Evaluation of the condition of the external layer of walls in the national technological system "S-Sz" (Szczecin System) of large-panel prefabricated construction

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Abstract. The condition of the texture layer of external walls directly affects the durability of a large panel building. The results presented in the article are the result of an inventory and a number of studies carried out in 2018 on selected residential buildings in a local variant of the Szczecin Large-Panel System. Both in situ tests (layer thickness measurement, reinforcement location, determination of lagging thickness) and laboratory tests (strength test, carbonation range) were carried out. Significant deviations of the examined wall elements from the design assumptions were proved. Moreover, it was found that the carbonated concrete of the external layer does not adequately protect reinforcing steel. With the use of so-called sandwich samples, the compressive strength of the concrete was confirmed. A favourable ratio of specimen dimensions was established for the results of lightweight concrete strength tests.

1 Introduction

In Poland, in the years 1960-1990, several updates of the heat transfer coefficient for external partitions were introduced - ultimately, after the introduction of the PN-82/B-02020 [1] thermal standard, all leading large panel systems had to be modernised by using "warmer", three-layer external walls. They consist of the following layers: load-bearing (structural), insulating and external.

It is the external layer of a wall that is most exposed to atmospheric factors, and thus to changing working conditions [2]. This layer is responsible not only for the appearance of the building, but also protects the thermal insulation and the load-bearing part (mechanical protection and weather protection) [3].

The research in this article concerns the walls of the large panel housing estate built in a local variant of the Polish nationwide technological system of Szczecin - it is responsible for nearly 14% of the large panel resources in Poland [4]. The "S-Sz" system (Szczeciński) also took part in the construction of many Poznań housing estates, e.g. in such housing estates as Grunwald (Kopernika housing estate), Nowe Miasto (Polan housing

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estate, Stare Żegrze housing estate), Jeżyce (Słowiańskie housing estate), Stare Miasto (Bolesław Chrobry housing estate, Jan III Sobieski and Marysieńka housing estate, Bolesław Śmiały housing estate, Stefan Batory housing estate, Władysław Jagiełło housing estate, Władysław Łokietek housing estate) [5].

2 Characteristics of the tested buildings

Six multi-family buildings of Stefan Batory's housing estate in Poznań were selected for research. The buildings were constructed in a large panel technological system "SL-85", i.e. a local variant of the Szczecin System. The buildings numbered 3-8 (selected for research) were put into use in 1987-1988 - the whole large panel housing estate was built in 1986-1993. The buildings have 5 above-ground storeys and one underground storey and according to the applicable technical conditions [6] they should be classified as "medium-high" buildings (MH). Table 1 shows the main technical data of the objects. Fig. 1 illustrates the projection of one of the buildings and Fig. 2 the façade with a visible external layer. The objects selected for testing were indicated by the housing estate administration as those which will be subject to thermal upgrading first.

Table 1. Characteristics of buildings

	Description:
Types:	residential, multi-family - SL-85 large panel technology
Building years:	1986-1988
Height / no. of storeys:	17.65 m / 5 floor above ground, 1 floor underground
Constructional arrangement:	transverse with structural wall modules 4.8 m and 2.4 m
Storey height:	2.8 m (2.5 m basement)
Type of external walls:	three-layer: load-bearing gable type "T" (40 cm thick) and longitudinal self-supporting type "Z" (36 cm thick)

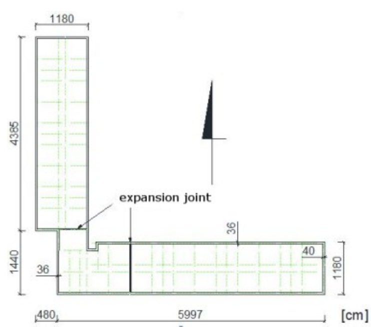

Fig. 1. Section of building No. 7.

Fig. 2. Facade of building No. 8.

3 General characteristics of wall elements

In order to assess the condition of the external layer, it is necessary to be aware of the structure of the whole, three-layer wall element. According to the "S-Sz" system solution, the panel consisted of an inner layer of expanded clay concrete B15 or standard concrete B20. Reinforcement of the load-bearing part was planned in the form of steel ladders 2 Ø 8 mm (34GS) located on the perimeter of window openings, and additionally in the lintels and corners of the openings.

Thermal insulation in the walls was made of 60 mm thick mineral wool (compressibility 8% under 4 kPa load).

The external layer was made of expanded clay concrete B15 or standard concrete B20 reinforced with welded nets made of Ø 4.5 mm (St0S) rods at a spacing of 15x15 cm [4]. The façade was to be finished in the form of aggregate texture or glass mosaic, and in local variations also in the form of painted plaster and an imprint in concrete.

3.1 Characteristics of the tested three-layer panel and the external layer

Wall elements of the analyzed buildings were three-layer: 36 cm and 40 cm thick (Table 1). The load-bearing layer consisted of expanded clay concrete with a thickness of 220 mm and 260 mm, respectively. Thermal insulation in the form of 60 mm thick wool was covered with a 80 mm thick expanded clay concrete external layer (Fig. 3).

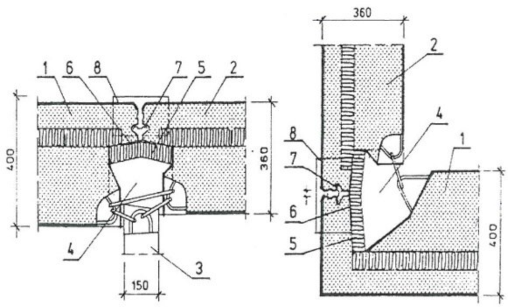

Fig. 3. Vertical joints of layered external walls made of expanded clay concrete "S-Sz" (modernisation). 1 - gable wall T, 2 - curtain wall Z, 3 - internal wall, 4 - concrete B15, 5-foamed polystyrene 60 mm, 6 - isofoil, 7 - profile insert, 8 - zinc coated sheet [4].

The elevations in the housing estate in question were finished off in various ways-as shown in Fig. 4. All the panels selected for the study had a finish in the form of a glass mosaic, which, for technological reasons, was made first, at the bottom of the structure – in a wall of this type the external layer was usually done first.

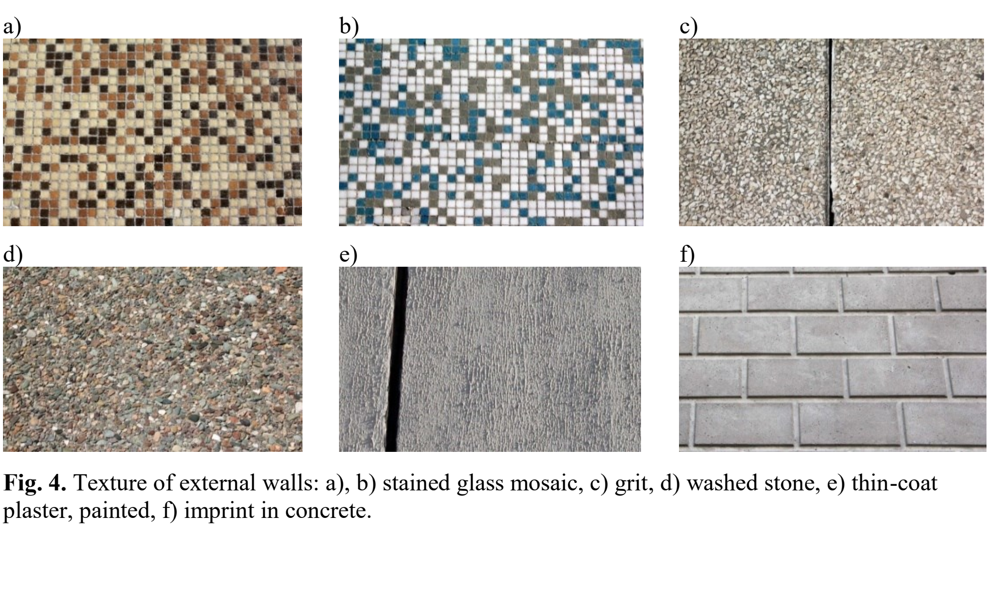

Fig. 4. Texture of external walls: a), b) stained glass mosaic, c) grit, d) washed stone, e) thin-coat plaster, painted, f) imprint in concrete.

3.2 Fixing of layers

Hangers Ø 8 mm and pins Ø 3 mm made of steel rods are responsible for the connection of the load-bearing layer with the external layer - in accordance with the assumptions of H13N4G9 grade (stainless steel and acid resistant, point weldable). The system provided for the use of both single and double-bent hangers (Fig. 5) - depending on the order of created layers and the local variety.

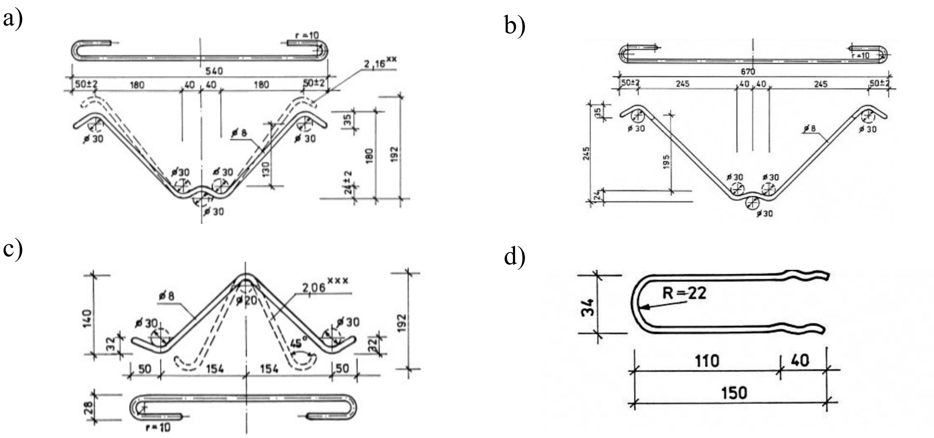

Fig. 5. Connectors used in three-layer walls: a) hanger type 2.16, b) hanger type 2.17 c) hanger type 2.06 (single-bent), d) pin [4].

Fig. 6 shows a diagram of the arrangement of hangers in the wall element depending on the type of element.

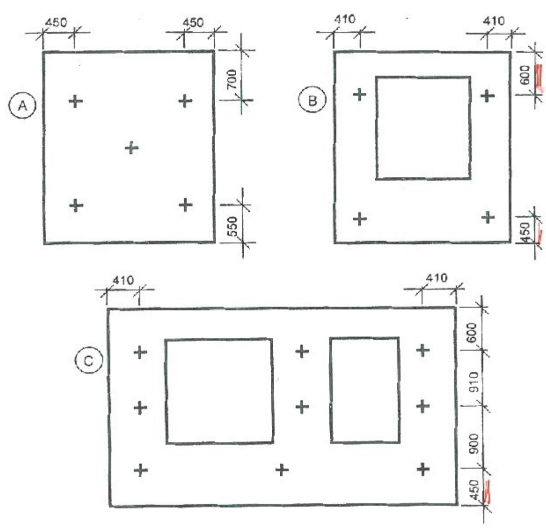

Fig. 6. Diagram of the arrangement of hangers in the walls of the Szczecin System [3].

4 Inventory of manufacturing and assembly defects and damages during the period of use

An inseparable element of technical diagnostics of industrial buildings is the macroscopic evaluation, which precedes material tests [7]. Rating covered all elevations of selected six five-storey buildings. The most frequent damages and their division are presented below:

- detached stained glass mosaic	1	D
- defects in the arrangement of the stained glass mosaic	1	W
- broken corners of the panels	1	W
- lintel damp patches	2	D
- uneven joints	2	W
- leaky joints	2	W, D
- sheared panel edges	2	W, D
- detached covering	3	D

Explanation of designations: 1 - occurring in most panels, 2 - occurring on individual panels of a given facade, 3 - occurring rarely (not more than two panels on a given building); and: W - defect or primary damage (production and assembly), D - degradation during exploitation.

In order to properly assess the condition of the panel, it would also be necessary to carry out an inventory of cracks as well as determine their type and width. However, in the case of stained glass mosaic finish and without access from the scaffolding, it is not possible to carry out a complete analysis of scratches. The main scratches are visible in the frames (delamination of the element) and in the corners of the panels.

The problem of leaking joints also needs to be emphasized - in many places, exposed joints as wide as 4-5 cm do not have a filling on the thickness of both the external layer and thermal insulation. Therefore, these are potential thermal bridges and places of rainwater penetration. Selected damages are visible in Fig. 7.

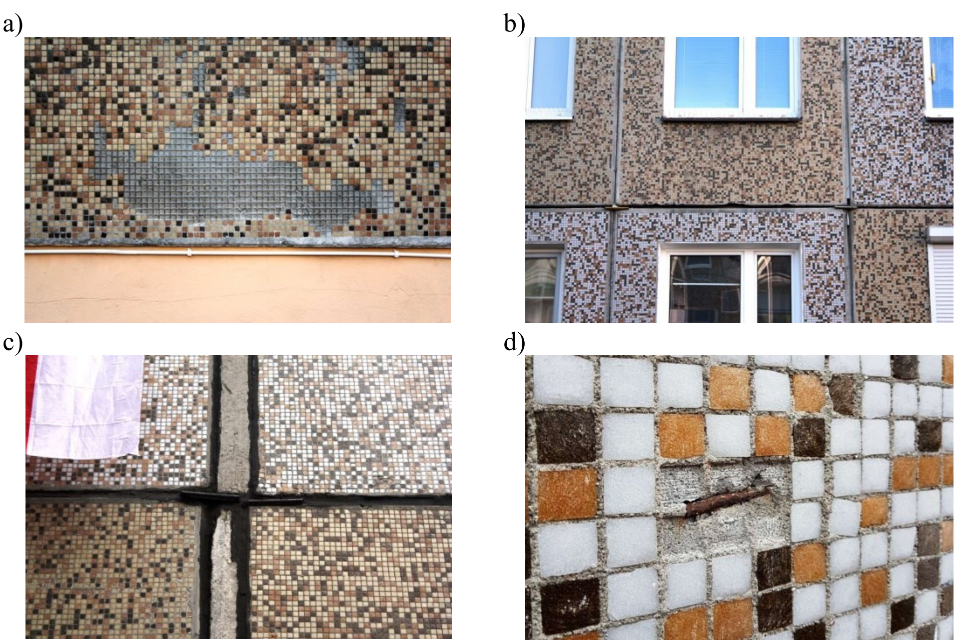

Fig. 7. Damage to the outer layer: a) detachment of the stained glass mosaic, b) chipping of the edges / leaking joints, c) lack of fitting of elements, d) detachment of the covering.

In 21 places the thickness of the external layer and the thermal insulation layer were measured - the results are presented in Table 2.

Table 2. Thicknesses of individual panels

No. of control boreholes	Thickness of external layer [cm]	Change in relation to the project [cm]	Thickness of the thermal insulation [cm]	Change in relation to the project [cm]	Total thickness (F+T) [cm]	Change in relation to the project [cm]
5/1	7.8	-0.2	4.8	- 1.2	12.6	-1.4
5/2	6.7	-1.3	5.3	- 0.7	12.0	-2.0
5/3	7.6	-0.4	5.5	- 0.5	13.1	-0.9
5/4	7.1	-0.9	6.3	+ 0.3	13.4	-0.6
5/5	7.8	-0.2	4.7	- 1.3	12.5	-1.5
5/6	8.8	+ 0.8	6.5	+ 0.5	15.3	+ 1.3
6/1	7.0	-1.0	5.2	- 0.8	12.2	-1.8
6/2	10.8	+ 2.8	6.1	+ 0.1	16.9	+ 2.9
6/3	9.0	+ 1.0	6.5	+ 0.5	15.5	+ 1.5
6/4	7.6	-0.4	5.5	- 0.5	13.1	-0.9
6/5	8.2	+ 0.2	6.0	ok	14.2	+ 0.2
6/6	8.8	+ 0.8	6.8	+ 0.8	15.6	+ 1.6
6/7	7.3	-0.7	6.1	+ 0.1	13.4	-0.6
6/8	9.0	+ 1.0	6.5	+ 0.5	15.5	+ 1.5
7/1	8.6	+ 0.6	6.5	+ 0.5	15.1	+ 1.1
7/2	8.9	+ 0.9	6.0	ok	14.9	+ 0.9
7/3	10.0	+ 2.0	6.9	+ 0.9	16.9	+ 2.9
7/4	10.3	+ 2.3	7.1	+ 1.1	17.4	+ 3.4
7/5	7.0	-1.0	6.5	+ 0.5	13.5	-0.5
7/6	6.7	-1.3	5.9	- 0.1	12.6	-1.4
7/7	7.7	-0.3	5.1	- 0.9	12.8	-1.2
design:	8.0		6.0		14.0	

The measurements indicate that the external layer is thickened in many panels in relation to the design assumptions - in extreme cases by as much as 2.8 cm. Interestingly, the total thickness of the external and thermal layer in many cases is greater than it should be (even by 3.4 cm). The thickness of thermal insulation in 8 cases is lower than assumed (38.1%), in two cases it is identical, and in 11 cases it is higher (maximum by 1.1 cm).

While the reduction in the thickness of thermal insulation is not surprising - it was most likely due to the deformation of the wool under the influence of the concrete mix weight during the formation of the element, the increase in the thickness of thermal insulation is. The probable reason is the mutual displacement of the element layers (bearing and external layers) under the influence of "hanging" the external layer and thermal and technological (shrinkage) stresses.

5 Depth and range of carbonation

The alkaline concrete reaction prevents the formation of rust. Over time, however, the blockade preventing iron anions from tearing off weakens through neutralization and carbonization of concrete (with the content of carbon dioxide contained in the air) [8]. It should be remembered, that the change of CO_2 diffusion in time, lead to a decrease ability of the carbonation, until it stabilizes [9]. Determination of the passivation state of the reinforcement steel of the external layer was done by determining the range of carbonation using phenolphthalein method - the test methodology is described in the standard [10].

The measurements were carried out on core boreholes taken from the elevation of the building. The diameter of the samples is 44 mm. The boreholes were split using a hydraulic press. The fracture surfaces were cleaned of loose particles under the influence of compressed air. The samples were laid down taking into account the position of the outer layer in one line. After the application of 2% phenolphthalein solution and after approximately 30 seconds, a measurement was taken. In order to better illustrate the range, the discoloration limit was indicated with a marker, several photos were taken showing changes on the surface of the rupture (Fig. 8).

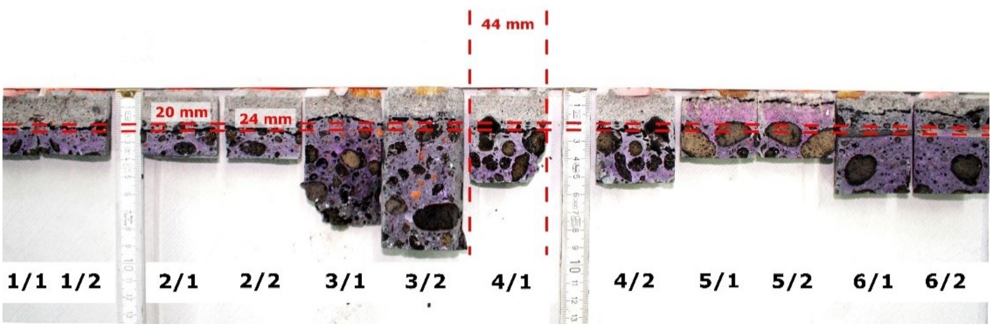

Fig. 8. Graphic analysis of the range of carbonation of concrete of the external layer

The maximum range of carbonation (24 mm) was measured and an attempt was made to determine the average range. The results for individual samples differ significantly, therefore the mathematical mean (19.9 mm) was assumed. Of course, the range of pH ≥ 11.8 (causing passivation of the reinforcement) will be smaller than the discoloured areas that indicate pH ≥ 9.0. The measured thicknesses of the covering (varying between 8 and 48 mm) as well as the 6/1 rebar (Fig. 9) visible on the sample at a depth of 22 mm allow us to believe that hangers and pins are exposed to corrosion initiation processes.

In addition, it is worth emphasizing that in many cases the carbonation range coincides with the colour change of concrete visible on the boreholes and in the holes (Fig. 9). Due to the fact that the slabs were made with an external layer downwards and the use of lightweight expanded clay aggregate, it was possible to segregate the concrete.

a) b)

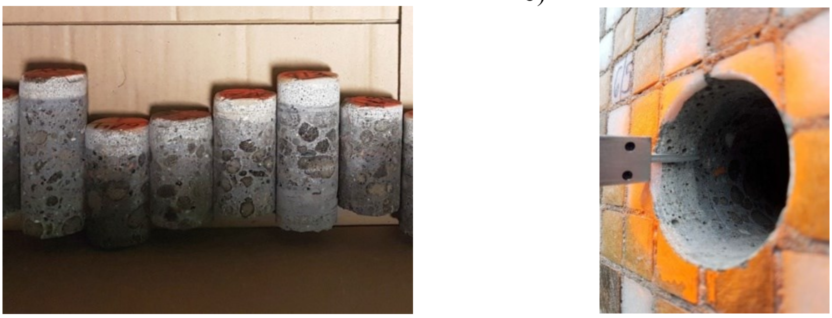

Fig. 9. Visible segregation of concrete mix: a) on boreholes b) in the hole

6 Compressive strength

In order to determine as precisely as possible the compressive strength of the external layer of concrete, core boreholes were taken from the structure. Two facades of each of the three buildings were selected for testing - in total 18 samples with a diameter of about 44 mm were taken. Such a small cross-section was chosen because of the attempt to maintain an appropriate proportion of the diameter and length of the sample (the design thickness of the element is 60 mm) and to minimize damage to the wall element and the façade.

The boreholes were drilled using a hammerless drill. In order to eliminate the risk of weakening the structure, the samples were cut from places which were devoid of steel hangers - the location was determined using the design assumptions of the system, assuming an appropriate margin of execution error.

Preparation of the samples was carried out in accordance with the recommendations of the applicable standard [11]. Deviations from the provisions of the standard were: borehole diameter, ratio of maximum aggregate size in concrete to core diameter greater than 1:3 (measured aggregate diameter $\leq$ 23 mm), as well as the ratio of length (l) to diameter (d) of the cylindrical sample, which was initially adopted close to 1:1. In order to mitigate the impact of the above deviations, the provisions of Appendix A to the standard were used [11]. The test was carried out on the Instron - SATEC testing machine.

After the first series of tests, it turned out that the measured compressive strength, reaching even 35.7 MPa, is significantly too high in relation to the design requirements (the assumed concrete class is B20) and material capabilities (expanded clay aggregate). The results were partially too high for the six tested samples, but also very divergent (10.1 - 35.7 MPa). It was decided to change the l/d ratio and to try to get close to the proposed l/d $\leq$ 2. Three samples were selected, the length of which fulfilled these assumptions and another series was carried out. The results are shown in the graph (Fig. 10). The maximum compressive stress of 26.5 MPa (before taking into account the correction factor) is still significant.

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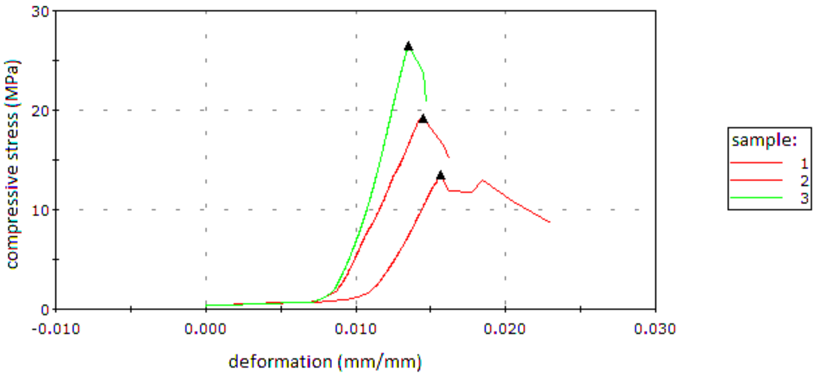

Fig. 10. Compressive strength curve of a series of samples l/d ≤ 2

On the basis of examples from literature [12] it was decided to conduct a third series of studies based on sandwich samples, the so-called sandwich samples. Due to the limited length of available samples it was the only possibility to achieve l/d ≤ 2 ratio.

According to the assumptions, the sample was glued with vikol wood adhesive and after the binding time declared by the manufacturer, another series of tests were carried out. Eight cylindrical specimens l/d = 1 were used to obtain 4 specimens meeting l/d ≤ 2 ratio condition. In Table 3 the results of compressive strength after the series with original specimens l/d ≤ 2 and glued specimens (sandwich) are presented. A graph of the sandwich samples is shown in Fig. 11.

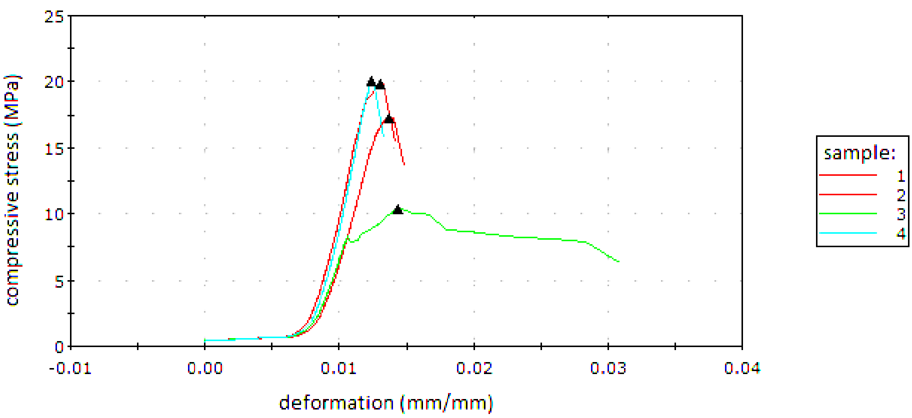

Fig. 11. Compressive strength curve of a series of sandwich samples

Table 3. The results of the core boreholes strength test.

	Sample label	Weight	Diameter	Height of anvil	Maximum compressive load	Maximum compressive stress	Stress after correction (appendix A of the standard)	Comments
		(g)	(mm)	(mm)	(kN)	(MPa)	(MPa)	
1	l/d □ 2 - 6.4	175.8	44	77	20.5	**13.5**	**14.4**	
2	l/d □ 2 - 6.3	209.6	44	89	29.2	**19.2**	**20.6**	
3	l/d □ 2 - 7.4	202	44	90	40.3	**26.5**	**28.4**	
4	sandwich – 5.6-6.6	182	44	89	26.2	**17.2**	**18.4**	
5	sandwich – 6.1-6.2	187	44	88	30.2	**19.8**	**21.2**	
6	sandwich – 7.1-7.2	~~170~~	~~44~~	~~88~~	~~15.8~~	**~~10.4~~**	**~~11.1~~**	uneven sample base
7	sandwich – 7.5-7.6	198.6	44	88	30.6	**20.1**	**21.5**	
	Standard deviation:	12.96	0.00	4.88	6.5	4.3	**4.6**	
	Mean	192.50	44.00	86.83	29.5	19.4	**20.8**	
	Minimum:	175.80	44.00	77.00	20.5	13.5	**14.4**	
	Maximum	209.60	44.00	90.00	40.3	26.5	**28.4**	
	Range	33.80	0.00	13.00	19.8	13.1	**14.0**	

The results clearly indicate that the l/d ratio of the sample has a very big influence on the results of the expanded clay concrete tests. The average strength after correction due to aggregate size and reduced sample diameter is 20.8 MPa. In order to determine the class of concrete, the standard about evaluation of concrete strength were used [13]. The characteristic compressive strength of concrete in the structure is the smaller of the two following values:

$$f_{\mathrm{ck,is}} = f_{\mathrm{m(n),is}} - \mathrm{k} \quad (1)$$

$$f_{\mathrm{ck,is}} = f_{\mathrm{is,lowest}} + 4 \quad (2)$$

The “k” depends on the number of results (in our case k = 7). The next values are: $f_{\mathrm{m(n),is}}$ = 20.8 MPa, $f_{\mathrm{is,lowest}}$ = 14.4 MPa. The smaller of the above values is 13.8 MPa. The designed concrete class is B20 (according to PN-B 06250: 1988 [14]), so the required $f_{\mathrm{ck,is,cube}}$ = 17 MPa. Due to the above, for the first condition (1), the concrete strength is lower than the assumed one (low number of samples affects the low result). For the second condition (2), the strength is 18.8 MPa and meet the requirements of class B20. The large differences in strength for individual specimens are worrying, but it can be assumed that this is directly related to concrete segregation and largely depends on the arrangement of the expanded clay aggregate.

7. Reinforcing steel

Approximate location and covering of the external layer reinforcement was determined using Ferroscan PS200 ferromagnetic device from Hilti. The measurements were carried out in accordance with the device manual [15].

Both 60x180 cm blocks and 60x60 cm images were scanned in order to determine the characteristic reinforcement points, including the location and anchoring of the steel hanger (Fig. 12).

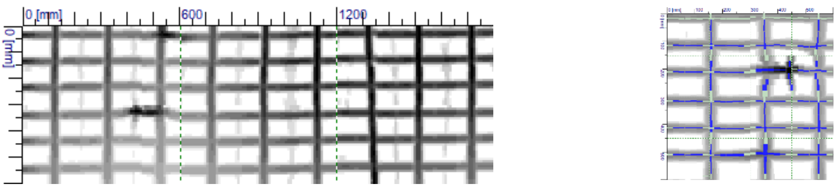

Fig. 12. Scanned block of 60x180 cm and a picture of 60x60 cm with a visible hanger.

The analysis of scanned images indicates the use (in the external layer) of reinforcement grid with equal spacing of 10x20 cm (laid horizontally or vertically) and the diameter of reinforcement rods of 4.5 mm (samples of rods were taken). The diameter of the hanger could not be verified. Samples of reinforcement bars confirm the use of smooth steel A0.

The 60x60 peel in Fig. 13 shows the element of the hanger and its shift in relation to the design assumptions - the measurement field was arranged in such a way that - taking into account the design assumptions - the hanger was in the middle. The shift is therefore 15 cm horizontally and 10 cm vertically.

Next, the thickness of the covering was measured in detail along the entire length of the selected gable wall (Fig. 13), i.e. 11.80 m. The horizontal and vertical displacement of the hanger is 15 cm.

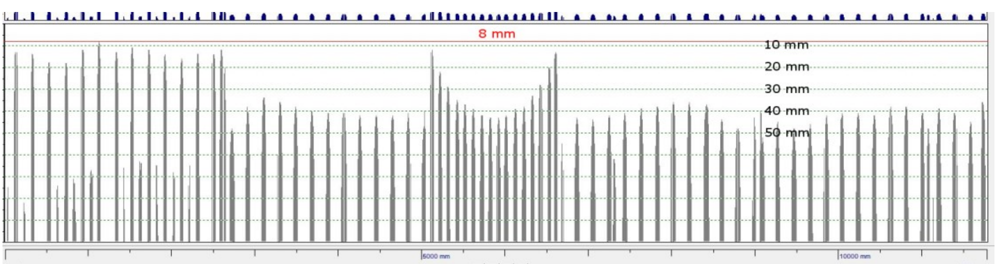

Fig. 13. Measurement of covering thickness over the length of the gable wall (11.8 m)

In the drawing it is possible to unambiguously separate five individual façade panels, it is also visible that the central one has a 10x20 cm vertical reinforcement grid and the rest of the panels a horizontal one. The covering on the measured elements varies between 8-48 mm (design assumptions are 15 mm). Undoubtedly, the reinforcement nets moved during concreting, probably due to incorrect fixing.

8 Conclusions

The presented results of the inventory of the concrete condition of the external layer indicate clear discrepancies between the layer thickness and the design assumptions. The same applies to the thickness of the insulation layer. The mentioned deviations are unfavourable, because a significant increase in the total thickness results in an increase in the weight of the panel and in the moment of force, which worsens the working conditions of steel hangers. Also worrying is the increase in thermal insulation thickness compared to the design assumptions, which may indicate the mutual displacement of the wall element layers.

A significant part of the panels has damages to the finishing layer, stained glass. Tests of carbonation range confirm the lack of proper protection of reinforcing steel against corrosion. However, the steel samples taken did not show any signs of progressive corrosion (only an overlap, which could have occurred at the stage of element production).

The obtained results of concrete compressive strength are favourable and in case of most of the samples confirm the design assumptions. In addition, the lack of areas of significantly thinned external layer (occurring in other systems) and the lack of clear scratches allows to believe that the element will remain stable for the next years. Only the discrepancies for the slabs may be worrying - they probably result from the segregation of the concrete.

As a result, the research has also shown how important it is to correlate individual dimensions of light concrete core boreholes in order to obtain real results. In the strength test, the method of gluing samples, which was not originally assumed, was used. Sandwich samples seem to be an effective solution for testing the strength of concrete for the specifics of the external layer element (thicknesses usually below 80 mm).

Therefore, actions should be planned to strengthen the anchoring of the external layer and at least to protect the current damage and to supplement the thermal insulation and waterproofing in the joints. This will protect the curtain walls against further degradation caused by atmospheric factors.

It should also be emphasized that the methodology of diagnosing external walls presented in the article may be disseminated in the assessment and evaluation of objects both in Poland and in other countries.

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