Modern Environmental Science and Engineering (ISSN 2333-2581)
February 2018, Volume 4, No. 2, pp. 173-181
Doi: 10.15341/mese(2333-2581)/02.04.2018/008
Academic Star Publishing Company, 2018
www.academicstar.us

Field Testing of Concrete External Layer of Curtain in Poznan Large Panel System after 30 Years of Environmental Influences

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Abstract: A large number of residential buildings in prefabricated large-panel technology were built in Poland and Europe in the 60-90s of the XXth century. It is a common claim that these objects, in the coming years, will have to be demolished due to their bad technical condition. However, a scope of that venture exceeds economic possibilities and creates significant environmental costs thus forcing us to look for other solutions.

The long-term and direct impact of the external environment on the outer layer of curtain walls is significant for the durability of large-panel buildings. The authors of this article decided to conduct field testing of 30 years old concrete external layer of walls in prefabricated technological Poznan Large Panel System on the aspect of theirs durability.

In 2016, the survey of cracks and a series of other tests of large-panel façade, residential building constructed in 1986 in Poland was executed. Several hundred large-size, triple-layer curtain-wall slabs were surveyed. Significant damages and defects of external layer were found. Other significant deviations in thicknesses of particular wall layers were proven.

At the second stage, many tests, both non-destructive and destructive, were conducted. They involved determining mechanical properties of an external layer. The concrete hardness was measured using with 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.

Key words: large-panel buildings, environment influences, carbonation, Poznan Large Panel System, durability and maintenance

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 Polish and European institutions [1-3]. 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, environmental impacts. The external layer of an external wall, which is analyzed 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. For this reason it is required that the concrete of the external layer is of good quality, highly airtight as well as resistant to carbonisation.

Several dozen years of operating, assembly errors, production negligence [5, 6], and lack of ongoing

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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. Fig. 1 shows the horizontal and vertical section through the curtain wall used in the facility. The projected thicknesses of particular layers are respectively: 6 cm (concrete exterior cladding layer anchored using pins and hangers with the load-bearing layer), 9 cm (insulation layer made of mineral wool), and 21 cm (structural layer).

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.

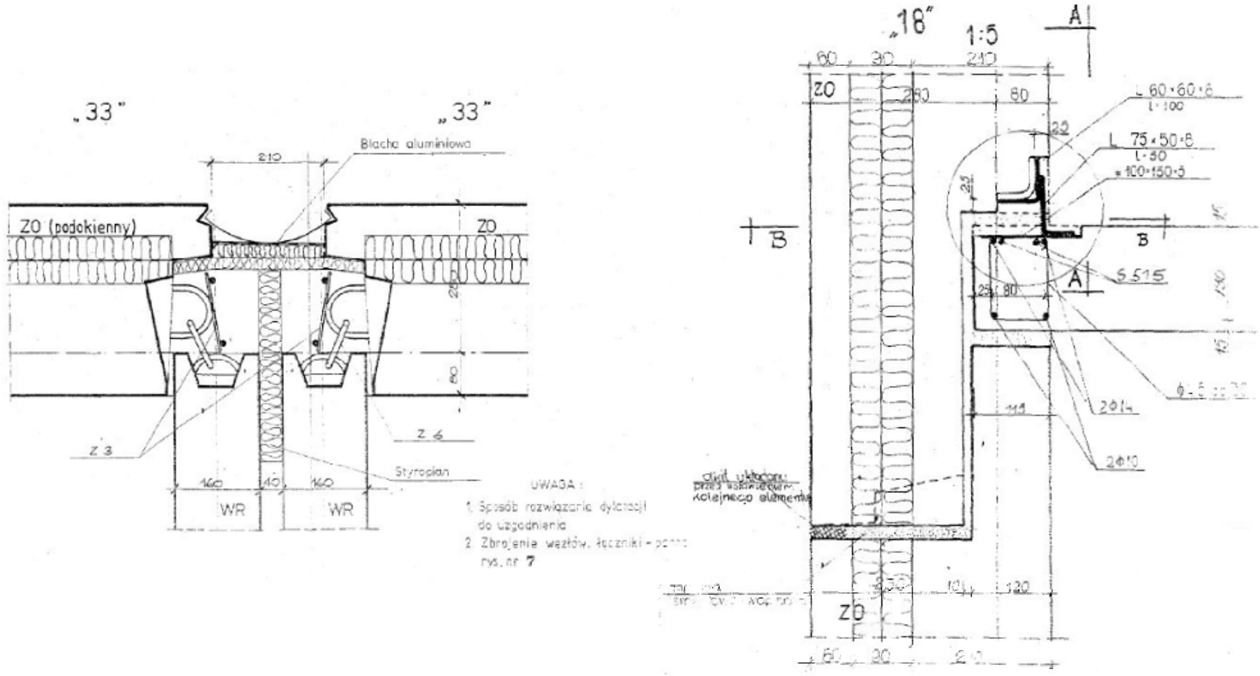

Fig. 1 Sections through a triple-layer curtain wall [10].

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 Rataje system (Poznan Large Panel System), which was created after introduction of the Design Technical Standard in 1974 (NTP-74) and a new heat standard in the early 1980s [11]. The building has 13 above-grounds 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 classification) buildings. The floors 3-13 are repetitive [9]. Fig. 2 shows the façade with extra loggias, which is currently subjected to modernisation. The described tests were also conducted on it. Table 1 shows mainly technical data on the building.

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 (Fig. 3).

Fig. 2 Façade with loggias.

Table 1 Basic parameters of the building [9].

construction year:	1986
height:	av. 36.80 m
dimensions (l/w):	~395.65 m × 11.5 m
floor space:	35,984.64 m^2
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 (except for walls of elevators)

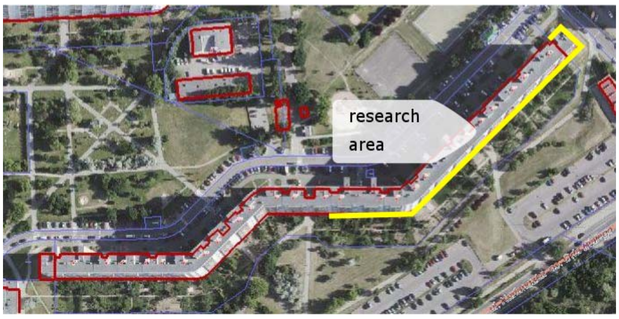

Fig. 3 Illustrative photo — top view of the building [13].

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 (Fig. 4).

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.

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 — Fig. 5) as often as the scratches going through the entire width or height of the component (Fig. 6). Scratchings of corners and edges also occur.

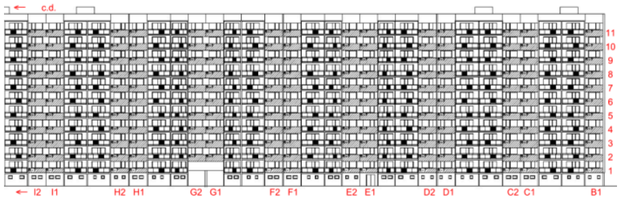

Fig. 4 Section of the façade fragment with panels divided into sections [9].

Fig. 5 Density of local scratches.

Fig. 6 Horizontal scratch, thickness of about 0.5 mm.

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 (Fig. 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 (Fig. 8).

Fig. 7 Degraded cardboard layer.

Fig. 8 Variation of the layer thickness.

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 (Fig. 9). Table 2 shows the summary of the survey results.

Fig. 9 Visible steel pin.

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%*

4. Thickness of Carbonization, 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 (Fig. 10).

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, microareas of the external layer for the concrete deterioration resulting from corrosive impacts of the environment for over 30 years were analysed. Fig. 11 shows a micrograph of the concrete surface, and Fig. 12 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. Fig. 12 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).

Fig. 10 Visualized analysis of the carbonation range of the external layer's concrete.

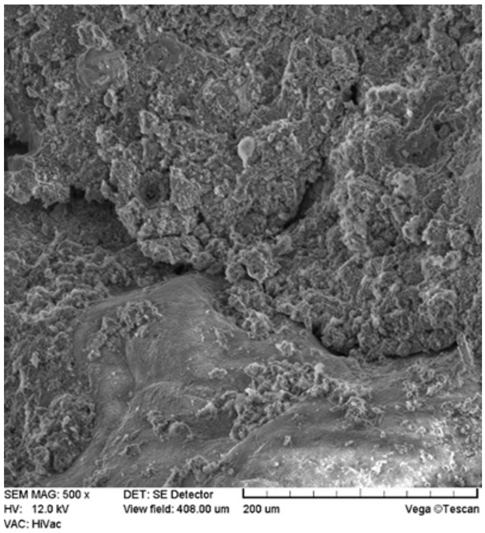

Fig. 11 Texture of surface layer— concrete deterioration.

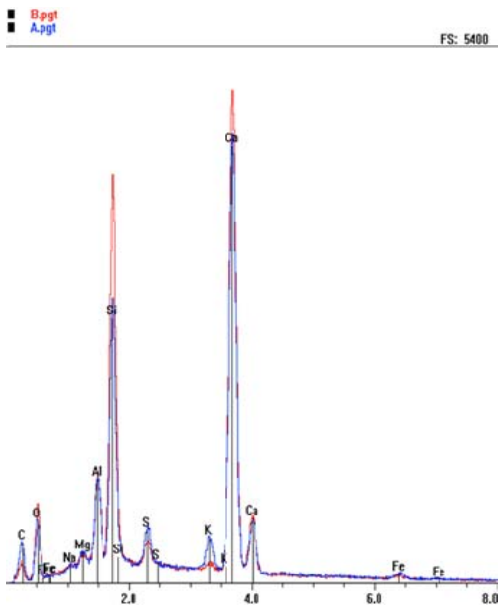

Fig. 12 Differences in XRD of the external (blue chart) and bottom layer (red chart).

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 (Fig. 13). Table 3 summarises the calculated characteristic strength f_{ck} for 13 tested panels of the external layer. The results were generated in the N Schmidt hammer]\EU PRO v.3.80 software. 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.

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

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

Fig. 13 Measurement of the L rebound number on a smooth surface of an end wall.

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 show 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 (Fig. 14) were specified. The measurement was conducted in accordance with the device's user manual [18].

In total, three full, 180×180 cm blocks and ten 60×60 cm images were scanned in order to determine the reinforcement's characteristic points — among others, placement and anchoring of the steel hanger (Fig. 15).

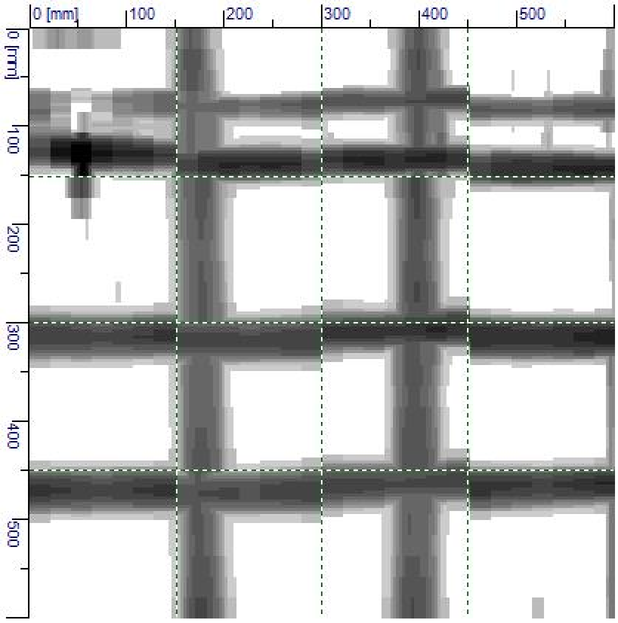

Fig. 15 Scan picture : 60× 60 cm image with a visible steel hanger (left top corner).

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 — on the length of 120 cm, the axial spacing of the horizontal bars changes from 13 cm to 27 cm.

Fig. 14 Scanning the reinforcement with the Ferroscan PS200 device.

8. Conclusions

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 (Fig. 6).

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. A diagnosis conducted in such manner was essential for the initiation of wall repairs and improvement of their thermal efficiency, at the same time ensuring safe conditions of their operation and modern functional and utility requirements.

Acknowledgement

Elaboration within the framework of project No. 01/11/DSPB/002 financed by MNiSW

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