Maintenance and Durability of the Concrete External Layer of Curtain Walls in Prefabricated Technological Poznan Large Panel System

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Abstract. The issue of usability and durability of large-panel building constructed several decades ago is a subject of an in-depth analysis of many domestic and foreign investments. When considering the durability of specific large-panel system, one should consider, among others, the process of executing external walls. The long-term and direct impact of weather conditions on the external layer of curtain walls is significant for the durability of large-panel buildings. For the needs of the presented paper, in 2016, the survey of cracks and a series of other tests of large-panel façade, residential building constructed in 1986, in Poland, in the PLP process system - Rataje was executed. Several hundred large-size, triple-layer curtain-wall slab with a 6-cm, concrete exterior cladding layer anchored using pins and hangers with the load-bearing layer, a 9-cm insulation layer made of mineral wool, and a 21-cm structural layer were surveyed. Significant deviations in thicknesses of particular wall layers were proven. Other significant damages and defects of external layers were found. At the second stage, many tests, both non-destructive and destructive, were conducted. They involved determining mechanical properties of an external layer. The concrete thickness was measured using a type N Schmidt sclerometer and core samples were taken from this layer in order to mark concrete’s compressive strength. The range of carbonation (by phenolphthalein method) and the actual location and condition of reinforcement were estimated using a ferromagnetic device to determine the condition of the external layer. The diagnosis conducted in such a manner was the verification of necessary repair of the walls and their thermal efficiency improvement while ensuring safe conditions of their operation and modern functional and utility requirements. It should be also emphasized that the method of diagnosing the external walls presented in this paper may be popularized when evaluating such facilities both in Poland and other countries of the Central Europe (Germany, Czechia, Slovakia, Lithuania, Bulgaria, and Ukraine).

1. Introduction
The issue of usability and durability of large-panel buildings constructed several decades ago is a subject of an in-depth analysis of many domestic and foreign institutions [1,2,3]. When considering the durability of specific large-panel systems, one should consider, among others, the process of executing external walls as well. They are one of the most important elements of the building deciding about its reliability by taking over significant loads from weather impacts. The external layer of an external wall, which is analysed in the paper, contrary to the load-bearing layer, is characterised by difficult, various operating conditions. The external layer is a curtain of a thermal insulation and a load-bearing layer (both mechanical and protecting against weather impacts) as well as it protects steel connectors in the
form of hangers and pins against corrosion [4]. The reliability of the structural layer depends directly on it. That is why the good quality of the external layer's concrete and its resistance to carbonisation, to the possible extent, is required. Several dozen years of operating, assembly errors, production negligence [5, 6], and lack of ongoing maintenance [7] allow to formulate a hypothesis that the curtain walls do not have proper stability and pose a threat to people and property. Expert’s opinions and tests conducted in different places of Poland confirm this [8, 9]. The tests presented in the following paper were conducted in a 13-floor large-panel building after 30 years of operation (1986-2016) in the PLP system. Figure 1 shows the horizontal and vertical section through the curtain wall used in the facility. The projected thicknesses of particular layers are respectively: 6, 9, and 21 cm (Figure 1).

Figure 1. Sections through a triple-layer curtain wall [10]

In order to recognise correctly the condition of external layer's concrete, both non-destructive (in situ) and destructive (laboratory tests of taken cored boreholes) methods were used.

2. Characteristics of the building
The studied facility was commissioned in 1986. It was constructed in the large-panel process, in the modernised R-76 process system. It is a version of the PLP – Rataje system, which was created after introduction of the Design Technical Standard (in Polish: Normatyw Techniczny Projektowania) in 1974 (NTP-74) and a new heat standard in the early 1980s [11]. The building has 13 overground floors (2 service and economic and 11 residential ones) and according to the applicable technical conditions [12], it should be included in the group of “high” (in Polish “wysoki” meaning “high”) buildings. The floors 3-13 are repetitive [9]. Table 1 shows mainly technical data on the building. Figure 2 shows the façade with extra loggias, which is currently subjected to modernisation. The described tests were also conducted on it. Figure 3 shows a detail in the form of an external wall's texture.

3. Survey of the external layer's condition
The survey of the façade and further tests were coordinated with the conducted works on thermal efficiency improvement. Therefore, the area of interest was restricted to the fragment of a wall with extra loggias and an end wall with two rows of panels on the frontal façade (Figure 4).
Table 1. Basic parameters of the building acc. to [9].

| Parameter                              | Value                                      |
|----------------------------------------|--------------------------------------------|
| Construction year:                     | 1986                                       |
| Height:                                | av. 36.80 m                                |
| Dimensions (l/w):                      | ~395.65 m x 11.5 m                        |
| Floor space:                           | 35,984.64 m2                               |
| Number of apartments:                  | 551                                        |
| Number of staircases/entrances:        | 25                                         |
| Structure:                             | transverse and longitudinal                |
| Spacing of load-bearing walls:         | 4.8 m and 2.4 m                            |
| External walls of the building:        | self-supporting, triple-layer (apart from walls of hoistways) |

![Figure 2. Façade with loggias](image)

![Figure 3. Texture of curtain wall](image)

![Figure 4. Illustrative photo – top view of the building. Background: [13]](image)

![Figure 5. Section of the façade fragment with panels divided into sections. Design: [9]](image)

The façade panels, for the needs of the survey, were divided into 30 sections along the building. Each section consisted of 11 panels of particular floors (Figure 5).

In total, 224 panels were surveyed, 85 panels had their external layer thickness measured after executing boreholes for mounting mechanical anchors (within modernisation works) – thickness measurement on each panel was executed in 2, 3 or 4 points (depending on the number of projected anchors). Independently, on all tested panels, the total thickness of the external layer and lagging was measured. Table 2 shows the summary of the survey results.
The end wall has definitely the greatest number of scratched panels – their share in that place is almost 80%. Local scratches are visible (or dispersed on the panel surface – Figure 6) as often as the scratches going through the entire width or height of the component (Figure 7). Scratchings of corners and edges also occur.

**Table 2. Summary of defects of and damages to the external layer.**

| Damage or defect:                                           | Number of panels with irregularities: | Number of checked panels: | Percentage share: |
|-------------------------------------------------------------|--------------------------------------|---------------------------|-------------------|
| visible scratchings, cracks                                 | 41                                   | 224                       | 18.3%             |
| visibly washed, falling grit                                | 25                                   | 180                       | 13.9%             |
| thickness of the external layer and lagging > 16 cm          | 59                                   | 224                       | 26.3%             |
| visible hangers, pins (detached covering)                  | 12                                   | 224                       | 5.4%              |

Another significant defect is visibly detaching grit, which is the finish of the external layer of the curtain walls. Almost 14% of panels has significant damages in this regard. It should be mentioned that grit covers only panels on the longitudinal wall. The end walls’ finish is a painting layer (Figs. 6 and 7).

Significant deviations from the design assumptions were found also by measuring thickness of the external layer and the external layer and lagging in total. The thickness of the external layer for measured panels is from 4 cm to even 12 cm. For over 26% of panels, the total thickness for two layers is >16 cm. The listed deviations are adverse because the significant increase in the total thickness causes increase in the panel mass and the acting moment, which worsens the operation conditions of the steel hangers. Whereas the panel thickness of 4-5 cm is non-compliant with the requirements of the building construction (required min. 60 mm) [14] and may lead to significant decrease in the component durability (Figures 8, 9). The last verified damage was detaching of the covering of hangers and pins, which may fasten the corrosion processes. The irregularities were found in the case of c. 5.4% of panels. The measured thicknesses of the covering in the detaching point varied in the range from 5 to 15 mm that is also below minimum requirements.

However, it must be added that traces of corrosion (even of the surface one) were not found on any of the visible hanger rods. Among the pins, several of them were visibly corroded. The measurement of the hanger fragments confirms the application of the A0 steel with the diameter of φ10 mm (Figs. 10,11).
4. Thickness of carbonisation, SEM and EDS analyses of the external layer

In order to mark the range of carbonation in the hardened concrete of the external layer, the phenolphthalein method was used – the standard [15] describes the tests' methodology. The measurements were conducted on the cored boreholes taken from the building's façade. The samples diameter is c. 44 mm, whereas the length depends on the external layer thickness. The boreholes were split using a hydraulic press. The split surfaces were cleaned of loose particles under compressed air. After applying 2% solution of phenolphthalein and lapse of c. 30 seconds, photos showing the change of colour in particular split areas were taken (Figure 12).

The maximum range of carbonation (18 mm) was measured. The attempt to determine the average range was also undertaken. Due to an uneven surface of the external layer and a significant share of grit in the surface layer, it is difficult to determine the depth precisely. However, it can be evaluated as c. 15-16 mm. Of course, the range of pH ≥ 11.8 (causing passivation of reinforcement) will be smaller than the coloured areas, which show pH ≥ 9.0. The covering thicknesses measured in several places fluctuating between 5 and 15 mm allow to believe that both the pins and the hangers are exposed to initiation of the corrosion processes.

By using the SEM and EDS, micro-areas of the external layer for the concrete deterioration resulting from corrosive impacts of the environment for over 30 years were analysed. Figure 13 shows a micrograph of the concrete surface, and Figure 14 shows the phase composition of the concrete in terms of contact with the atmosphere and in the layer placed 9 cm from the surface.

The figures show the concrete deterioration. Figure 4 shows the ageing of the surface layer involving leaching the elements of Ca and Si (basic ingredients of concrete) increase in the content of elements of C, S and K (carbonation, acid rains).
5. Concrete hardness

The condition of the concrete of the external layers must also be determined using strength parameters, such as hardness and compressive strength. By conducting non-destructive sclerometric tests using a Schmidt sclerometer, both the first parameter and, indirectly, the other one can be determined.

In the test, an N type device was used, whereas the standard [16] was used methodologically. The tested concrete of the external layer was in the air-dry state, and the temperature during operation was 20-25°C. The surface of the component was prepared for the measurement every time by splitting the surface layer of grit as well as grinding and smoothing with a wire end. Every time, a hammer was placed perpendicularly to the tested surface. On each component, 9-12 measurements were conducted in 6 places (Figs. 15, 16).
Table 3 summarises the calculated characteristic strength $f_{ck}$ for 13 tested panels of the external layer. The results were generated in the "Młotek Schmidta N [in Polish: N Schmidt hammer] \EU PRO v.3.80" software.

**Table 3.** Characteristics strength of concrete of the external layer calculated on the basis of the L rebound number

| Component/panel: | G2.8 | G1.9 | G1.11 | A4.8 | A1.9 | A2.11 | B1.11 | A3.11 | B1.9 | I1.10 |
|------------------|------|------|-------|------|------|-------|-------|-------|------|-------|
| $f_{ck}$ [MPa]   | 13.1 | 18.5 | 24.1  | 23.0 | 11.1 | 12.1  | 20.9  | 19.7  | 8.6  | 11.5  |

Significant differences in the measurement may result from conducting the test on the component that was not rigid enough. According to the recommendations, the minimum thickness of the component should be 100 mm. Moreover, the specifics of the external layer involve its suspending from the load-bearing layer, which additionally lessens the rigidity. Hence, the part of the ram's energy was used for component vibrations.

6. Compressive strength

In order to precisely determine the compressive strength of the external layer concrete, the cored boreholes were taken from the structure. Five façade panels were selected for the test. Three boreholes of the diameter of c. 43-44 mm were taken from each one. The boreholes were performed using a non-impact drill. In order to eliminate a threat caused by the structure weakening, the samples were cut after inserting the K2-type steel chemical anchors (by the Inwestbud company). The manager of the building planned the assembly of the reinforcing connectors within the conducted thermal efficiency improvement.

The samples were prepared according to the recommendations of the applicable standard [17]. The departures from the standard provisions were: borehole diameter, ratio of maximum aggregate dimension in the concrete to the core diameter greater than 1:3 (measured diameter of aggregate ≤ 20 mm) and ratio of the length to the diameter of the cylindrical sample – almost 1:1. In order to overcome the impact of the above departures, provisions of the Annex A to the standard [17] were applied. In order to avoid fragment of steel rods in the taken samples, the location of the reinforcement was verified using a ferromagnetic device.

Table 4 summarises the averaged results of compressive strength after conducting a test on a universal testing machine by Instron – SATEC. The results shows divergences in strength of the concrete of the external layer reaching 76%.
### Table 4. Results of testing the core boreholes

| Sample label | Average compression stress [MPa] | Stress after including correction (Annex A of the standard) [MPa] | Comments |
|--------------|---------------------------------|-------------------------------------------------------------|----------|
| 1D-(1-3)     | 28.9                            | 31.8                                                        | one sample not included |
| 2D-(1-3)     | 20.9                            | 22.9                                                        | one sample not included |
| 3D-(1-3)     | 26.6                            | 29.2                                                        | -         |
| 4D-(1-3)     | 36.7                            | 40.4                                                        | -         |
| 5D-(1-3)     | 29.6                            | 32.5                                                        | -         |

7. Reinforcement location

By using the Ferroscan PS200 ferromagnetic device by Hilti, an approximate location, a diameter, and a covering of the external layer's reinforcement (Figure 17) were specified. The measurement were conducted in accordance with the device's user manual [18].

![Figure 17. Scanning the reinforcement with the Ferroscan PS200 device](image)

In total, three full, 180x180 cm blocks (Figure 18) and ten 60x60 cm images were scanned in order to determine the reinforcement's characteristic points – among others, placement and anchoring of the steel hanger (Figure 19).

The analysis of the scanned images shows the application (in the external layer) a reinforcement mesh with even two-side spacing from 20 to 22 cm and the diameter of the rebars of 6-8 mm. Locally, the measurement shows the diameter of the bars of φ10 mm but it may be a device's measurement error. The taken sample of the rebar confirms the use of smooth steel A0 with the diameter of 6 mm. The covering of the reinforcement mesh on the measured components fluctuates in the range of 27-51 mm. In single places, the covering is min. 22 mm, which seems to be a favourable result anyway. Among measurements, there are also the ones showing significant, unacceptable deviations in the rebars arrangement (Figure 20) – on the length of 120 cm, the axial spacing of the horizontal bars changes from 13 cm to 27 cm.

8. Adopted repair method

The basic assumption of the conducted investments is improving the comfort of use by improving thermal insulation of external partitions and improving the facade's aesthetics. Reinforcement and repair of the structural system was emphasised as well. It was decided to apply the following repair and modernisation methods:

- lagging process – the light wet method "ETICS,"
- chemical anchoring of particular layers of triple-layer external walls with K2 steel connectors.
- secondary filling the joints of prefabricated components with polyurethane sealant,
- regarding the damaged balcony loggias' panels, patching holes and scratches with the PCC II/III type repair mortar.

From the point of view of the paper's nature, the description of the works using steel anchoring connectors must be extended. The K2 adhesive anchor system (Inwestbud type) is applied on the basis of the issued technical approval [19]. Figure 21 shows the connector scheme.

A steel stem and a pressure screw are made of steel resistant to corrosion, at least of A4 grade. The connectors must be placed using vinyl-ester or epoxy-acrylic resin. The maximum length of the produced anchors is 210 mm; the applied diameter is 20 mm. For the discussed facility, the conditions – listed in the approval – of the minimum anchoring length amounting to 50 mm (at thickness of the load-bearing layer ≥ 60 mm) were met. According to the approval, the calculated load-bearing capacity of the connection for the thickness of the external and insulation layer of 60 mm each (and at the permissible movement volume of 3 mm) is $V_{sd} = 10.9$ kN. The minimum steel grade of the structure and external layers cannot be lower than C12/15. In addition, the concrete surrounded by the executed reinforcements must be free of scratches and cracks. Because the works were executed in summer, attention must be paid to the permissible temperatures of applying resins and proper gelation times.
9. Conclusions
The presented results of survey of the condition of the concrete external wall shows visible divergences in thickness of the layer in relation to the design assumptions. The same concerns the lagging layer's thickness. The significant part of the panels has a washed finishing layer in the form of white grit. There is also a problem with uncovered steel hangers after detaching of the covering. The studies on the carbonation ranges confirm the lack of proper protection of reinforcing steel against corrosion. However, the obtained results of concrete's compressive strength are favourable. Only divergences for the panels variously located on the building's façade may raise concerns.

Therefore, actions aimed at reinforcing the external wall's anchoring and improving thermal efficiency of the external partitions seem to be necessary. These will additionally protect the curtain walls against further deterioration caused by the impact of weather conditions. However, the proposed chemical anchoring system may not be applied uncritically. Particular attention must be paid to the places, where the total thickness of the external and load-bearing layers exceeds 16 cm (in accordance with Table 2, over 26% of the tested panels). Due to minimum depth of anchoring and the maximum available length of the anchors, in the selected places, other connector system must be applied. Several verified panels did not meet the condition of minimum concrete grade – before executing the reinforcement, complementary tests must be conducted. The location of connectors in the scratched areas of the panels should be avoided as well (Figure 7).

It should also be emphasised that the method of diagnosing the external walls presented in this paper may be popularised when evaluating other facilities both in Poland and other countries of the Central Europe (Germany, Czech republic, Slovakia, Bulgaria, Ukraine), where the monoculture of the large panel constituted 40% to 70% of the commissioned residential buildings executed in multifamily residential development.

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