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Confinement of concrete in two-chord battened composite columns

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Abstract. This article provides an analysis of the complex character of stress distribution in concrete in stub columns consisting of two HE160A steel sections held together with batten plates and filled with concrete. In such columns, evaluating the effect of concrete confinement and determining the extent of this confinement constitute a substantially complex problem. The issue was considered in close correspondence to rectangular cross section tubular elements filled with concrete, concrete-encased columns, as well as to steel-concrete columns in which reinforcement bars are connected with shackles. In the analysis of concrete confinement in two-chord columns, elements of computational methods developed for different types of composite cross sections were adopted. The achieved analytical results were compared with calculations based on test results.

Keywords: composite structures; steel-concrete columns; two-chords battened columns; concrete confinement; load bearing capacity

1. Introduction

Reaching critical values of load-bearing capacity in stub steel-concrete columns is accompanied in most cases with cracking and crushing of concrete. Destruction of such kind was noticed in CFST columns made of tubes of relatively thin sidewalls, designed with respect to seismic threats. Similar observations were made in research conducted by the author of this paper on columns fully encased in concrete, as well as on two-chord battened columns filled with concrete. Hence, analysis of effective load-bearing capacity of concrete in composite columns appears to be an extremely important issue. Consequently, research has been done on possible methods of enhancing the load-bearing capacity of concrete subjected to axial compressive stress in conditions where the risk of lateral buckling was substantially reduced. In the 60ies of the previous century, Rüsch and Stöckl (1969) suggested a relationship between using helical reinforcement steel and subsequent improvement of confined concrete strength. Furthermore, Mander *et al*. (1988) provided a thorough analysis of a complex character of stress distribution in concrete, taking into account different types of steel-concrete columns. Similar research has been conducted in recent decades with regard to composite columns (El-Tawil and Deierlein 1999). In particular cases, clear parallels are noticeable in the methods of testing and theoretical solutions

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suggested for composite elements, should these be compared with methods and solutions applied for researching steel-concrete columns.

It is well known that restraining deformation of concrete is related to pressure exerted by lateral reinforcement, a steel tube or steel section. In the effect of confinement, compressive strength of concrete increases. That is conditional on numerous factors, which can be classified as the following:

- type of structure and type of confinement related to it (steel-concrete or composite column)
- type of composite cross section (ex. concrete-encased section or concrete filled tube)
- shape of cross section (circular, rectangular, polygonal)
- geometry of the cross section - dimensions and thickness of tube walls.

Experimental research as well as numerical calculations most often aim at assessing quantitative and qualitative influence of the above parameters on the effective concrete strength in different types of composite columns cross sections. Chen and Lin (2006) offered a comprehensive analysis of the effect the shape and layout of steel sections in a given cross section have on stress distribution in concrete and the resultant load capacity of columns. The confinement of concrete in CFST rectangular cross section columns was presented by Ge and Usami (1994) and Cai and He (2006). Additionally, research has been carried out on CFST rectangular cross sections, with a view to deriving benefits from the complex character of stress distribution in concrete. These can be achieved through seeking solutions at the stage of construction design (Hu *et al*. 2003). Susantha *et al*. (2001) proposed a method of predicting how the stress-strain relationship changes, taking into account the triaxial character of stress in concrete in tubular columns of various cross section shapes. The effect of concrete confinement has also been studied in CFST elliptical cross section columns (Yang *et al*. 2008), as well as in columns of the FRP type (Yuan *et al*. 2008). Analysing the problem of concrete confinement in columns consisting of steel tubes of rectangular cross sections often aims at assessing how to account for the confinement effect in calculating the load-bearing capacity of such columns so as to comply with current standards. For safety reasons, the effect that the complex character of stress distribution in concrete has on enhancement of load-bearing capacity in rectangular cross section tubular columns is most often disregarded, and accounted for only in circular cross sections. That problem was explored by many researchers. A method of computing load-bearing capacity for rectangular cross section CFST columns was suggested in (Huang *et al*. 2008), and the results were compared with computations according to American standards and the Chinese standard. In the same source the state of knowledge was established with particular regard to analytical and experimental research being conducted at present in numerous research centers concerning the issue of concrete confinement in composite columns.

2. Evaluation of concrete confinement in various types of composite columns

Destruction in stub columns is connected with crushing of concrete. Nonetheless, the mechanism of destruction depends directly on the behaviour of both materials constitutive of the composite cross section. Thus, the behaviour of concrete should be analyzed with regard to the interaction between steel and concrete as the load increases, and the type of composite section considered has prevailing influence on the relationship between the behaviour of concrete and the introduction of load. The interaction of structured steel and concrete is particularly noticeable in

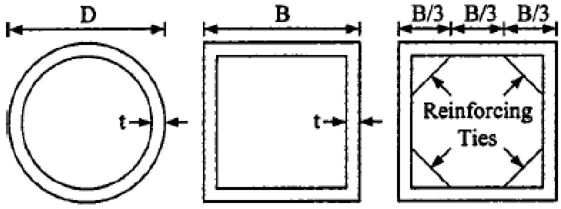


Fig. 1 Cross-sections of tested columns (Hu *et al*. 2003)

circular cross section columns of CFST type. In such columns the effect of concrete confinement is absent at early stages of load introduction. That results from the fact that at low values of load, the Poisson's coefficient for concrete is substantially lower than for steel. As the load increases, lateral deformation in concrete takes place, exerting pressure on the steel tube. At that stage of load introduction, steel undergoes stretching, and redistribution of stress from concrete to steel is observed during tests. In (Hu *et al*. 2003) results were presented of numerical analysis which aimed at assessing the effect of adopting particular solutions in design of CFT columns on enhancement of concrete strength with regard to the complex character of stress distribution. Three types of cross sections were examined (Fig. 1), with different dimensions of the cross sections and thickness of the walls t.

The conducted analysis confirmed that using circular cross section tubes benefits the state of stress distribution in concrete. However, that concerns the cross sections in which the relation of diameter to thickness of the plate does not exceed definite values, namely, when $D/t < 40$. In columns of a square cross section, the effect of confinement is reduced when compared with that in circular tubes, particularly ones with thinner walls, i.e., when $B/t > 30$. In those, there is a potential danger of local buckling of the steel plate. The suggested remedy in such cases would be introducing additional reinforcement to improve stiffness of the element.

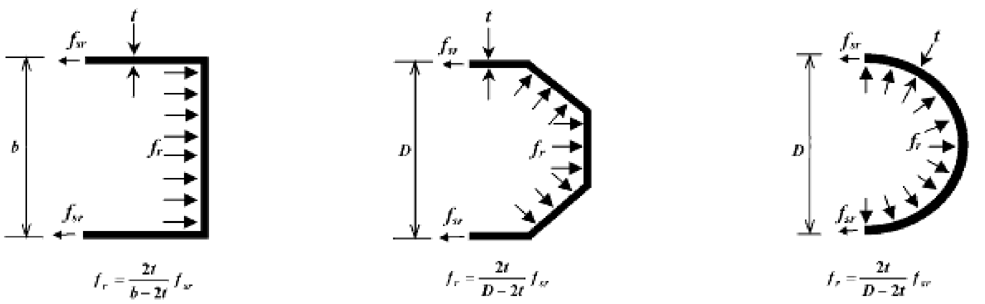
In the 60ies, Rüsch and Stöckl (1969), proposed a relationship between the helical reinforcement steel and the corresponding increase of confined concrete strength fcc. The relationship was expressed with the following equation

$$f_{cc} = f_c + k \cdot p \tag{1}$$

where: f_c is the strength of concrete in the uniaxial distribution of stress, k is a factor dependent on the internal friction angle of concrete, and p – lateral strain on the interface of the concrete and the circular hoop reinforcement.

Eq. (1) presented resulting from the triaxial character of stress in concrete, is also applicable with regard to circular section composite columns. The provided value of maximum radial pressure depends principally on boundary values of ductility of steel in the tube as well as on the tube dimensions. Generally, the value of the pressure is dependent on geometrical properties of a cross section (Fig. 2). The interrelations required for calculating pressure in box section and octagonal section tubes, which are given in Fig. 3, have been modified according to formulas standing for circular sections (Susantha *et al*. 2001).

Following the results achieved in their analysis, Susantha *et al*. (2001) suggested a method of calculating the maximum value of lateral pressure f_{rp}^* in steel columns filled with concrete, with respect to circular, box and octagonal cross sections. The authors stipulate that the provided



(a) Box section (b) Octagonal section (c) Circular section

Fig. 2 Lateral pressure patterns in various sections (Susantha *et al.* 2001)

formulas can be applied only in specific conditions complying with these adopted in their study. Regarding box section columns, the assumed computational value of strength for unconfined concrete was from 10 MPa to 50 MPa. A recommendation was made for the dimensions of the analysed cross sections to be duly selected in order that the effect of local buckling, which occurs when axial load is introduced, could not be ignored. The final assumption was made based on the interrelation provided by Ge and Usami (1994).

$$\frac{f_b}{f_y} = \frac{1{,}2}{R} - \frac{0{,}3}{R^2} \leq 1{,}0 \tag{2}$$

where f_b is the value of critical compression stress leading to local buckling, and R denotes the plate width-to-thickness ratio parameter relating to geometrical and physical properties of the cross section, which is given by

$$R = \frac{b}{t}\sqrt{\frac{12\left(1-\nu^2\right)}{4\pi^2}}\sqrt{\frac{f_y}{E_s}} \tag{3}$$

Ultimately, the interrelation proposed in (Susantha 2001) for computation of the maximum values of lateral pressure in box section columns is expressed by the following equation

$$f_{rp}^{*} = -6{,}5R\frac{\left(f_c\right)^{1,46}}{f_y} + 0{,}12\left(f_c\right)^{1,03} \tag{4}$$

The above equations show that the effect of local buckling can be ignored when the value of R is not higher than 0,85. In (Susantha *et al.* 2001) the value of R between 0,35 and 0,82 was assumed. Following the authors, calculations done for thick-walled tubes yield similarly good results, despite the fact that such tubes are rarely used in construction of steel-concrete columns. Furthermore, research shows that the increase of concrete strength which is due to confinement is much less prominent in thin-walled tubes than in tubes with thicker walls. The issue of the influence of the confinement of concrete on concrete strength in columns consisting of rectangular section tubes filled with concrete was further pursued by Yan-sheen Huang *et al.* (2008). Consequently, a method of computing load-bearing capacity of such columns was proposed, based on methods of assessment suggested by Mender *et al*. (1988), to evaluate how strength increases in steel-concrete columns. In (Huang *et al*. 2008), an assumption was made that the mechanism of inducing the concrete confinement effect in a rectangular section tube is comparable to one which occurs in steel-concrete columns with lateral reinforcement. The difference lies in the results produced by such confinement, which remain in a direct connection with load-bearing capacity of concrete subjected to triaxial stress. While depending on a well-established relationship presented in (Mander *et al*. 1988), the authors in (Huang *et al*. 2008) propose a method of determining the effective lateral pressure resulting from confinement, using the following formulas

$$f_{l1} = k_e \cdot f'_{l1} \qquad f_{l2} = k_e \cdot f'_{l2} \tag{5}$$

where f'_{l1} and f'_{l2} stand for the pressure exerted on the narrow and broad faces of the tube, respectively (Fig. 3), whereas f_{l1} and f_{l2} are the corresponding effective pressure values, taking account of the concrete confinement.

The relationship between pressure and circumferential stress in a tube is expressed with the following equations

$$f'_{l1}(B-2t) - 2f_{sr1} \cdot t = 0 \qquad f'_{l2}(D-2t) - 2f_{sr2} \cdot t = 0 \tag{6}$$

The value of the confinement effective coefficient ke is directly linked to lateral (cross-sectional) ke1 and longitudinal ke2 components, according to the following formula

$$k_e = k_{e1} \cdot k_{e2} \tag{7}$$

With regard to tubes, an assumption should be allowed that the value of the coefficient ke2 equals 1, whereas the lateral component is suggested as

$$k_{e1} = \frac{A_{e1}}{A_{cc1}} \tag{8}$$

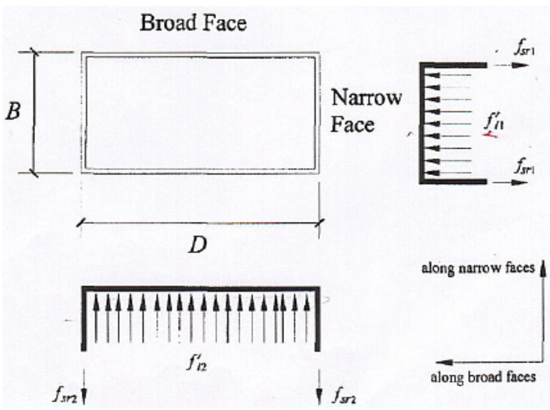


Fig. 3 Lateral confinement of concrete core (Huang *et al*. 2008)

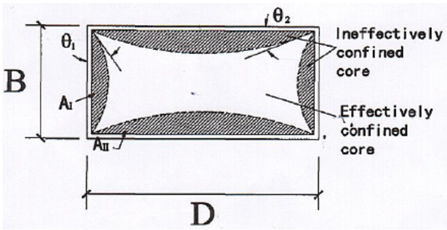


Fig. 4 Effectively confined core for rectangular CFT columns (Huang *et al*. 2008)

where Ae1 is the cross-sectional confined concrete and Acc1 is the entire area of the concrete core (Fig. 4).

The geometrical properties of a rectangular cross section of a tube filled with concrete underlie the conclusion that the confinement coefficient k_{e1} can be expressed with the following formula

$$k_{e1} = 1 - \frac{(B-2t)\tan\theta_1}{3(D-2t)} - \frac{(D-2t)\tan\theta_2}{3(B-2t)} \tag{9}$$

In (Huang *et al*. 2008) the authors additionally propose equations for calculating the effective tangent angles θ, the values of which are related to the dimensions of a cross section, as well as to the ductility limit for the steel tube and the plate thickness. For calculating values of circumferential stress f_{sr} in a tube and the pressure exerted on the element in the effect of axial load, in (Huang *et al*. 2008), Huber-Mises-Hencky's hypothesis was applied

$$f_{a1}^2 - f_{a1} f_{sr1} + f_{sr1}^2 = f_y^2 \tag{10}$$

where f_{a1} is the value of stress in the steel of the broad face of the tube (see Fig. 3), equivalent to the value of stress accompanying local buckling (2).

Ultimately, when the value of the width-to thickness ratio R for a cross section does not exceed 0,85, in (Huang *et al*. 2008), the authors propose the values of stress to be adopted in computations as

$$\begin{aligned} f_{a1} &= 0{,}89 f_y \\ f_{sr1} &= -0{,}19 f_y \end{aligned} \tag{11}$$

The values of stress in the narrow face of a steel tube are computed in an analogous manner.

The relationships proposed in (Huang *et al*. 2008) are applicable for determining the strength of concrete filling a rectangular section steel tube of an axially loaded column. For that purpose, the authors in (Huang *et al*. 2008) refer to the relation between different stresses on the octahedral plane. The formula suggested for computing load capacity of columns finally took the following form

$$N_u = A_{s1} f_{a1} + A_{s2} f_{a2} + A_c f_{cc}' \tag{12}$$

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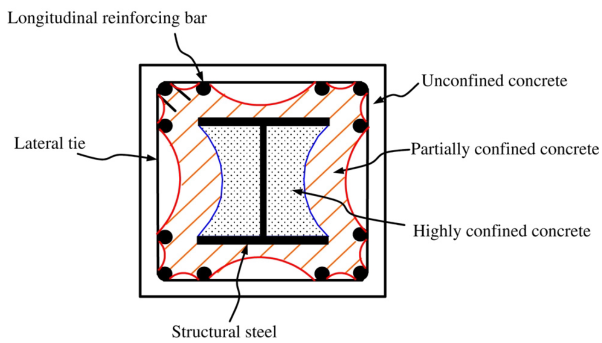

Fig. 5 Materials in a concrete encased steel composite column (Chen and Lin 2006)

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The shape and dimensions of the regions of specifically confined concrete are influenced by numerous factors. Beside distribution of lateral reinforcement, the type and dimensions of structural steel are equally prominent.

The immediate aim of the research presented in (Chen and Lin 2006) was to provide an analysis of the degree of concrete confinement in particular regions of various cross sections. Experimental investigations and theoretical computations resulted in defining the values for particular confinement factors, thus providing a method for evaluating the enhancement of concrete strength with regard to the specified part of the cross section. Accordingly, the strength of confined concrete in the considered types of concrete-encased columns can be determined by applying the following equations:

- for partly confined concrete

$$f'_{cc} = K_p f'_{co} \tag{13}$$

- for highly confined concrete

$$f'_{cc} = K_h f'_{co} \tag{14}$$

where *Kp* and *Kh* are the values of confinement factors for partly confined and highly confined concrete, respectively. In a theoretical calculation model, the load-bearing capacity of the examined columns is constituted by the total sum of load-bearing capacity of the particular components of a cross section, i.e., structural steel, longitudinal reinforcement, as well as unconfined, partly confined and highly confined concrete. The conducted analysis proved that increasing load would accordingly diminish the function of structural steel in transmission of the effective force, while at a certain level of strain the highly confined concrete parts of the section take over the greater part of effective load.

3. Own experimental research carried out on two-chord battened composite columns

Experimental research was conducted on regions of concrete confinement in two-chord columns loaded axially, as in Fig. 6.

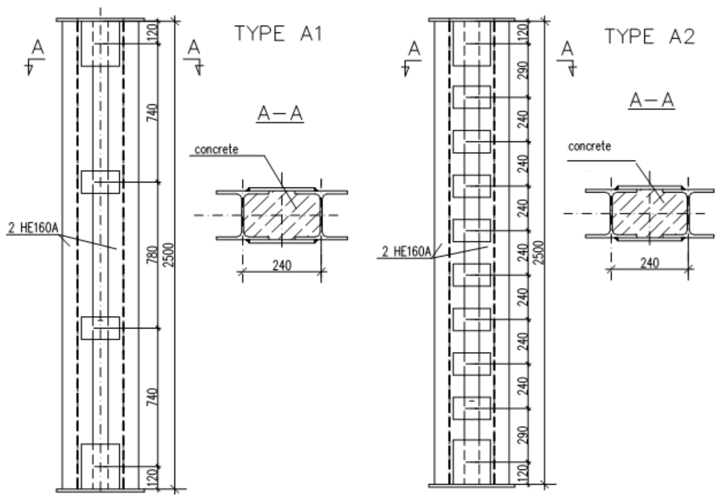


Fig. 6 The tested model diagram for columns A1, B1A1, B1A1/P and A2, C1A2

The cross sections of the elements consist of two H-shaped S235 grade steel sections HE160A and the spaces between flanges and webs of beams are filled with concrete (Żółtowski *et al*. 2006). To assess load-bearing capacity as well as deformability of the considered columns, experimental research was carried out on 41 elements in natural scale (see Fig. 6). In the process of experimental analysis, the variable parameters adopted were the strength of the concrete contained between the H-shaped steel sections and the spacing between batten plates connecting the sections. Two spacing distances were considered: of 240 mm and 780 mm. For a broader scope of the analysis, additional three elements were produced and examined, in which the H-shaped steel sections were connected not with battens but with continuous sheets of steel on the whole length of the element. In two series of tests additional reinforcement consisting of four 12 mm longitudinal bars and 6 mm diameter shackles was placed in the examined columns. Furthermore, tests were carried out on columns filled with concrete containing steel fibres. Two steel columns, specially designed with a view to conducting comparative research between these and the steel-concrete elements, were not filled with concrete. The columns were tested in three cycles with regard to different loading patterns. Effective force was applied to the entire cross section, exclusively to the steel sections, or solely to the concrete core. The tested elements were subjected to compressive axial force in a hydraulic press (Fig. 7). While increasing load, measurements were taken of the lateral and longitudinal deformations, as well as of the displacement of the column. After maximum values of compressive force were registered, loading was maintained in the press so that the mode of destruction/form of damage which took place in the element could be observed (Szmigiera *et al*. 2010).

The research presented herein was carried out on axially loaded elements classified as stub. In columns of that type the influence of particular variable parameters on the load-bearing capacity and deformability of elements subjected to testing is immediately obvious. Research results presented in specialist literature indicate that in slender columns factors such as the strength of concrete or the degree of concrete confinement have a lesser influence on load-bearing capacity of an element. Besides, engineering practice shows that axially loaded steel columns consisting of two T-shaped sections have better load-bearing capacity properties than equivalent columns made

Fig. 7 A test stand for two-chord columns with batten spacing of: (a) 780 mm; (b) 240 mm

of rectangular section tubes, which is clearly due to more favourable geometrical properties of the former. Consequently, two-chord steel columns consisting of HEA, HEB or HEM sections are widely employed in designing structures where high load-bearing capacity is a prerequisite. To take the matter further, the next step to consider would be filling steel columns with concrete to profit from composite structures specific properties of load bearing. Columns of that type can be used in highly-loaded structures, e.g., in the floor method of constructing the underground parts of buildings in high-density urban areas (Mitew-Czajewska *et al*. 2012), or in the up & down method. The apparent advantage of the above solution is the possibility of enhancing load capacity by filling with concrete two-chords steel columns which are constituent parts of existing structures. Bearing in mind all the above, carrying out the research was essential, because most standards regulations concerning steel-concrete composite elements lack design recommendations for compression elements consisting of two sections.

Table 1 Properties of the selected elements, including tests results

Notation of series	Distance of battens [mm]	Average value of compressive strength of the unconfined concrete [MPa]	Average value of yield strength of steel [MPa]	Failure load [kN]
A1	780	-	309	2160
A2	240	-		2140
B1A1	780	33,3		3060
C1A2	240			3333
B1A1/P	780	54,4	269	3600
B2A1	780	79,5		4333
C2A2	240			4377
D1A3	Continuous steel sheets			4663

4. The concrete confinement effect in the examined two-chord steel-concrete columns

Analytical research on the character of stress in the concrete encased between the H-shaped sections was conducted for the selected types of columns, the material properties of which as well as results achieved for destructive forces are tabulated in Table 1. For comparison, results regarding the tested steel elements A1 and A2 were provided. Three elements of the same parameters were considered in each series of tests.

The problem of evaluating the degree of concrete confinement in two-chord columns is highly complex due to the fact that the regarded columns consist of two H-shaped sections which are tied with battens. Hence, it should be possible to investigate the issue by analogy with concrete-filled tubes, as well as with concrete-encased columns, or even steel-concrete columns in which reinforcement bars are connected with shackles. Therefore, elements of different methods of calculation used for particular types of columns were employed in the analytical research. The results were compared with calculations done on the grounds of measurements of lateral deformations registered in the tests.

In the first phase of the analysis, the methods described in Section 2 were availed of, concerning the confinement of concrete in rectangular CFST columns. Following the equations proposed by Susantha *et al*. (2001) and Yan-sheng Huang *et al*. (2008), the values of circumferential stress and lateral pressure were established in the two-cord columns used in the experiments. The circumferential stress as well as the lateral pressure acting on the webs of the H-shaped sections and on the battens of the tested columns were determined by taking the values of lateral deformation in steel measured in the process of testing. Fig. 8 presents a model of a steel section adopted to analyse the confinement of concrete on the basis of the achieved test results. For the purpose of making the calculations clearer, the entire cross section of the examined columns was divided into halves along the axis parallel to the webs of the H-shaped steel beams. The calculations were made with a view to defining the values of the lateral pressure acting on the sides of the web at the maximum values of load. Similarly to what is found in (Susantha *et al*. 2001) and in (Huang *et al*. 2008), the assumed model provides for independent interactions with regard to the web sides and to the battens.

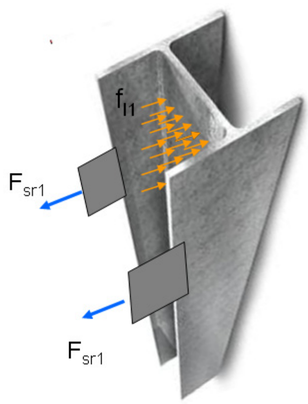


Fig. 8 A section model adopted for calculating concrete confinement-induced lateral pressure on the steel web of the H-section; Fsr1 – circumferential forces in the areas of battens, calculated from values of lateral strain registered in tests

Table 2 Comparative results of circumferential stress and maximum lateral pressure values

Notation of series	Width-to-thickness ratio parameter R	Circumferential stress in steel [MPa]			Lateral pressure on the webs [MPa]			k_e^{test}
		acc. to *S	acc. to *Y	own tests	acc. to *S	acc. to *Y	own tests	
1	2	3	4	5	6	7	8	9
B1A1	0,38	21,4	60,2	56,6	2,87	1,86	1,3	0,17
C1A2				52,0			3,85	0,55
B1A1/P	0,34	35,7	55,5	65,1	4,78	1,71	1,48	0,17
B2A1	0,32	43,5	51,3	91,4	5,83	1,59	2,08	0,17
C2A2				54,7			4,05	0,55
D1A3	0,29	46,3	51,3	197	6,2	6,9	6,0	1,00

*S - (Susantha *et al*. 2001)
*Y - (Huang *et al*. 2008)

Table 2 presents comparative results of computations done according to (Susantha *et al*. 2001), (Huang *et al*. 2008) and of own research conducted on two-chord steel-concrete columns. Values reported in Cols. 3, 4 and 6, 7 were obtained with computations done following relations provided in (Susantha *et al*. 2001) and (Huang *et al*. 2008) (as in Section 2), taking account of the mechanical properties of materials and the geometrical properties of cross sections of the tested columns (see Tables 1 and 2, Col. 2). Besides, values of stress and pressure achieved in the result of tests performed by the author of this analysis were reported in Cols. 5 and 8. The values of pressure are based on the values of circumferential stress calculated with Hooke's Law using the measurements of lateral deformation in steel taken during the tests. Pressure acting on the web sides was calculated with the data obtained in that way, and account was taken of the batten spacing in the regarded type of column. The values provided in Table 2 are the average values calculated with the values achieved for 3 columns in each test cycle/series (calculations were made for 18 columns altogether).

The achieved results show that the method proposed in (Huang *et al*. 2008) enables researchers to evaluate the effect of concrete confinement in the examined two-chord columns more precisely, and to better match their assessments with the results obtained in the tests, than the method developed in (Susantha *et al*. 2001). That concerns particularly columns with wider spacing between battens. Additionally, for the columns filled with concrete of lower strength, the achieved values of circumferential stress corresponded closely to the calculated values of such stress based on lateral deformation. Consequently, it can be assumed that in such columns the values of circumferential stress in steel practically depend on boundary values of the ductility of the steel plate. Hence, adopting in computations the relation as in $fsr = 0{,}19\, fy$ produces reliable results, whereas considerable differences in the results of circumferential stress and particularly of lateral pressure values are observed in computations done for the columns filled with high strength concrete, with narrow spacing between battens. In that case, calculations based solely on the mechanical properties of the steel plate and the geometrical properties of the section do not yield dependable results.

The analysis of the test results based on the registered measurements of deformation in steel provides that in the columns with wider batten spacing the increase of lateral pressure values, directly linked to the increase in strength, amounted to 60%. A parallel increase in the columns with narrower spacing was 5%. The conclusion is that in the examined two-chord columns, the concrete strength considerably affects the lateral pressure values registered for the elements with wider batten spacing. As the load-bearing capacity of concrete increases, the effect is proportionally enhanced. No parallel relationship was revealed when analysing columns with narrow batten spacing.

In the opinion of the author, the method assumed for calculating the confinement coefficient ke1 (see Eq. (9)) applied in (Huang *et al*. 2008) to compute the value of lateral pressure is of key importance for achieving reliable results.

With that equation, the factor for the confinement of concrete in the examined two-chord columns was determined, assuming the most disadvantageous value of the angle $\theta 1$ = $\theta 2$ = 450 (see Section 2, Fig. 4). The value of the factor in calculations done for all the analysed columns was determined as *ke* = 0,23. However, considering results achieved in the tests, different values of that coefficient were established, dependent on the spacing between battens (see Table 2, Col. 9). The achieved results suggest good conformity of the coefficient ke between theoretical and experimental results for columns of uniform batten spacing (0,17 – for narrow batten spacing and 0,55 – for wide batten spacing), irrespective of the concrete strength.

Analysing the computational results achieved according to the relations suggested in (Susantha *et al*. 2001), it may be claimed that in the equations employed for determining the value of pressure, too much importance is placed on the concrete strength when the concrete is confined by a steel tube. The author asserts that, regarding substantial discrepancies between numerical calculations and experimental results, the method proposed in (Susantha *et al*. 2001) should not be applied for the purpose of valuation of the concrete confinement effect in two-chord battened steel-concrete columns.

A good coincidence in results, however, was obtained for D1A3 type columns, in which continuous sheets of steel were used instead of battens. In that case, their cross section directly corresponds to that of rectangular concrete-filled tubular columns, with regard to which methods of calculation adopted for the purpose of this analysis were developed. Simultaneously, the results of calculations based on the test results for lateral pressure in D1A3 type columns indicate that for such columns the confinement coefficient, adopted for calculations done as proposed in (Huang *et al*. 2008), should have the value of *ke* = 1. That leads to a conclusion that in the test columns made of steel sections tied with continuous sheets of steel instead of battens, the concrete core is confined on the entire cross section (cf. Eq. (9)), which puts in question validity of the assumed distribution of confinement regions presented in Fig. 4, as proposed by the researchers in (Huang *et al*. 2008). According to the author, it may be attributed to higher stiffness of the flanges of the H-sections, onto which rigid steel sheets were weld, in comparison to the plate in standardized steel tubes.

The above considerations referred to evaluating lateral pressure acting on the walls of steel sections at destructive load. In Table 3 the estimates of concrete strength values were given for comparison, with account taken of the confinement effect at destructive load values in the examined two-cord columns consisting of two H-shaped steel sections connected with battens. The strength of confined concrete in the various types of columns used in the tests was assessed following the equation provided in (1). The concrete strength in a particular column was considered to be the starting value in the computations. It was determined in previously conducted research which involved examination of samples taken from bore-holes (see Table 3, Col. 2 in brackets). Col. 6 of Table 3 gives a percentage increase in the concrete compressive strength based on the conducted research and referred to the value of that strength in a column.

Table 3 Comparison of the confined concrete strength in the examined columns

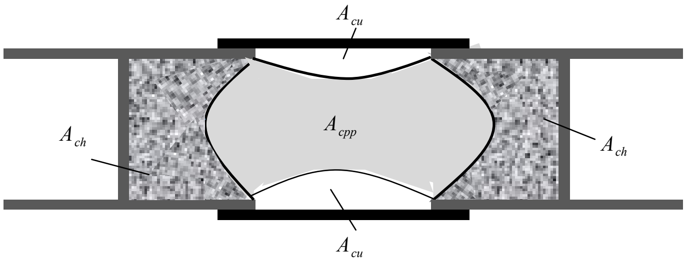
Notation of series	Average value of compressive strength of the unconfined concrete (in column) [MPa]	Value of compressive strength of the confined concrete [MPa]			Δ [%]
		acc. to *S	acc. to *Y	from own tests	
1	**2**	**3**	**4**	**5**	**6**
B1A1	33,3 (29,97)	41,45	37,41	35,17	17
C1A2				45,37	51
B1A1/P	54,4 (48,96)	68,08	55,8	54,88	12
B2A1	79,5 (71,55)	94,87	77,91	79,95	12
C2A2				87,75	23
D1A3	79,5 (71,55)	96,35	99,15	95,55	34

*S - (Susantha *et al*. 2001)
*Y - (Huang *et al*. 2008)

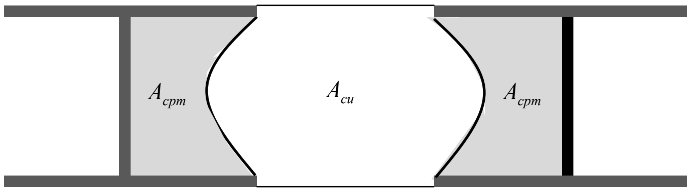
The highest increase in the strength in relation to the confinement of concrete was noted in the columns with narrow spacing between battens (C1A2 and C2A2). Comparing the results concerning the increase in concrete strength inevitably results in a conclusion that for two-chord columns with narrow spacing the method proposed in (Susantha *et al*. 2001) is more accurate than the formulas suggested by Yan-sheng Huang at al. in (Huang *et al*. 2008). Discrepancies observed between numerical calculations and experimental results amount to mere 8%. However, for columns with wider batten spacing the strength values calculated with equations provided in (Huang *et al*. 2008) fall closer to the test results. The determined values of the concrete strength differ only by 2% to 6%.

The reported discrepancies between results obtained in the procedure adopted for assessing the values of pressure exerted on the webs of the H-shaped sections may be associated with deficiencies in endorsing the methods used for the analysis, without due regard to the design of the examined columns. While both of the methods were developed for the purpose of determining the pressure in rectangular tubes continuous along the whole length of the element, the analysed battened columns do not meet the standards for being prismatic.

Following the conducted research it may be assumed that for the examined two-chord elements, numerical analysis of a selected cross section is commendable, as it is in the case of columns comprising two steel sections and fully encased in concrete. A method based on the analysis provided by (Chen and Lin 2006), concerning individual composite columns fully encased in concrete was employed and subsequently modified in this study (see Section 2) to calculate the load-bearing capacity of two-chord columns with respect to concrete confinement. It was presumed that in the test columns the cross sectional area should be divided into discrete regions of different concrete confinement parameters depending on the considered region but also on the location of the cross section along the length of the compression element. Assumptions regarding distribution of the concrete confinement regions in the analyzed two-chord columns are illustrated in Fig. 9. Two different cross sections were considered. In the first one, which is restricted by battens, i.e., in the part of column embraced by battens, enhancement of concrete strength is presumed in the regions corresponding to similar regions discretized in concrete-encased columns (Fig. 9(a)). The other cross section represents the concrete confinement regions in the parts of test columns located between battens (Fig. 9(b)). It was assumed that on that cross section the concrete



(a)



(b)

Fig. 9 Regions for predicted concrete confinement in the analysed steel-concrete columns: (a) cross section in the column parts embraced by battens; (b) cross section between battens; *Acpp* – partly confined concrete on the cross section in the column parts embraced by battens, *Acpm* - partly confined concrete on the cross section between battens, *Ach* – cross section of highly confined concrete, *Acu* – cross section of unconfined concrete

is only partly confined in the regions between the web and adjacent flanges of an H-shaped section.

The concrete confinement regions presented in Fig. 9 were identified through analyses. That eventually allowed for providing a technical model for the examined two-chord columns. Within that model the results obtained in computations exhibit good agreement with the test results.

Computations of the load-bearing capacity of the analysed columns were based on the well-known formula for the axially loaded stocky composite columns

$$N_{cc} = N_s + N_c \tag{15}$$

N_s is the load capacity of steel part of cross-section

$$N_s = 2 \cdot A_s \cdot \sigma_s \tag{16}$$

where:

A_s – area of H-shape cross-section,

σ_s – stresses in constructional steel

N_c is the load capacity of concrete filling the columns, with regard to degree of confinement, performed following the formula suggested by the author

$$N_c = s_p\left[\left(K_p \cdot A_{cpp} + K_h \cdot A_{ch}\right)\right] \cdot \sigma_c + s_m\left[\left(K_p \cdot A_{cpm} + A_{cu}\right)\right] \cdot \sigma_c \tag{17}$$

Cross sectional areas of concrete: highly confined Ach, partly confined Acpp and Acpm, as well as unconfined Acu, respectively, were provided in Fig. 9.

In Eq. (17), the ratios s_p and s_m allowed the author to account weighted average of concrete part of tested columns (for the parts of columns represented by the cross sections which are restricted by battens - s_p and those which are not battened - s_m, related to the whole length of the column - see Fig. 6). The values of the ratios hence depend on the batten spacing in the analysed columns and have been determined as follows:

- in columns with batten spacing of 780 mm (column types B1A1 and B2A1):
  s_p = 0,288, s_m = 0,712,
- in columns with batten spacing of 240 mm (column types C1A2 and C2A2):
  s_p = 0,576, s_m = 0,424

The values for the ratio s_p were established as a relation of the total length of battens (measured along the length of the steel section) to the total length of the element. The values for the ratio s_m are defined by the relation comparing the length of the column between battens to its total length.

For a more accurate analysis of the researched elements, the real values of stress in steel and in concrete were introduced into Eqs. (16) and (17) – σ_s and σ_c, respectively. They were evaluated in the analysis of the stress-strain curve obtained through the measurements of strain taken in the experiments carried out on two-cord test columns. It was observed that at the values of load corresponding to destructive force, in all the cases the achieved values of stress in concrete and in steel in the examined columns were slightly lower than the values of strength previously identified in the research. Hence, to achieve more accurate results, the following values of stress in steel and concrete were introduced into Eqs. (16) and (17), namely

$$\sigma_s = p_s \cdot f_y \tag{18}$$

$$\sigma_c = p_c \cdot f_{cu} \tag{19}$$

where p_s and p_c denote levels of stress at which strength gain is observed with regard to the boundary of the ductility of steel and the strength of unconfined concrete based on results of laboratory tests of material samples. Confinement coefficients were assumed following (Chen and Lin 2006), as *Kp* = 1,08 and *Kh* = 1,23. The adopted values of those coefficients account both for the dimensions of steel sections and for the batten spacing distances. Results of calculations based on the above relations were listed in Table 4.

Table 4 The effect of concrete confinement on enhancement of load-bearing capacity in test two-chord columns

Notation of series	Steel p_s	Concrete p_c	Failure load from tests N_u^{test}[kN]	N_{cc} [kN]	N_{cu} * [kN]	N_u^{test}/N_{cc}	N_u^{test}/N_{cc}	N_{cc} / N_{cu}
1	3	4	5	6	7	8	9	10
B1A1	0,91	0,91	3060	3063	3009	**1,00**	1,02	1,02
C1A2	0,97	0,9	3333	3270	3188	**1,02**	1,05	1,03
B2A1	0,96	1,0	4333	4159	4021	**1,04**	1,08	1,03
C2A2	0,98	0,99	4377	4503	4300	**1,03**	1,08	1,04

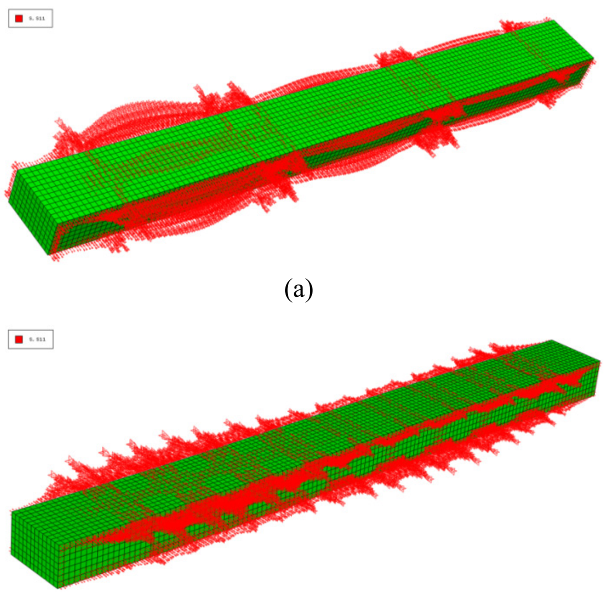
* *Ncu* is the value of load-bearing capacity of columns, concrete confinement disregarded

The load-bearing capacity analysis conducted for two-cord steel-concrete columns with respect to concrete confinement, using Eqs. (15)-(19), demonstrated very good agreement between the computational results and the values of destructive force obtained in the conducted tests (see Table 4, Col. 8). In most cases the experimental values of destructive force were even slightly greater than the computed values taking account of the concrete confinement.

It was established on the basis of the achieved results that in columns produced of the concrete of a higher strength class, the effect of confinement on the load-bearing capacity of the columns is more conspicuous. Simultaneously, in those columns the degree of confinement is not related to batten spacing. That may support previous results obtained in experimental research which suggest that in columns filled with concrete of higher strength characteristics, the values of destructive force are not related to the spacing distances between battens (see Table 1). The relation is observed, though, in columns of a lower concrete strength class. A conclusion may be drawn from the data provided in Table 4 that for the tested columns the estimated maximum strength increase due to concrete confinement was 8%.

5. Numerical research

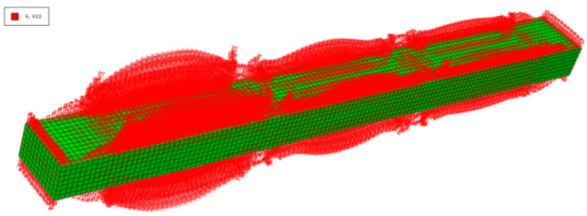
In order to complement the research and verify the adopted model of the presumed distribution of concrete confinement regions in the two-chord test columns the numerical analysis in Abaqus/ Standard (2006a, b) was conducted for the elements filled with concrete of a lower strength class (of types B1A1 and C1A2).



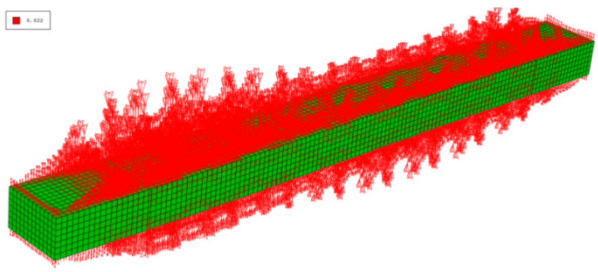
(a)

(b)

Fig. 10 Distribution of compressive stress in the concrete and intensity of stress on the concrete surfaces in perpendicular direction to the webs of sections, plotted for column types: (a) B1A1, with batten spacing of 780 mm; (b) C1A2, with batten spacing of 240 mm



(a)



(b)

Fig. 11 Distribution of compressive stress in the concrete and the intensity of stress on the interface in perpendicular direction to the battens, plotted for column types: (a) B1A1 with batten spacing of 780 mm; (b) C1A2, with batten spacing of 240 mm

Steel elements of the investigated column (2 × H-beam HE160A and continuous connecting sheet) were modelled using 4-noded shell elements with appropriate thicknesses. An elastic – plastic material model with von Mises yield criterion and strain hardening was used with the following parameters: density $\rho = 7.85 \cdot 10^{-9}$ t/mm^3, modulus of elasticity E = 205 GPa, Poisson ratio υ = 0.3, and strain hardening parameters according to the steel tensile tests conducted experimentally. Concrete was modelled using 8-noded brick elements with Concrete Damaged Plasticity material assumed. The default parameters were defined accordingly to the ABAQUS manual and tests of concrete samples used in the experiment. Load was defined as a prescribed motion of a top rigid plate of the column. Further information about the numerical model can be found in (Kwasniewski *et al*. 2011).

Figs. 10 and 11 show the distribution of compressive stress in the concrete and the intensity of stress on the interface between the concrete and the steel surfaces of the sections or battens. The plots were developed on the assumption that the load was applied on the left hand side of the plot.

Figs. 10 and 11 illustrate that the greatest compressive stress on the concrete surface occurs in the areas of the inner corners between the webs and the flanges. Irrespective of the batten spacing, higher values of the stress were recorded in the regions near those columns ends to which load was applied.

The battens or the batten spacing have no influence on the increase of compressive stress in concrete in perpendicular direction to the battens (Fig. 11). However, they induce the increase of the values of stress when it is perpendicular to the webs. It may be claimed hence that in the researched columns the highest pressure occurs in the inner corners between the webs and flanges of the H-sections, in the battened parts of sections, in regions close to the area where load was introduced. The numerical analysis results confirm that the concrete confinement effect is present in the researched two-chord columns.

6. Conclusions

The conducted analysis shows that the state of stress in concrete in the researched two-chord columns follows a pattern similar to the one present in concrete-encased beams of a single web I-section or of double open channel form, rather than that which is found in CFST column of rectangular cross sections. The computational results for two-chord columns presented in Table 4 were established to be in very good agreement with the results previously obtained from the experiments. Hence, the method developed by the author through modifying the solutions proposed in (Rüsch and Stöckl 1069), can be employed with good success for evaluating load-bearing capacity with regard to the concrete confinement effect in two chord battened columns. In the opinion of the author, the method proposed herein and applied to columns comprising two H-shaped sections can be used in researching similar elements, e.g., columns consisting of steel channel sections with flanges directed inward on both sides of the cross section.

Keeping in mind that in real conditions boundaries between the regions of different concrete confinement characteristics are fuzzy, the state of stress in concrete should be defined by a continuous function. Consequently, the distribution scheme for the particular regions of confinement given in Fig. 9 should reasonably be regarded as a computational model. The same stands for all models depicting the distribution of stress in concrete in composite columns in a whole variety of types other than those researched in this study.

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