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RESEARCH ARTICLE

Experimental Investigation of Thin-walled Column-end Joints Encased in Ultra-lightweight Concrete

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Abstract
A novel thin-walled structural system is under development which aims to utilise the beneficial effect of continuous bracing achieved by encasing CFS elements in ultra-lightweight-concrete. This paper deals with the experimental analysis of column-end joints. Altogether six types of end connection and four cross-sections were tested by 18 unbraced and 48 braced specimens. Based on the results four main zones of the load-displacement curves were defined which represent the behaviour of the different types. The failure modes and ultimate loads were identified and were found to be in close correlation with the results of previous tests on compression members. The experimental test helped to identify the main parameters of thin-walled column-end joints which affect the behaviour of the structural detail.

Keywords
encased stub column, column-end, connection, braced elements, thin-walled, experiment

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1 Introduction

In the field of structural engineering the design of cost efficient structures is highly important. The intention of designing efficient structures led the way to develop steel structures made of cold-formed steel (CFS) elements. Since the governing failure mode of CFS thin-walled elements is buckling (either local, distortional or global), intensive experimental research was carried out on the effect of secondary structures, which can significantly increase the standardised ultimate strength of steel elements. The first tests were reported in the 1940’s by Winter and his co-workers in [1], which was followed by many other research work [2]-[10]. The systematic research which started in the middle of the 20th century, resulted in advanced design codes (e.g. AISI S100-07 in the USA, AS/NZS 4600 in Australia and New Zeeland, EC3-1-3 in Europe), which allowed the spread of CFS elements. In recent years CFS elements are used more and more as primary load-bearing structures in pallet racks, industrial buildings and residential houses. CFS structures are found to be efficient for moderate load levels and provide fast and effective construction method, which reduces the cost of erection.

One of the disadvantageous features of these structures, however, is that the several requirements arising during the design of residential buildings can only be satisfied by several different materials (heat insulation, insulation against moist, finishing of surface). This drawback can be avoided if a single material with optimised properties is used. Such building material may be the polystyrene aggregate concrete (PAC), which contains polystyrene granules and admixtures to improve material properties [11]. Since the mixture used for this research does not include gravel, the mechanical behaviour of the PAC is quite similar to those produced by foams, and is governed by the amount of cement paste [12]. For practical application the material properties can be characterised through the bulk density of the mixture, which is also the function of the amount of cement paste. Nowadays PAC is mostly used for non-structural elements such as insulation, filling material and fire protection [13]. Using PAC as infill of walls in light-gauge residential buildings the beneficial properties of the material may be utilised (heat/moisture insulation, fire protection). Moreover, since the PAC would fully surround the CFS elements, bracing of CFS elements can also be counted on, which results in even more cost efficient structures.

A current research and development project at the Department of Structural Engineering, Budapest University of Technology and Economics (BME) is being undertaken to develop a design method for residential buildings made of PAC encased CFS elements. Previous research work – presented in [14] and [15] – was focused on beams and columns to gain information on the load-bearing behaviour of the structural system. Both member tests (with single beam and column) and large scale panel tests (made up of several members) were studied. Tests showed that the PAC-bracing has beneficial effect on the stability phenomena and can increase the load-bearing capacity by 10-190 % depending on the parameters of the specimens. The material was able to restrain the global and distortional buckling modes of steel elements; thus the main failure mode was found to be local buckling. Shear tests were also conducted on wall panels [16], which confirmed the previous findings on the important role of local buckling.

In order to have an efficient building system, the increment of load-bearing capacity due to PAC-bracing in column-end connections should be at least the same as it was found in the member tests. For this end, the aim of the research was to investigate several types of column-end joints, their behaviour and ultimate load increments to find optimal build-ups in terms of load-bearing capacity, simplicity and cost efficiency. As these connections should be the part of a complex structural building system, the actual build-up of specimens should consider the characteristics of it. The connection types and specimen arrangement were determined based on these conditions.

2 Test program

Altogether six different joint types were investigated in the presented experiments, applying four steel cross-sections. Each specimen was a 300 mm high stub-column with two 300 mm long U-section tracks at both ends forming the column-end joint. The load was introduced by a 140x50 mm plate element which concentrated the loads at the C-section columns. A sample of specimens can be seen in Fig. 1.

The first built-up was the traditionally used simplest one, where the C- and U-sections were connected with one self-drilling screw at each side of each end through the flanges of the sections. The type, where the U-sections had 0.9 mm thickness, is called as A1 henceforth. As the anticipated failure mode of the elements was local buckling at the connection zone, where the stiffening effect due to PAC is the less effective, all subsequent modifications were concentrated to this zone. The first modification used 1.5 mm thick U-profiles (A1T type) to strengthen it and thus increase the failure load if the failure includes both profiles. Another type was similar to the A1 with double amount of screws, establishing stronger connection between the constituting parts (A2 type).

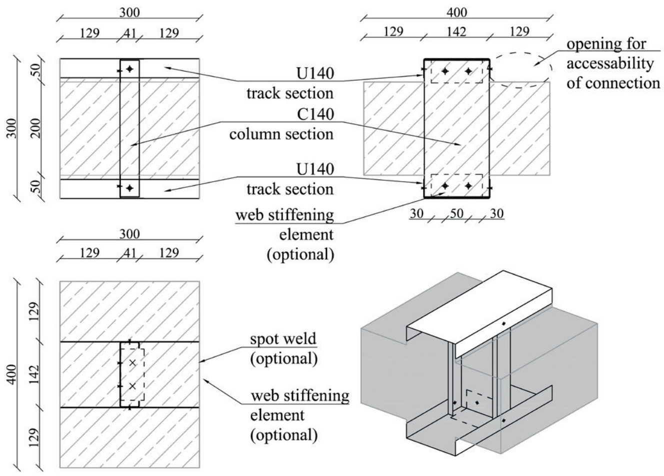

Fig. 1 Column-end joint element build-up of a C140 cross-section

As the spot-welding becomes more effective, this innovative connection type may also be favourable establishing the connection of the column-end. So the A1 type was modified to analyse the behaviour of spot-welded specimens (type AW). The arrangement was the same as in the case of the A1 type elements, but spot-welds provided the connection between the C- and U-sections, instead of the screws. To establish the welded joint a Pei Point PX1500P welding machine was used. The produced welds were full-strength connections, and had no sign of defect of welding.

Based on previous results of panel tests reported in [16] a new type of connection was also tested. As the failure of those elements was indicated by the buckling of the web, a web-stiffened type of connection was also designed (W type). The stiffening was done with an angle section connected to the web of the C- and U-sections. The connection between the C-profile and the angle section was done by self-drilling screws, while the U-section was connected to the angle by spot-welding (see Fig. 1). Apart from this stiffening, the original connection was also made between the flanges of the C- and U-sections.

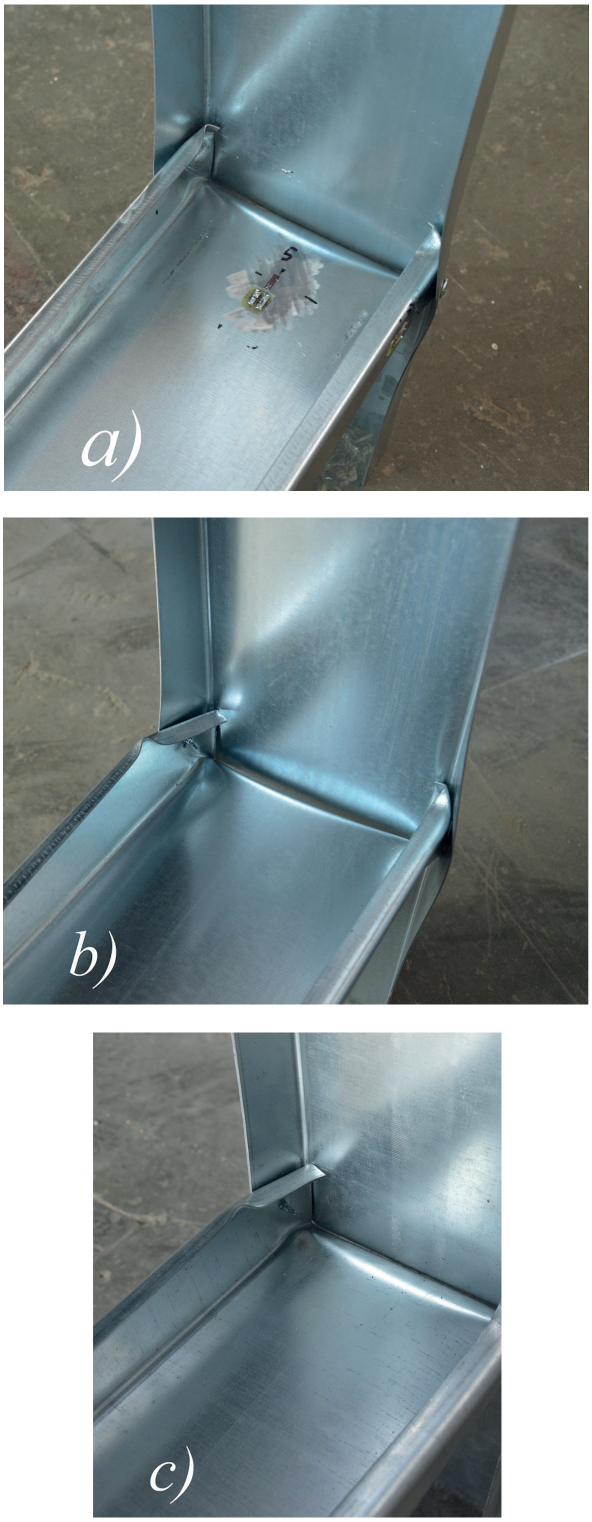

Fig 2. Failure modes of unbraced column-end elements: a) crinkling of column-end; b) mixed type; c) distortional-like failure

Both unbraced and braced elements were tested. The PAC-block was 400 mm in thickness resulting in 155 mm and 130 mm concrete cover for C90 and C140 cross-sections, respectively. The last arrangement type representing the separation walls (called AT) had only 30 mm of concrete cover on both sides resulting in smaller PAC-thickness (150 mm and 200 mm, respectively). The PAC block was 300 mm in height in all cases.

The four applied steel cross-sections consisted of two shapes (C90 and C140), and two thicknesses (0.9 mm and 1.5 mm). The flange of each profile was 41 mm wide, while the stiffener lip was 13 mm. The rounding-off of the sections resulted in a small initial gap between the webs of the track and column sections. The applied steel grade was DX51D+Z. Coupon tests were completed to determine the actual yield stress of each material, the results are summarised in Table 1.

Table 1 Material properties of steel sections

#	nominal thickness [*mm*]	yield strength [*N/mm*^{2}]	tensile strength [*N/mm*^{2}]	corresponding specimens
1	0.9	372	391	C90-10 all
2	0.9	364	391	U90-10 all
3	0.9	314	391	C140-10, U140-10 without PAC
4	0.9	343	371	C140-10 with PAC
5	0.9	367	382	U140-10 with PAC
6	1.5	355	382	C90-15 all
7	1.5	305	380	C140-15, U140-15 without PAC
8	1.5	282	352	C140-15 with PAC

The PAC mixture was the same as used in previous tests (WM-type, see [14]) with nominal bulk density of 200 kg/m3. Each concrete block had a 50 mm empty space at the top and bottom end (Fig. 1), which are representing the openings necessary for accessing the joint zone during real construction. The tests were carried out by a Zwick Z400 loading machine, in the age of 28 days of PAC. The axial force and crosshead displacement of the loading machine were measured during the tests along with the bulk density of the specimens. The final combinations of tests can be seen in Table 2. Each type was triplicated, thus altogether 18 unbraced and 48 braced experiments were done.

Table 2 Experimental matrix

#	type	mixture	description	C90-10	C90-15	C140-10	C140-15
1	A1	0	basic without PAC	-	-	+	+
2		WM	basic with PAC	+	+	+	+
3	AT	WM	basic with PAC, separating wall	+	+	+	+
4	AW	WM	spot-welded with PAC	-	-	+	+
5	W	WM	stiffened web with PAC	+	+	+	+
6	A1T	0	basic with U90-15 profile	-	-	+	+
7	A2	0	double screw without PAC	-	-	+	+
8		WM	double screw with PAC	-	-	+	+

3 Test results

Each of the unbraced specimens showed a failure mode which concentrated at the connection zone. These failure modes could be categorised into three types: (i) crinkling of column-end; (ii) distortional-like failure, involving both C- and U-sections; and (iii) mixed failure mode. The first deformations always occurred in the web, the final failure was indicated by the failure of the flanges. These modes can be seen in Fig. 2. The number of screws did not have any effect on the behaviour. The ultimate loads of these modes were quite close to one another.

Most of the PAC-encased elements showed the same behaviour as the unbraced ones, i.e. the failure occurred at the connection of the U- and C-profiles. The failure started at the web of the columns, which buckled, followed by the deformations of the flanges (Fig. 3). This behaviour was found in all specimens of the A1, A2, AT and AW types regardless to the applied steel cross-section.

Fig. 3 Connection failure of encased elements

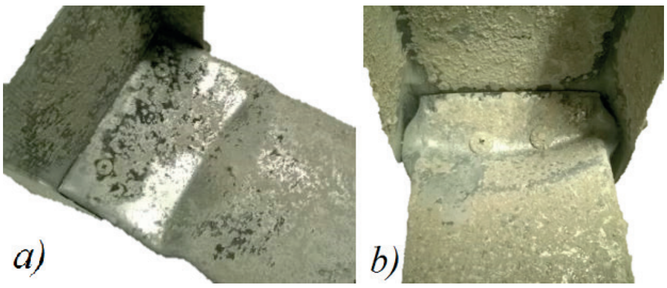

Fig. 4 Failure shapes of encased elements: a) local buckling; b) connection failure

The specimens having stiffened web (W type) showed a rather different failure mode. As the web had increased stiffness, the buckling could not occur at the end of the specimen, which resulted in that the failure zone shifted towards inside, and had the shape of the local failure mode of the member tests (Fig. 4 a)) [15]. All of the W type elements showed the same failure mode, except one with C90-15 cross-section (Fig. 4 b)). As this profile has the smallest web plate slenderness, it implies that the web stiffening is not as efficient in this case.

Fig. 5 shows four types of load-displacement curves registered during experiments. The curves can be divided into four different zones. The first zone corresponding to small load intensities is governed by the initial stiffness of the connection. The second zone is where the initial gap dominates the behaviour. The third zone is where contact is reached between the column and the track, while the fourth zone describes the post-ultimate behaviour.

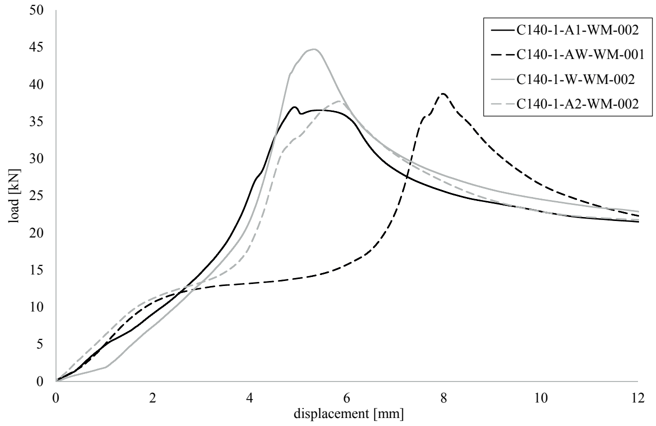

Fig. 5 Typical load-displacement curves of specimens

Most of the specimens had the same initial stiffness, regardless whether the connection was made up by screws or spot-welds. This behaviour is due to the fact that these connections did not produce any slipping behaviour for small load intensities. In some cases, the initial stiffness of the connection was found to be lower (e.g. specimen C140-1-W-WM-002). In these cases, a hardening was observed after a few millimetres of displacement, and no further effect was present due to the differences in the connection.

As the load increased the load-bearing capacity of the connection was reached, and if the initial gap was wide enough, a plateau-like behaviour started, which is the second zone of load-displacement curves. The self-drilling screws started to rotate, bending the flanges of the constituent steel elements as well. The spot-welds showed a similar behaviour, plastic deformations occurred in the flanges, too (Fig. 6). Not all the specimens showed this slipping behaviour, and the length of the slip was not uniform either. The spot-welded specimens behaved very much alike, all of them had a long plateau. Other specimens (e.g. C140-1-W-WM-002) stepped over this behaviour.

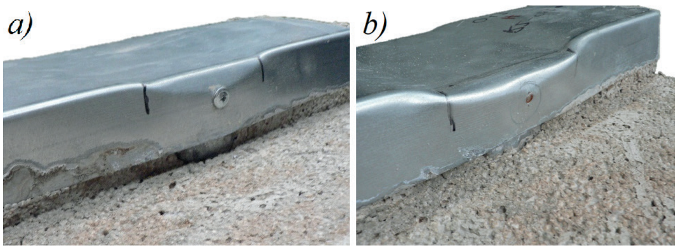

Fig. 6 Deformation of the connections: a) self-drilling screws; b) spot-welds

In the third zone significant hardening could be observed due to the fact that the initial gap closed. This resulted in that the web of the columns began to take part in the load transmitting, thus increasing the stiffness. The final stiffness of the

Table 3 Ultimate loads of column-end types

#	type	mixture	C90-10 ult. load [kN]	C90-10 incr. [%]	C90-15 ult. load [kN]	C90-15 incr. [%]	C140-10 ult. load [kN]	C140-10 incr. [%]	C140-15 ult. load [kN]	C140-15 incr. [%]
1	A1	0	-	-	-	-	21.93	-	58.36	-
2		WM	30.36	-	63.51	-	38.54	-	78.12	-
3	AT	WM	31.93	5	75.65	19	40.70	6	67.43	-14
4	AW	WM	-	-	-	-	39.28	2	62.68	-20
5	W	WM	35.73	18	66.52	5	46.18	20	72.31	-7
6	A1T	0	-	-	-	-	22.59	-	53.61	-
7	A2	0	-	-	-	-	23.80	-	57.89	-
8		WM	-	-	-	-	35.33	-8	69.16	-11

specimens was found to be close to one another, which means that the load transferring is identical in the different cases, no additional relative displacement developed between the track-sections and columns.

The last zone of the load-displacement curves starts after the ultimate load, when the final failure is reached. As it can be seen in Fig. 5 the post-ultimate behaviour and residual strength of the different specimens are similar, which is in good correlation with the similarity of the failure modes. The load-displacement curves of the W type specimens can be categorised into these zones too, although the failure mode of these elements are different from the others.

4 Discussion of test results

4.1 Ultimate load levels

Table 3 shows the measured load values for each investigated arrangement. One of the important findings is that stiffening the U-section (A1T type) did not result in increased load-bearing capacity, moreover a decrement could be observed for columns made of thicker plate. This shows the effect of eccentricity of the screw connection, which cannot be avoided in this type of joints.

The increments in Table 3 were calculated based on the ultimate load of the corresponding PAC-braced A1 type values. It is clear based on the values in Table 3, that specimens with 1.5 mm plate thickness showed rather scattered results. For the C90-15 specimens the application of smaller PAC-block was highly beneficial (19 % increment), however, the same for C140-15 specimens was highly unfavourable (14 % decrement). The other arrangements applying C140-15 cross-section showed decreased load-bearing capacity, too.

4.2 Effect of bulk density of PAC

For resolving these findings, the resistance to yield strength ratio of each specimen was plotted versus the bulk density of PAC (see Fig. 7 and Fig. 8). The same build-up is represented by the same marker style, while the colour densities represent the plate thickness of steel. As it was described before, the A1, A2, AT and AW type specimens showed the same failure mode, thus they can be treated together. The trend lines on the diagrams were calculated based on this assumption for connection failure (i.e. without the W arrangement). Based on these diagrams it is clear that the resistance of specimens made up of thinner plate is less affected by the bulk density of PAC. The smaller resistance of the 0.9 mm thick profiles is significantly increased by the bracing effect of PAC, thus the variance of the material properties of PAC does not have strong effect on the ultimate load-bearing capacity. For 1.5 mm thick plates, however, this effect is more significant between the two properties. As the stiffness of the steel core is greater in this case, the increment due to PAC-bracing is lower and strongly affected by the variance of the properties of PAC. As the bulk density and elastic modulus of PAC are correlated [12], this connection also appears between the ultimate load and the bulk density. For thinner plates the PAC can provide almost the same bracing effect, regardless of its actual material properties. This behaviour agrees well with previous findings for compression elements (see [15]). Thus the scatter found in the ultimate loads (see Table 3) can be explained by the different bulk densities. As fewer data are available for the web-stiffened elements (black markers), a similar correlation cannot be found with similar certainty.

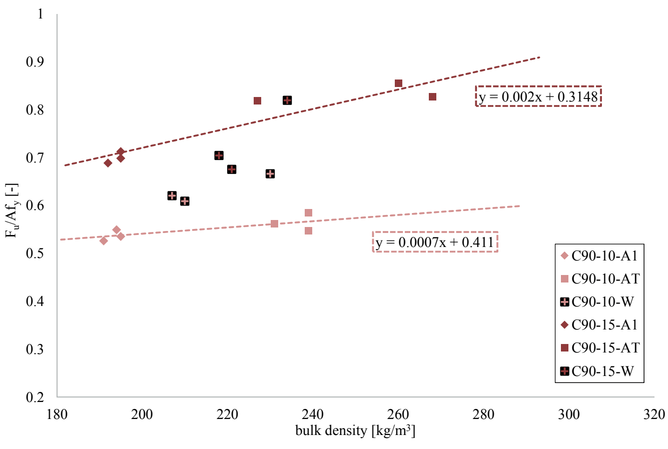

Fig. 7 Ultimate load to yield strength ratios vs. bulk density of PAC for C90

Table 4 Comparison of compression member and column-end results

#	cross-section	yield stress [N/mm2]		ultimate load [kN]			diff.
		member tests	connection tests	member tests – original	member tests – modified	connection tests	[%]
1	C90-10	372	372	36.76	36.76	35.73	-2.8
2	C140-10	314	343	41.38	45.20	46.18	2.2
3	C140-15	305	282	78.18	72.28	72.31	0

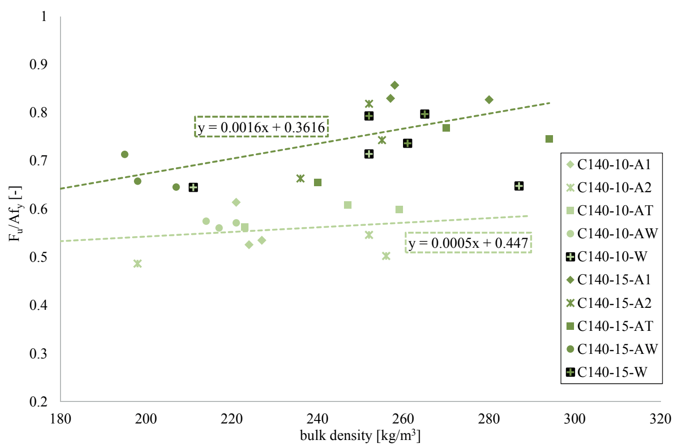

Fig. 8 Ultimate load to yield strength ratios vs. bulk density of PAC for C140

It is clear based on the results of thinner plates, that significant modification in the resistance could be achieved by the web-stiffening (achieved increment around 20 %). For 1.5 mm plate thickness no considerable increment is present. This confirms the failure mode of Fig. 4 b). As the failure mode is found to be the same as in the compression member tests in [15], a comparison is made between the ultimate loads, taking into account the different yield stresses of the base materials.

The results are in good correlation – as shown in Table 4 –, which means that using this web-stiffened column-end connection the load-bearing capacity can be increased to reach the value corresponding to the element failure.

The overall average increment due to PAC-bracing is 65 % for C140-10 and 8 % for C140-15 profiles, assuming 200 kg/m3 nominal bulk density and taking into account the different yield stresses of steel.

4.3 Failure mode

As can be seen in Fig. 5, some of the specimens showed a small decrement in the load level, or in the stiffness near to failure. This disturbance results from the failure of the column-web. As found in the case of unbraced elements, the final failure of the connection occurs when the flanges of the column fail. Thus the effect of the failure of the web can be observed in the load-displacement curves prior to the failure, if the flanges have enough additional load-bearing capacity. When the load-bearing capacity is not sufficient, the failure of the web induces the failure of the flanges, too. Based on the observations the following parameters have significant effect on the behaviour and failure of encased column-end joints: (i) resistance of the flanges of the column; (ii) resistance of the web of the column; (iii) width of initial gap; (iv) behaviour of connection between track and column; (v) imperfections; and, in correlation with the findings of [15], (vi) material properties of infill material. Based on these components a mechanical model can be derived to calculate design resistance of this kind of joints.

5 Conclusions

A novel thin-walled structural system is under development which aims to utilise the beneficial effect of continuous bracing achieved by encasing CFS elements in ultra-lightweight-concrete. After investigating compression, flexural elements and shear wall panels it was necessary to experimentally analyse the behaviour of column-end joints in order to have a more efficient building system. Altogether six types of end connection were tested by 18 unbraced and 48 braced specimens. All modifications were done in order to strengthen the connection zone, and to examine novel connection types. Altogether four cross-sections were tested in a build-up which contained the characteristics of the future building system.

The results showed that the unbraced elements failed due to connection failure, which could be categorised into three sub-modes. These sub-modes had similar load-bearing capacity and were always marked by the failure of the flanges. The dominant failure mode was found to be similar to those produced by the unbraced elements, i.e. connection failure occurred. Based on the load-displacement curves of the braced specimens, four zones were detected which govern the behaviour: (i) initial zone; (ii) slipping zone; (iii) contact zone, and (iv) post-ultimate zone. The three applied types of screwed end connections (A1, A2, and AT) were equally efficient for each investigated cross-section when applied with PAC-encasement. The spot-welded connection (AW) was fully efficient, too, both in terms of failure mode and load-bearing capacity. The failure mode of web-stiffened specimens (W) showed different failure mode, with increased load-bearing capacity for thinner plates. This failure was found to be the same as of the compression members before, i.e. local buckling occurred. Investigating the effect of PAC bulk density on the ultimate loads it was found that for 0.9 mm plate thickness it had not significant effect, but for plates with 1.5 mm the correlation could not be neglected. The significant parameters describing the behaviour of column-end joints were identified.

The presented experiments and results can provide with a background for designing cost efficient buildings by PAC-encased CFS elements. Further numerical analyses should be made in order to widen the parameter range of the results as well as to derive design procedures for this essential structural detail.

Acknowledgement

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