to represent correctly inelastic material properties in the FE models. The steel is modeled as an elastic-plastic material with isotropic hardening [6]. The Hüber-Misses yield surface is defined by giving the value of the uniaxial yield stress as a function of uniaxial equivalent plastic strain. The input is based on the true stress – strain curve, recalculated based on the coupon test data.

In ABAQUS, complex, inelastic materials can be modeled by combining different material behaviors. The elastic behavior of concrete is assumed to be isotropic and linear. The inelastic concrete behavior is modeled using model *Concrete Damaged Plasticity with two sub options *Concrete Compression Hardening, and *Concrete Tension Stiffening [6]. It is recommended for plain and reinforced concrete [6]. This model uses concepts of isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity to represent the inelastic behavior of concrete and other brittle materials. The material parameters for concrete were obtained from tests on cast cylinders and also approximated using proportions taken from [7].

Due to high nonlinearities accompanying traced inelastic unstable collapse and postbuckling behavior the Riks analysis was chosen as analysis procedure [6]. The solution proceeds incrementally along the static equilibrium path, providing loads and displacements which satisfy equilibrium equations with acceptable tolerance. For nonlinear problems, as one described here, solutions are depending on the magnitude of increments. Although usually they follow initially the same equilibrium path, very often the solution near the critical point experiences problems with convergence and premature termination. For all cases considered here the fast and proper solution was obtained for the default set of analysis parameters and limited number of points defining inelastic material response.

Figure 4 compares two equilibrium paths for two FE models of the columns type B1A1. The models represent two extreme solutions for modeling concrete – steel interaction. In the first model the meshes for concrete core and steel components are not connected and there is frictionless one way contact defined. In the second model both meshes are fully connected through merged nodes. Figure 4 shows that there is little difference between the results. The relative large deformation is captured by concrete tensile softening.

In the present work the imperfections, always present in actual tests, were applied as a loading eccentricity. Figure 5 presents three equilibrium paths for the column A2 and three different locations of the reference

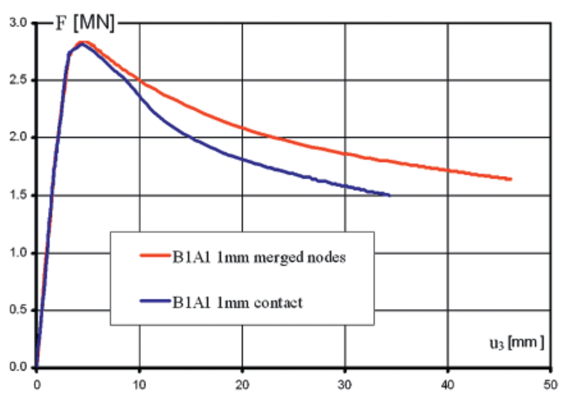

Figure 4.
Comparison of results for case B1A1 and two models, with contact and merged nodes

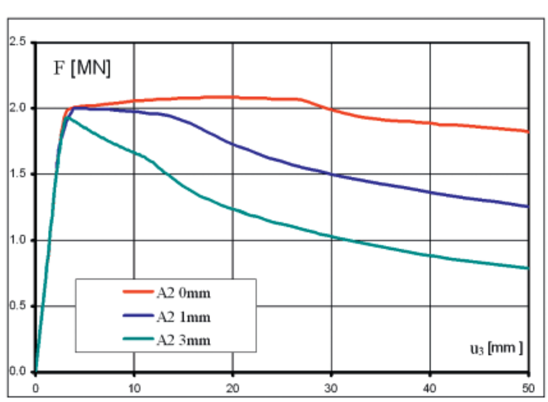

Figure 5.
Equilibrium paths for case A2 and three different locations of reference point

point where the prescribed displacement (loading) was applied. The considered positions of the reference point are e = 0, 1, and 3 mm along y (2) axis (parallel to the webs). The comparison of the results shows small reduction of critical load and visible change in location of plastic zones as displayed in Figure 5, especially for e = 0 mm.

5. ANALYSIS OF EXPERIMENTAL AND NUMERICAL RESULTS

Preliminary numerical analysis, illustrated in Figures 4 and 5, allowed for the selection of FE models for further comparison study based on the experimental and the numerical results. The chosen FE models have initial eccentricity of 1 mm applied and the steel-concrete interface modeled through merged nodes. The repeatable behavior of these models was closest to the experimental responses.

The tests on the steel columns showed that the ultimate loading is independent on the spacing of buttenings. The load-strain diagrams, for both steel columns, show that the largest strains are located at the internal flanges, connected by the buttenings [5]. Based on the relations load-strains for both steel columns it was determined that the largest increment of strains was localized in the internal flanges, connected with the lacings.

The concrete fill increased significantly the magnitude of the ultimate loading, by 42% at an average for the columns type B1A1 and by 55% for the columns type C1A2 (see Table 1). It should be noted that the applied concrete C 20/25 has the lowest strength class recommended by standards for composite structures. Similarly to the steel columns, the damage zone for the composite specimens was localized at the upper segments. At that area the deformation of steel and concrete parts was also the largest.

The composite columns type C1A2 showed about 10% larger ultimate loading than the columns type B1A1. The overall behavior of the composite columns shows that although the steel part takes considerable amount of the loading, the ultimate strength is determined by the concrete core. The experimental tests and numerical analysis have proven, what was expected, that increased number of lacings confines the deformation of concrete and increases the ultimate loading of the composite columns. In the columns C1A2 the local deformation in the flanges was also smaller. Similarly to the steel columns, the maximum deflection of the composite columns with concentrated lacings (type C1A2), was along the direction parallel to the webs and appeared in the cross-section localized lower than it was in the columns B1A1. It means that the spacing of the lacings affects the buckling mode for the composite columns.

Figures 6, 7, 8, and 9 show the equilibrium paths in terms of the relation between load and total contraction, received from the experiments and the FE calculations. It can be seen that the experimental equilibrium path ends at the ultimate loading, when the tests were terminated due to some limitations of registration devices.

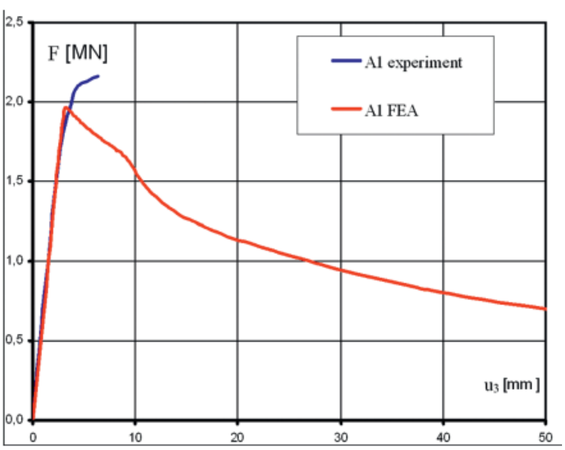

Figure 6.
Comparison of equilibrium paths for steel column A1

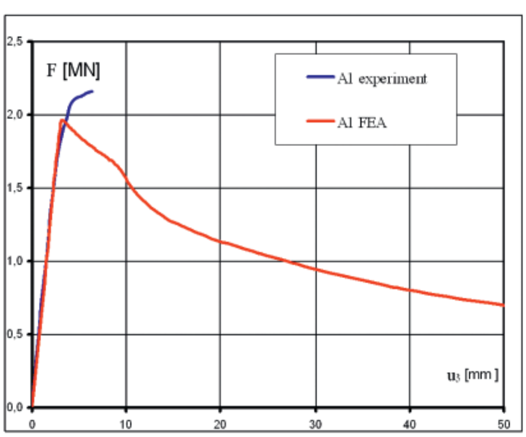

Figure 7.
Comparison of equilibrium paths for steel column A2

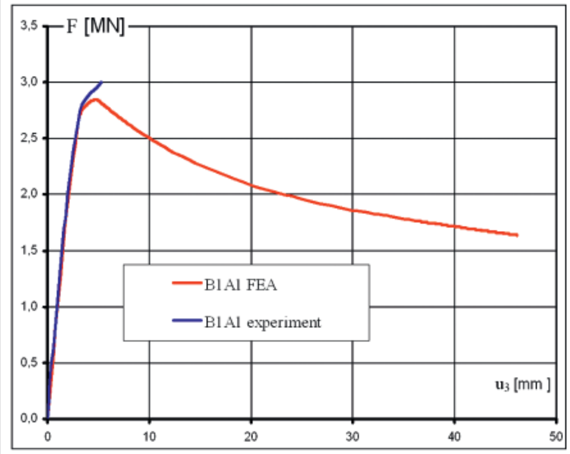

Figure 8.
Comparison of equilibrium paths for column B1A1

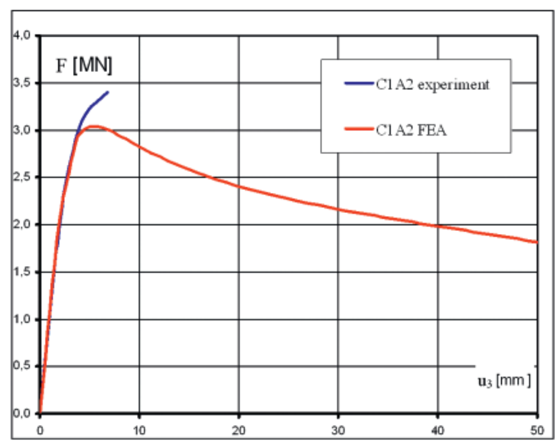

Figure 9.
Comparison of equilibrium paths for column C1A2

The diagrams in Figures 6, 7, 8, and 9 show pretty good correlations between numerical and experimental results. Also the failure modes received from the computer simulations, such as presented in Figures 10 and 11 are similar with the experiment. All the numerical results show lower estimation of the ultimate loading comparing to the experiment. One of the reasons of this discrepancy is the difference between the ideal boundary conditions applied in the FE and their actual representation in the columns during the experiments. In the FE models the loading is applied to the top plate through a frictionless hinge while in the experiments there were additional moments due to some friction.

Table 3 presents magnitudes of the ultimate loadings obtained experimentally and numerically.

Table 3.
Calculated and experimentally determined ultimate loading and efficiency

Column	Experiment F_E [kN]	FEA F_F [kN]	F_F/F_E
A1	2160	1965	0.91
A2	2140	1997	0.93
B1A1	3060	2728	0.89
C1A2	3333	3013	0.90

The results show that the numerical estimation is smaller for each column by about 10%. It should be noticed that the numerical results presented in Table 3 are for the loading eccentricity of 1mm. For larger eccentricities the difference between the forces is even larger (see for example Fig. 5). Figures 10 and 11 show comparison of deformations received from the computer simulations and the experiments for columns filled with concrete.

6. CONCLUSIONS

The paper presents comparison of experimental and numerical results for two-chord steel columns with different spacing of buttenings, filled with concrete strength class C20/25.

The comparison of the numerical results with the experimental data shows clear tendency for underestimation of the ultimate loading. The difference is 10% and it is almost the same for all tested columns. It seems that it is the effect of made assumptions especially regarding the boundary conditions.

Based on both, experimental and numerical analyses, it can be concluded that the fill with concrete C20/25 significantly increases load capacity of two-chord steel columns. For columns B1A1 the increment is 42% and for C1A2 – 55%. The numerical analysis also confirmed the increased load capacity for the columns with smaller buttenings spacing.

The same tendency was recognized by the authors of the study [2].

After analysis of all the strain diagrams and based on the FE analysis it can be stated that there is composite action at the connection between steel and concrete parts in considered cross-sections. At the same time the numerical analysis showed that the way of modeling the steel-concrete interaction (contact or merged nodes) does not affect the obtained magnitude of ultimate loading. More detailed study on the effect of bounding will be carried out in the next research.

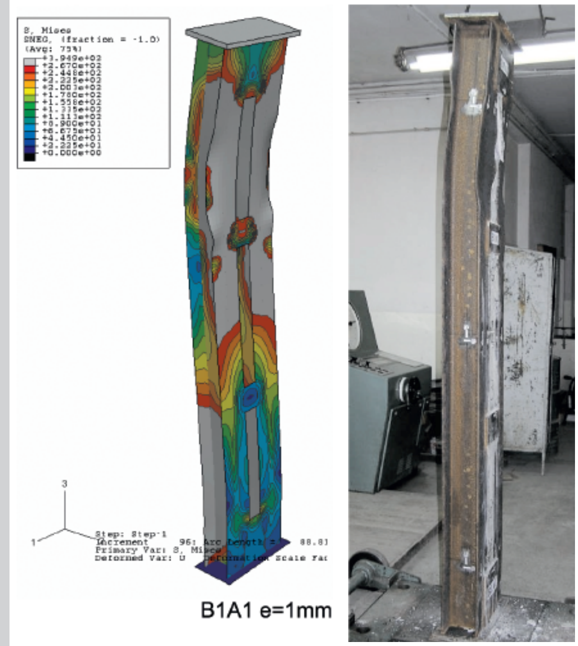

Figure 10.
Comparison of actual and simulated deformation for column B1A1

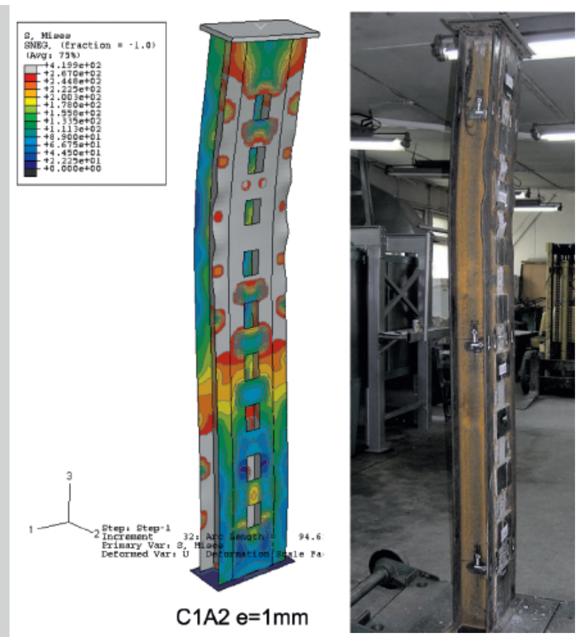

Figure 11.
Comparison of actual and simulated deformation for column C1A2

As it was mentioned before, the best correlation between numerical and experimental results was obtained for the assumed eccentricity of 1 mm, even though during the experiments the effort was made to apply loading axially. It allows to draw the conclusion that the design formulas developed currently for such columns, should take into account unexpected eccentricities, like it is in codes for concrete structures.

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