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Improving Thermal Insulation Properties for Prefabricated Wall Components Made of Lightweight Aggregate Concrete with Open Structure

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Abstract. Porous concrete is commonly used in civil engineering due to its good thermal insulation properties in comparison with normal concrete and high compression strength in comparison with other building materials. Reducing of the concrete density can be obviously obtained by using lightweight aggregate (e.g. pumice). The concrete density can be further minimized by using specially graded coarse aggregate and little-to-no fine aggregates. In this way a large number of air voids arise. The aggregate particles are coated by a cement paste and bonded together with it just in contact points. Such an extremely porous concrete, called 'lightweight aggregate concrete with open structure' (LAC), is used in some German plants to produce prefabricated wall components. They are used mainly in hall buildings, e.g. supermarkets. The need of improving thermal insulation properties was an inspiration for the prefabrication plant managers, engineers and a scientific staff of the Technical University of Kaiserslautern / Germany to realise an interesting project. Its aim was to reduce the heat transfer coefficient for the wall components. Three different wall structure types were designed and compared in full-scale laboratory tests with originally produced wall components in terms of load-carrying capacity and stiffness. The load was applied perpendicularly to the wall plane. As the components are not originally used for load-bearing walls, but for curtain walls only, the wind load is the main load for them. The wall components were tested in horizontal position and the load was applied vertically. Totally twelve wall components 8.00 x 2.00 x 0.25m (three for every series) were produced in the prefabrication plant and tested in the University of Kaiserslautern laboratory. The designed and tested components differed from each other in the amount of expanded polystyrene (EPS), which was placed in the plant inside the wall structure. The minimal amount of it was designed in the original wall component type. Besides, two improved types of prefabricated wall had built-in steel lattice girders. The failure mode was the same for all the tested components: diagonal cracks occurred on the sides of each component due to their insufficient shear-force-capacity. The span deflection was measured during all the tests by means of LVDTs. Load-carrying capacities obtained in the tests were for all wall structure types similar and much higher (many times) than internal forces (i.e. bending moments and shear forces) calculated for a wind load acting on a typical hall building according to the German codes. An increased amount of EPS (up to 30 per cent in volume) did not influence significantly the wall structural strength. The use of the steel lattice girders caused some technological problems and led to a quality loss of the produced components. The future use of the lattice girders would require a change in the production process: it would have to be more labour consuming.

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1. Introduction

Lightweight aggregate concrete (LAC) is a valuable building material thanks to its good thermal insulation and strength properties. Therefore, it is used in self-supporting walls of the non-residential buildings such as supermarkets. Reducing density of the normal weight concrete can be achieved by using lightweight aggregate (e.g. pumice or claystone). The concrete density can be further minimized by using similar-sized lightweight aggregate particles, without sand. The particular aggregate particles are coated with cement paste in the concrete mixing process. Thin layers are created on the particles. Thanks to it, the aggregate particles glue together at the contact points, creating a uniform structure with a large number of air voids. The specific weight of the LAC reaches about 10 kN/m^3 (this was the case in the studies presented in this paper), i.e. 2.5 times less than for the normal weight concrete. The thermal transfer coefficient of LAC wall falls 10 times compared to the normal weight concrete wall. The drop in strength is also significant: in the tests described in the paper the average compressive strength of concrete in the range of 4.6 to 7.5 MPa has been achieved. This represents around 20% of the strength of the normal weight concrete commonly used in the modern construction. Wall components made of LAC are also suitable for fire walls due to their high thermal insulation properties. They prevent emission of high temperatures and separate building parts in case of fire. The high noise absorption capacity is another advantage of such walls.

Research on lightweight aggregate concrete with open structure is limited. The reason for that is relatively rare use of the LAC in the civil engineering in comparison with the normal weight concrete. The main field of the LAC use is a prefabrication. The European code including rules for design of prefabricated walls made of the LAC is [1]. This code incorporates some dimensioning and detailing rules from the code [2], which is the main European document for designing concrete structures. The laboratory tests conducted on the reinforced concrete structures made of LAC indicate a relatively high uncertainty. Consequently, the design formulas for the load-carrying capacities are conservative. The reason for that is a high variability of the LAC material properties. For example, Goltermann [3] in 1995 investigated experimentally about one hundred samples of walls and columns made of LAC, subjected to an axial and eccentric compression and obtained load-carrying capacities, which were much higher than estimated values. In case of small eccentricities, the proportion of the both experimental and calculated load-carrying capacities was between 1 and 5. For higher eccentricities the proportion exceeded 10. When the compressive force was subjected outside the cross-sectional core, but still inside the cross-section contour, the proportion of the experimental and estimated load-carrying capacities was equal even to 100 and more. The estimating load-carrying capacity of compressed walls and columns according to the modern code [1] provides a better consistency with experiments, but is still conservative [3] in comparison with reinforced concrete structures made of the normal weight concrete.

In the developed countries sustainable development politics is implemented in the construction industry since many years. The more and more restricted building regulations with respect to the energy consumption for heating residential buildings are being introduced. A German company producing large-sized curtain wall components made of LAC (figure 1) decided to invest resources in the optimization research in terms of thermal insulation. Three types of walls were designed. In each of them the proportion of expanded polystyrene (EPS) in the volume of the prefabricated component was increased compared with the original version of the wall produced so far. The increase was from 11.1% to 30.1%.

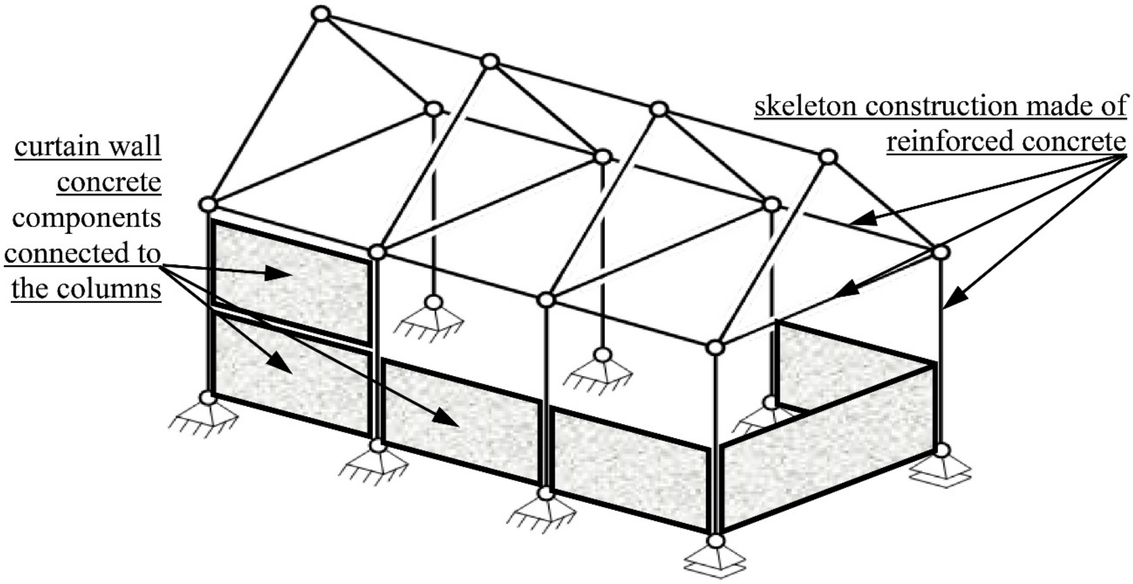

Figure 1. Structure diagram of a building erected from prefabricated wall components

2. Research Program

The research was conducted on the large-sized wall components produced by the company. It was decided to assume the uniform dimensions of all tested wall plates: 8.00 x 2.00 x 0.25m (figure 2). At a total plate thickness of 25 cm, EPS core was 8 cm, and the top and bottom concrete layers were 8.5 cm each. Both layers of concrete were reinforced with 8 mm diameter bars arranged longitudinally at a constant spacing of 9 cm and 6 mm diameter bars arranged transversely at a constant spacing of 20 cm. Reinforcement bars were located in the middle of the top and bottom concrete layers. Each of the wall components had a stiffening frame around their edges, made of LAC without EPS core. The thickness of this frame was 15 cm. The frame was extended in the area of transport anchors installed in the prefabrication plant for the purpose of the plates transport and assembly. The zones around each of the transport anchors were made exclusively of LAC and without EPS filling. Four anchors were built into each plate. Their length was 80 cm. Consequently, the depth of the zone made of LAC was 86 cm and its width varied in the range of 50 and 110 cm. The anchor zones are clearly visible in figures 2 and 3. Three test specimens were made for each of the four test series: one referential (original wall structure) and three newly designed wall structures. The following subsections discuss in detail all four structure types. A total of 12 elements were tested. The test specimens were marked with a two-digit symbol. The first digit is the test series number (1 to 4) and the second digit is the test specimen number of the given series (1 to 3). The digits are separated by a dot.

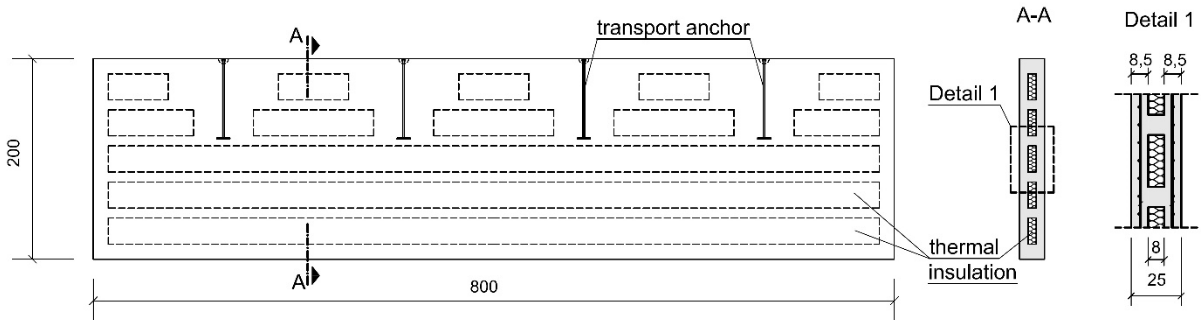

Figure 2. The structure of the wall component of the first test series

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Figure 3. The reinforced wall component (the first improved type) during the concreting process

2.1. The referential wall component structure

As with all tested wall components, the wall structure in the mass production of the company (figure 2) consisted of a 15 cm LAC frame surrounding the whole component and four longitudinal ribs of 10 cm thickness. The ribs were separated by EPS plates of 8 cm thickness, which constituted thermal insulation. There were no transverse ribs, barring strengthened zones around the transport anchor.

2.2. The improved wall component structure - the first type

The first type of improving wall component structure (figure 3) consisted in reducing the number of longitudinal ribs (from four to three) and reducing their width (from 10 cm to 8 cm). The volume of built-in EPS increased by 11.1% compared to the original wall. Changing the structure obviously resulted in the reduction in the longitudinal stiffness of the plate, so it was decided to design a second type of the wall component structure with increased stiffness of the ribs.

2.3 The improved wall component structure - the second type

This type did not change any geometrical dimensions in relation to the first type but the longitudinal ribs were reinforced with steel lattice girders commonly used in the floor slabs of *Filigran* type (figure 4).

2.4. The improved wall component structure - the third type

Expecting good results of the use of the lattice girders, it was decided to design another plate structure type, this time completely devoid of concrete longitudinal ribs. Their role was met by built-in steel lattice girders - three pieces per wall, located in the plan exactly in the axes of concrete ribs used in the first and second improved structures. The volume of built-in EPS was increased by as much as 30.1% in relation to the referential wall.

3. Test set-up and test procedure

All of the wall components were tested for their load-carrying capacity and strain caused by loads perpendicular to their plane. The main load is the wind pressure or suction. The discussed wall components do not perform bearing function in buildings because of the low strength, but they act only as curtain walls. When designing a test set-up, the uniformly distributed load such as wind pressure and

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suction was abandoned. Instead, the scheme with simply supported beam loaded with two concentrated forces positioned roughly at ¼ span was applied (figure 5).

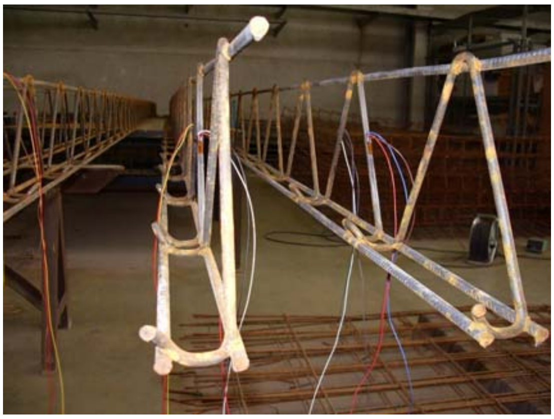

Figure 4. Steel lattice girders prepared to be built in the tested wall components of the improved structure (the second and third types). Installed strain gauges with electric wires are visible

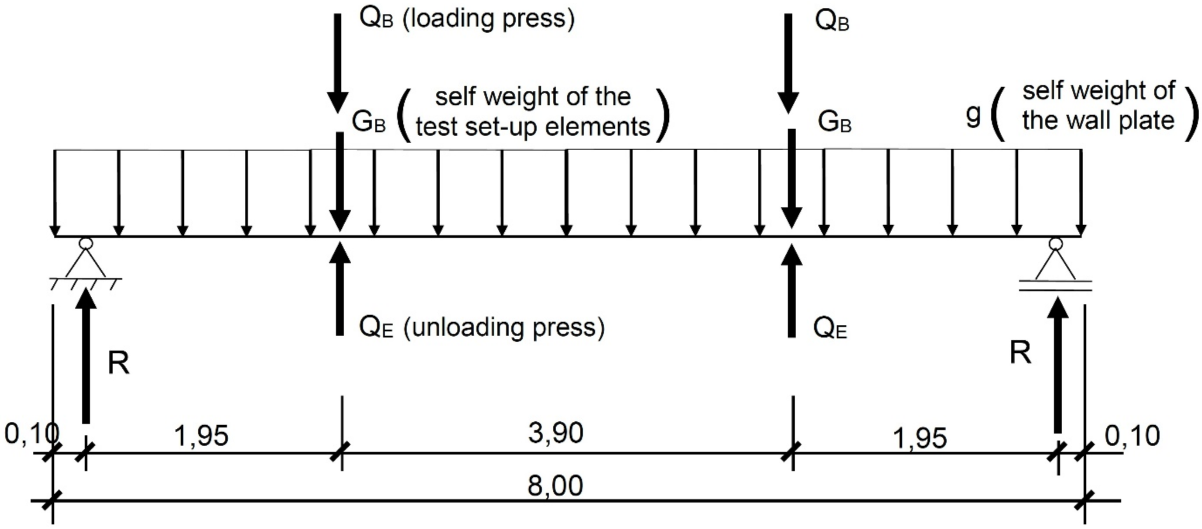

Figure 5. Scheme diagram of the test set-up

The challenge was the plate self-weight which, in fact, never affects a wall component as unfavourably as a simply supported beam with span equal to the length of the component. It is not a case even during transport or assembly of the prefabricated units. Therefore, the plate self-weight was predominantly eliminated by the additional support of the plate at two points located exactly in the same place as the span load points. This support was made on hydraulic cylinders, which, in the course of the experiment were gradually lowered until they completely transferred the weight of the plate to the external supports. At this stage of the experiment, marked as phase one (out of two), all measurements were performed that continued in phase two. The second phase of the experiment consisted in loading two concentrated forces transmitted from the top of the hydraulic press through a system of three orthogonally-oriented steel sections beams (figure 6). Obviously, the self-weight of these beams was also taken into account. Figure 7 illustrates a sample course of the experiment in time, illustrating its two phases: lowering the intermediate supports and proper loading.

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During the loading process in the subsequent phases unloading and loading force values (expressed in [kN]) were recorded using force gauges. Moreover, plate deflection values were recorded using linear variable differential transformers.

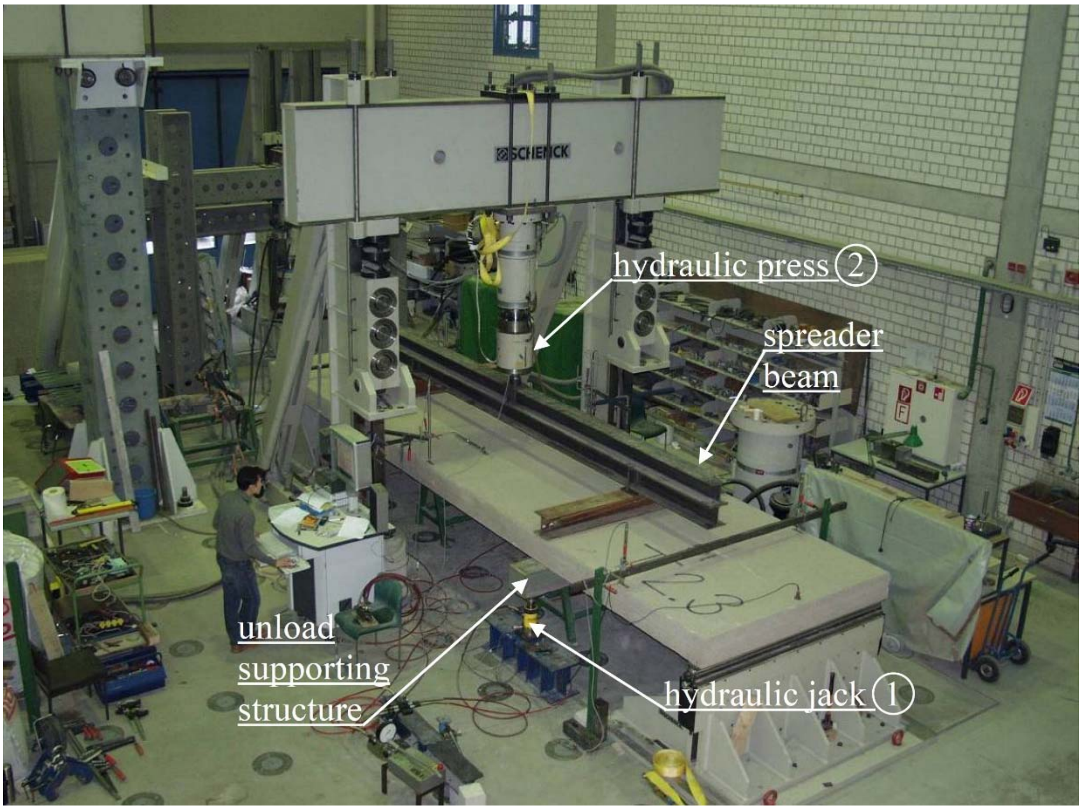

Figure 6. General view of the test set-up

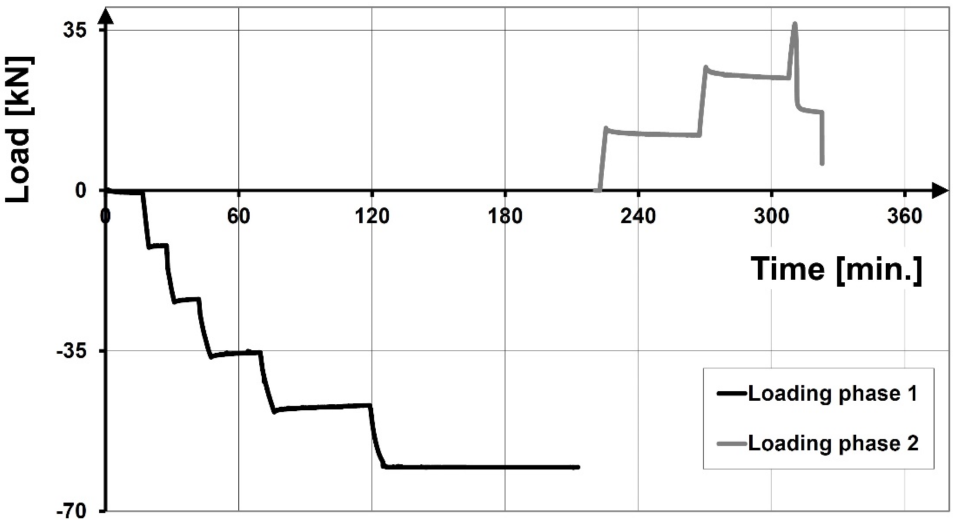

Figure 7. The course of the experiment over time on the example of the first component of the second test series

4. Results and discussions

4.1. The results of the experiment

The strength of concrete of all twelve wall components was tested in a press on 100 mm-diameter-cylinders of height 100 mm, cut from plates using a crown drill. The test specimens were cut from plates after they were tested and taken from the test set-up. Drilling sites were selected in the areas where the transport anchor was built in. Three concrete cylinders were tested for each wall component. The highest average concrete strength achieved was 7.5 MPa (for component 3.2) and the lowest concrete strength was 4.6 MPa (for component 4.2). The concrete specimens used for the strength testing were then used to test its dry density according to code [4]. The determined average dry density of the concrete ranged from 930 kg / m^3 to 1130 kg / m^3 and gravimetric water content - from 8.9% to 16.1%.

Reinforcement steel bars ϕ8 mm laid in the tested plates as well as steel bars of stiffening lattice girders built in the plates of series 3 and 4 were tested as well. Both bars ϕ8 mm of their bottom chords as well as bars ϕ7 mm of their cross braces were tested. For each bars category three test specimens were taken. The code test method [5] was used. The conventional yield strength (corresponding to the strain of 0.2%) achieved ranged from 510 to 581 MPa.

All the test specimens were failed in the same mode: due to insufficient shear force capacity. A typical view of the plate after failure is shown in figure 8.

Figure 8. A typical failure mode of the wall components on the example of specimen 2.2

The relation between the applied load and the midspan deflection of the plate was similar in nature for all tested plates. Figure 9 illustrates the example relations obtained for plates of the second test series.

The highest load-carrying capacity was achieved for the third test series, while the lowest for the fourth one. The lowest capacity was obtained for the plate 4.1 and was 21% lower than the average capacity of all twelve plates, whereas the highest capacity was obtained for the plate 3.1 and was 17% higher than the average capacity of all twelve plates. However, it must be noted that the lowest concrete compressive strength was obtained for the fourth test series. Deviation from the mean value of concrete strength of all twelve plates was on average 18% for the fourth test series.

In the wall components of third and fourth test series serious defects occasioned by built-in steel lattice girders were observed. After the test completion the plates were cut by a diamond saw in a transverse direction. On the section surface, horizontal slits formed between the concrete layer laid below and above the top chord of the lattice girders (i.e. on the concrete top layer of the plate) were detected. It was found that these defects must have occurred in the plant when the top layer was compacted with a special heavy steel cylinder. Under its weight, the bars of the top reinforcement mesh were bent in the areas above the EPS, and they were resting on steel lattice girders. When the cylinder passed, the bars elastically resumed to their original position, but the concrete of the top layer that coated them from the bottom remained dented.

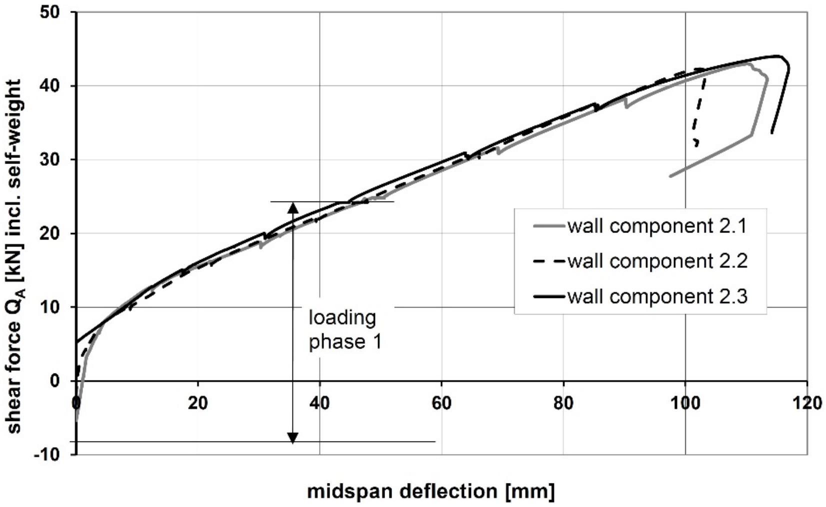

Figure 9. Load-deflection curves for all three wall components of the second test series

It was determined that in order to avoid this defect the change of the prefabricated elements production technology would be necessary. The top bars of the reinforcement mesh should be threaded under the top chords of steel lattice girders rather than being supported by them. The production process would be then more labour-consuming, which obviously would increase the labour cost.

4.2. The static-strength calculations of the wall components and the comparison of the results with experimental load-carrying capacity

The static-strength calculations of the tested wall components were performed in two main aspects. Firstly, the maximum characteristic internal forces in the component were determined by the standard wind pressure or suction. Secondly, the shear and bending capacity of the wall plate was calculated for each of its structure type. In the first analysis, the code [6] was used, whereas in the second one - code [1], considering the average strength of concrete obtained from the tests.

For the purpose of the first analysis, a typical hall building structure (e.g. supermarket) was assumed according to figure 10, constructed from the prefabricated components. The assumed height of the building was H = 8.00 m, the wind zone 4, the inland area according to code [6]. The wall component built into the corner of the building and subjected to wind suction proved to be extremely loaded. In this component, the internal forces were calculated using scheme of a simply supported beam subjected to an uniformly distributed load of 1.14 kN/m^2 on the length of 3.2 m and 0.76 kN/m^2 on the remaining length of 4.8 m. A maximum bending moment of 7.08 kNm/m and a maximum shear force of 4.01 kN/m were obtained. These extreme values of internal forces in the serviceability limit state were compared to the extreme values of bending moments and shear forces obtained in the experimental investigation (figure 11).

The load-carrying capacities determined theoretically according to the code for structures made of LAC [1] were also included in the comparison. The aim was a verification whether the reinforced concrete structure working model used in the code is adequate for the discussed prefabricated wall components. Results of the comparisons presented in figure 11 are limited to the test series 1 and 2. The shear failure forces achieved in experiments were more than five times higher than the shear forces in the serviceability limit state. Ergo, the components were designed safely. The calculation method of the components load-carrying capacity according to code [1] has proven to be well calibrated. The ratio of experimentally determined load to theoretically determined load was 0.91 on average. The fact that it is less than 1.00 may raise concerns about the safety of the code shear dimensioning procedure. However, this is solely owing to the fact that in place of the characteristic values of strength, as indicated in the code procedure, the mean values of concrete strength obtained from the tests were taken into account in the described calculations.

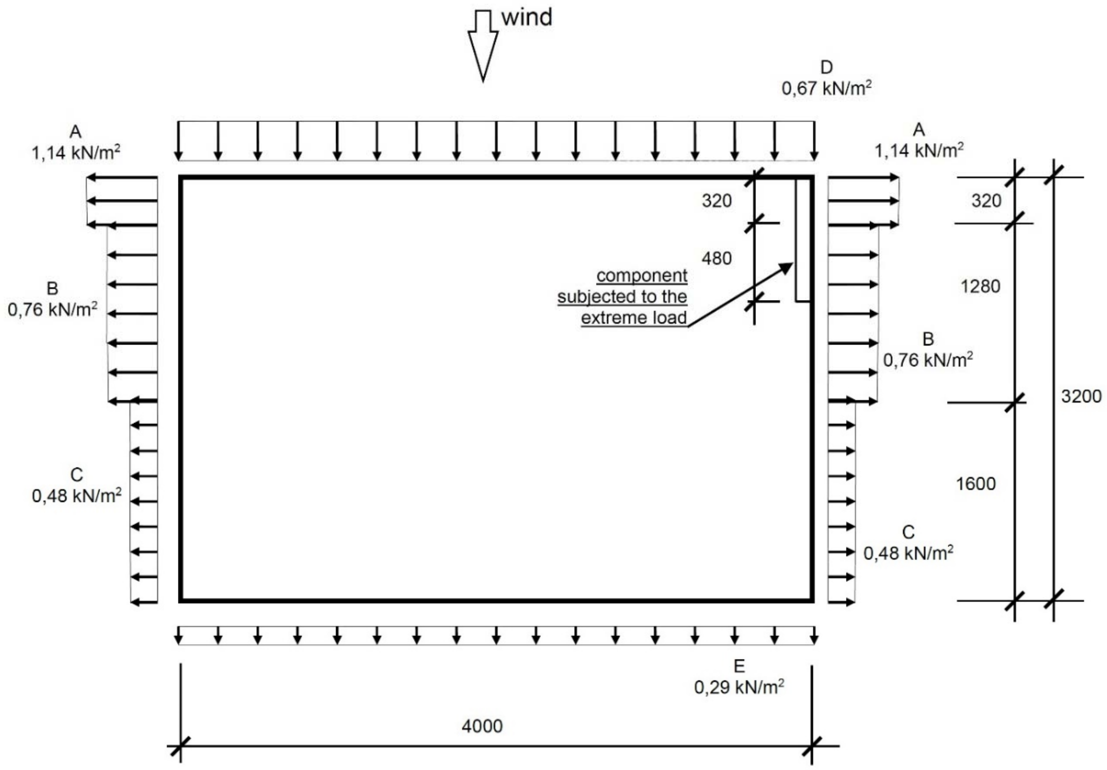

Figure 10. Wind load of a typical building constructed from the tested wall components. Plan view, dimensions in [cm]. Characteristic values of wind pressure and suction were given.

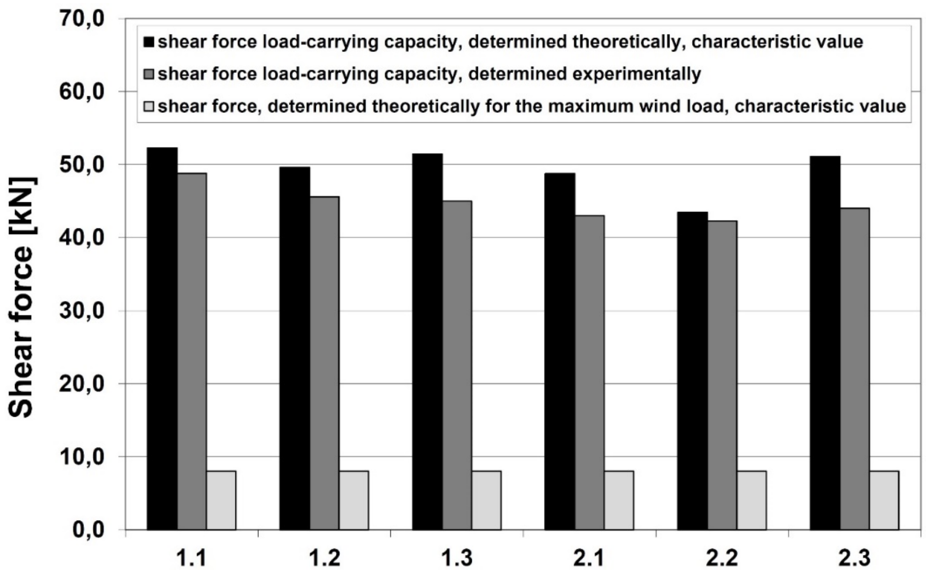

Figure 11. Shear force load-carrying capacity of the wall components in the test series 1 and 2 (determined theoretically and experimentally) against shear force in the serviceability limit state

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During testing, the formation and spreading of the cracks on the lateral edges of all the wall components was recorded. None of the tested components exhibited cracks in the area of the load corresponding to the serviceability limit state.

5. Conclusions

All of four tested structure types of the prefabricated lightweight aggregate concrete wall components made of LAC showed large up to several-fold load-carrying capacity reserves compared to the wind load that may actually occur. In addition, the load-carrying capacity of each structure type proved to be close to each other. It does not mean, however, that any structure type could be applied. For the technological reasons indicated in section 4.1, the second and third improvements to the structure would not be suitable for application, i.e. those with steel lattice girders. Only the first improved type might be recommended. In this type the share of built-in EPS in the total prefabricated component volume increased by 11.1% when compared to the original version of the wall structure produced so far, which can be considered satisfactory. It would also be possible to implement structure types with the lattice girders, but would require a change in the production process causing prefabricated components construction more labour-consuming.

The code [1] predicted well the shear resistance of the tested wall components made of LAC.

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