

Prefabricated RM Façade Panels – Search for the Safe Solution

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Abstract. The problem, which appeared during the design works on the untypical masonry openwork of the front elevation of the academic building in Poland was presented and discussed in this paper. The original solution of masonry external façade was too risky and practically impossible for realization from the workmanship point of view. For this reason authors were proposed to make this elevation wall as prefabricated construction consisted of medium scale prefabricated elevation panels made of openwork clinker units and masonry joints with reinforcement. Two solutions of prefabricated panels were elaborated: first by the design office and second one, significantly modified, proposed by the authors. Taking into consideration fact that proposed prefabricated panels are not the typical reinforced masonry possible to design based on Eurocode 6, the methodology of “supporting design by test” was accepted to verify the correctness of proposed solutions. The carried out tests of both types of prefabricated panels with results and their discussion are also presented here. The results have shown the lack of safety for the first type of prefabricates and good behaviour, safety and durability of the final, modified solution.

1. Introduction

For years masonry external walls of buildings were built as typical solid single layer structures, external and internal plastering. With the passing of time, the development of building materials and technologies allowed for the construction of more complex external masonry walls, like two or three leaf walls, cavity or diaphragms' walls or enclosure walls, where façade leaf is not the structural component only architectural element. Such solution, from structural point of view, has some advantages. Internal part of the wall is usually the load-bearing element guarantee the adequate resistance to transmitting the internal stresses, produced by gravity and imposed loads. Addition layer of thermal insulation protects the interior of the building against the environmental influences. Whereas the external leaf (façade) can be the external finishing of the wall and protective layer for the thermal insulation (in case of cavity wall) or can be use as typical architectural, decorative external part of the building, sometimes not structurally connected with other parts of the external wall [1].

Nowadays it is popular and common to make the external layer of a triple-layer enclosure masonry wall using clinker bricks. Clinker layer, except for its aesthetic value, in case of using solid or vertically perforated bricks is characterized by high durability and resistance against any environmental impacts. Moreover, clinker brick does not fade in time and neither algae nor moulds grow on its surface. Solid natural colour after firing and possibility of glazing are the most important features used to obtain the desired aesthetical architectural look of the façades characteristic only for ceramic materials [2]. Furthermore, the ceramics – being a natural material – provide proper micro-climate inside and increase thermal insulation properties of the building.



Properly formed brick façades can pass the light following the idea that brickwork can be transparent. It is worth to mention the 2012 project of Brick Origami in the London School of Economics by architects O'Donnell & Tuomey (figure 1). The bricks on the external walls are used in a new way with an openwork pattern that creates dappled light inside; at night, when the lights are on inside, the building seen from the street looks like a glowing lattice lantern. Lighting and delicacy of the facade is also well visible in the 2010 project by Mestura Arquitectes (figure 2).



Figure 1. Brick Origami, London School of Economics [3]



Figure 2. Primary school, Barcelona [4]

Nevertheless, increasing slenderness of the walls combined with low bending strength of masonry requires introduction of reinforcement into the masonry wall, especially in bed joints [5, 6]. Thanks to this reinforcement, the masonry structures are resistant not only to compressive stresses but also to tensile and shear stresses [7]. Combination of brick (with high compressive strength) and reinforcement (with high tensile strength) provides wide architectural possibilities. As a result, it is possible to construct ceramic façades of extraordinary shapes [8]. This solution may be used only if the reinforced masonry façade can be taken as typical reinforced masonry structures possible to design and calculate based on the regulation given in EN 1996-1-1:2005 (Eurocode 6) [9]. Especially it is necessary to ensure the proper adhesion between masonry mortar and ceramic clinker masonry units. Sometimes, by the different reasons there is not possible to make the brick layering at the building site and the external façade layer have to be prepared as prefabricated construction. And it is precisely with this situation that the authors of this paper dealt with.

2. Description of the research problem

During the design process of the new faculty of one of the universities' in Poland the architects took the solution of the external enclosure wall as typical three-layer construction consisted of main internal load-bearing wall (concrete) with external thermal insulation covered by thin façade clinker tiles glued directly to the surface of the thermal insulation (of course with using of appropriate polyethylene plasters' net). As external finishing of the façade of the building they set up to build the independent front elevation as reinforced masonry openwork wall with fully orthogonal pattern (without bonding of masonry units in adjacent courses – see figure 3) of special clinker masonry units with shape and dimensions shown in figure 4. The designed façade was some similar to that one, presented in figure 2.

According to the basic project the external openwork masonry façade was to be built at the building site by bricklayers. The distance of this wall to the main part of the external wall was 12 cm and it was too short distance for the proper making of the brickworks, especially for correct filling all the joints by mortar. Authors suggested introducing some changes in the idea of the front layer of the façade. Instead of in building site made brickwork was proposed using of the prefabricated reinforced masonry panels, mounted to the surface of the main part of the wall by the steel structure.

The original solution of the prefabricated façade prepared by the design office, more detailed described in p.3.1, was, in authors' view, too risky by the reason of the possibility of uncontrolled

failure. Moreover, taking into consideration the high dimensional deviations of the ceramic opened masonry units (see figure 4) connected with the problems of their production by ceramic company (firing process of clay elements with open shape without internal ribs) it was extremely difficult to guarantee the appropriate dimensional deviations of the whole prefabricated panels and fixing screw positioning.

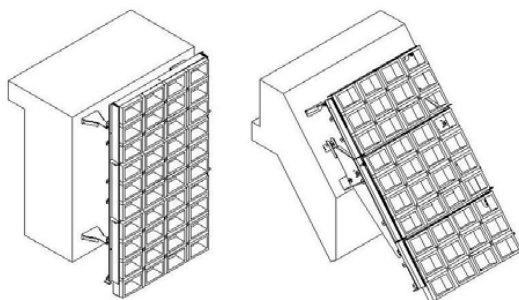


Figure 3. The idea of the solution of the façade

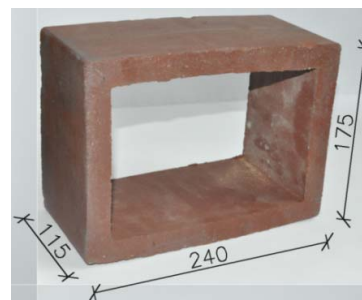


Figure 4. View of the clinker masonry unit (with main dimensions)

In such situation, authors were proposed their own design proposition of the prefabricated elevation panel eliminating most of the basic disadvantages of the first solution. Proposed, modified construction of the panels, described in p.3.2, cannot be taken as typical reinforced masonry, because the main load-bearing element was the reinforcement placed into joints in both orthogonal directions and stiffly fixed (by welding connections) to the external bordered frame. All these stainless steel elements are working together, both in tensile and compression zones. The classical using of regulations for reinforced masonry, specified in Eurocode 6 [9] was disputable. Moreover, there was difficult to find any information about tests describing the behaviour of such construction. Only one report from research of similar masonry panels, but with additional superficial strengthening by reinforced concrete medium thick layer was found [10], but these results were not possible to adapt to analysed panels. Therefore, it was decided to use the methodology “design supported by testing” according to ISO 2394:2015 [11] regulations.

The first part of the research program conducted by the authors included testing and analysis of two types of prefabricated elements: *series I* – panels according to the first solution and *series II* – panels of the authors’ modified concept. *Series I* included two elements; in *series II* three elements were tested and analyzed.

3. Characteristic of the façade panels

3.1. Panels – the first solution

Prefabricated façade panels, including in the first test series (*series I*), were made on the basis of an original elevation project. The elements were consisted of clinker masonry units with the dimension of 240×170×115 mm (see figure 4), integrated by the mortar joints of 30 mm width. The panels were reinforced by the carbon steel rebar, with diameter of 6 mm, located in every vertical and horizontal joint. The shape of the prefabricated element and the detail of reinforcement are shown in figure 5.

Series I consisted only of two elements, listed in table 1. The specimens were made with the same type of mortar (general purpose mortar with brown pigment dedicated to clinker masonry). The consistence of the mortar, during prefabrication process, was different and depended on the amount of mixing water used. The specimens were signed as: *HL-wf-rs-4×4-SPb-n.i*, where following symbols means: *HL* – mode of loading (horizontal loading), *wf* – without external frame, *rs* – ribbed reinforcement steel, *4×4* – arrangement of the ceramic masonry units filling, *SPb* – type of mortar (*b* means mortar with brown pigment), *n* – content of mixing water (in liters per one bag of dry mortar mix), *i* – following number of specimen.

During production of the clinker masonry unit some defects, in form of small cracks sometimes with higher than 0.5 mm width, in ceramic units were observed (figure 6). The area of the elements was relatively equal, what can be seen in a figure 7. There were no major irregularities and distortion.

Table 1. Designation of elements from *series I*

	Type of mortar	Mixing water content [l]
<i>HL-wf-rs-4×4-SPb-4.1</i>	General purpose mortar with	4
<i>HL-wf-rs-4×4-SPb-5.1</i>	brown pigment	5

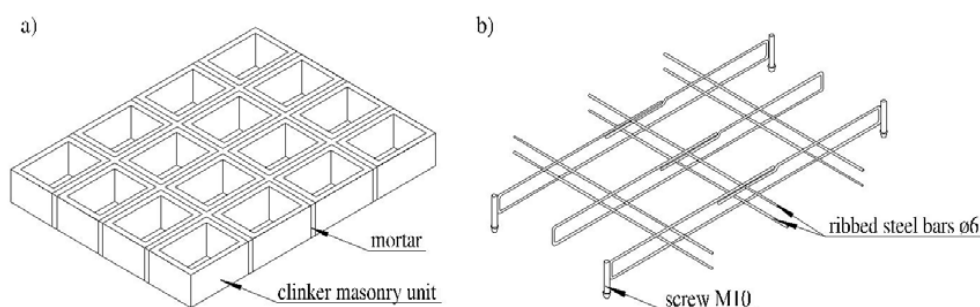


Figure 5. Element from *series I*: a) view of prefabricated element, b) detail of reinforcement



Figure 6. Cracking of clinker masonry unit as the result of the firing process during production



Figure 7. View of the element's surface

3.2. Panels of the authors' modified concept

The construction of the specimens of *series II* was modified, in relation to the elements from *series I*. The mode of reinforcing has been changed and an external boundary stainless steel frame, with the cross-section of 3×70 mm, was introduced.

Generally, the elements were consisted of the clinker masonry units – in arrangement of clinker units 4×4 – integrated by the mortar joints of 30 mm width. The new reinforcement was made of stainless steel smooth bars, with diameter of 6 mm, and was located in every vertical and horizontal joint. The bars were connected to the external boundary frame by welding. Additionally, the external clinker elements were glued to the frame by means of silicon glue to their stabilization during filling joints by the mortar. Such a solution creates a steel spatial structure, which integrate all the components of the prefabricated panel.

The shape of the prefabricated specimens of *series II* and the detail of their reinforcement connected with the frame are shown in figure 8.

Series II included three elements, identical from structural and material point of view. In all elements the same type of mortar (but with grey pigment), with the same amount of mixing water (5 l per bag of dry ready mortar mix) were used. The specimens were signed as: *HL-bf-ss-4×4-SPg-5.i*,

where *bf* means the presence of the boundary external stainless steel frame, *ss* is the reinforcing steel (stainless steel); other symbols have the same meaning as in elements from *series I* (*g* means mortar with grey pigment).

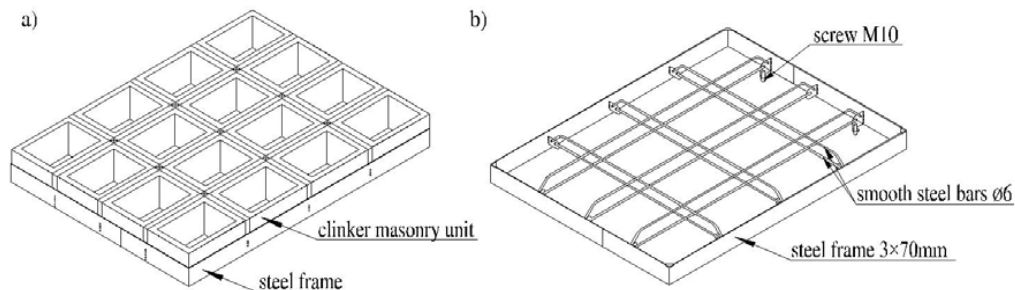


Figure 8. Element from *series II*: a) view of the prefabricated panel, b) detail of reinforcement

Before testing, all specimens were checked. A lot of different types of defects –like absence of silicone strip between the ceramic units and frame, shrinkage cracks, incorrect filling of the mortar joints (figure 9), strong surface irregularities (figure 10) – were observed.



Figure 9. Incorrect filling of the mortar joint



Figure 10. Surface irregularities

4. Test stand and test procedure

4.1. Test stand

Prefabricated panels were tested in a special designed test stand, reflected the real location of the panels on the elevation of the building. Each element was fixed to the steel transverse profiles using screws welded to the reinforcement and led out from the prefabricated panel, and steel plates mounted direct to the steel supporting profiles. In *series I* the steel plates were made of piece of steel sheet, but in *series II* a special shaped stainless steel elements were used (the cover plate is visible on figure 9). In figure 11 is shown the view of the panel from *series I* located in the test stand, just before testing. The general view of the whole test stand during the testing of *series II* is visibly in figure 12.

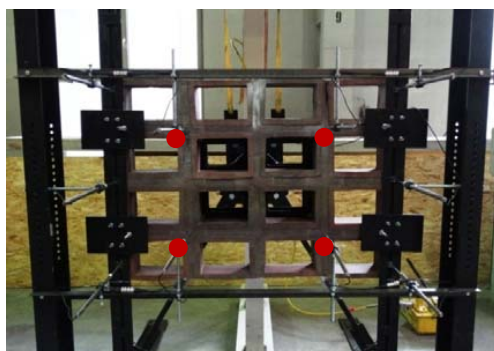


Figure 11. Specimen of *series I* prepared to test. Red dots marked points of the applied concentrate forcesFigure 12. View of the test stand with specimen of *series II* ready to test

In case of testing the elements from *series II*, the supporting profiles and the cover plates were made of stainless steel – this same as will be use in real construction of the elevation.

4.2. Loading process and measurement equipment

Both types of panels were loaded in direction perpendicular to their surface using steel traverse and hydraulic jack. The load as set of 4 concentrated forces was located on the crossing of joints, what is marked by the red spots in figure 11. The elements of *series I* were loaded in one cycle, up to the failure. However, in *series II* cyclic load was applied. The specific of loading the specimens of *series II* is listed in table 2. Two first tested panels were loaded in the same way – five cycles with the range of load amounted from 5 kN to 20 kN – but in last cycles (the sixth) the maximum load was higher; the specimen was loaded up to the 25 kN. In last tested specimen (*HL-bf-ss-4×4-SPg-5.3*) only the last cycles was different; the element was loaded up to the failure – what means up to the deformation of the vertical supported steel profile. The element was not failed completely.

During the tests, applied force and displacement perpendicular to the specimen surface were recorded. Both those values were measured with the LVDT gauges and frequency of 2Hz. The displacement was determined in eight points. Four points was on the vertical steel supporting profiles (two from every side) and four was located in the same place, where the load was applied (on the crossing of the joints – see figure 11).

Table 2. Values of loads in elements from *series II*

Specimens	Maximum load in first cycle	Cyclic loading range (five cycles)	Maximum load in last cycles
<i>HL-bf-ss-4×4-SPg-5.1</i>	20 kN	5 kN - 20 kN	25 kN
<i>HL-bf-ss-4×4-SPg-5.2</i>	20 kN	5 kN - 20 kN	25 kN
<i>HL-bf-ss-4×4-SPg-5.3</i>	20 kN	5 kN - 20 kN	34 kN

5. Test results

5.1. Cracking stage

During the testing of all specimens the cracking load (F_{cr}) with corresponding bending moment (M_{cr}) were recorded and specified. Additionally, the deflection of whole prefabricated element – based on the measurement of displacement – was calculated. The deflection was defined as the difference in the direct displacement of the prefabricated element (mean value at four points, where the load was applied) and the deflection, measured on the steel supports (mean value at four points on the steel profiles).

The most important data recorded for all tested specimens (cracking load, cracking bending moment and corresponded to them deflection) are listed in Table 3.

Table 3. Values of load, bending moment and deflection noticed at cracking

Specimens	Cracking load [kN]	Cracking bending moment [kNm]	Deflection at cracking [mm]
<i>Series I</i>			
<i>HL-wf-rs-4×4-SPb-4.1</i>	9.72	1.24	5.54
<i>HL-wf-rs-4×4-SPb-5.1</i>	8.51	1.09	5.48
<i>Series II</i>			
<i>HL-bf-ss-4×4-SPg-5.3</i>	4.03	0.52	0.73
<i>HL-bf-ss-4×4-SPg-5.4</i>	3.93	0.51	0.98
<i>HL-bf-ss-4×4-SPg-5.6</i>	4.60	0.59	0.83

It can be visible, that the elements of *series I* have cracked some later, at the bending moment more than two times higher, than the elements tested in *series II*. The deflection of those elements was also higher, but its' increasing has been not proportional (adequate) to the increasing of the cracking load. The deformation of elements of *series I* was more than six times higher, than recorded in testing of specimens of *series II*. Such behaviour can be explained by changing which occurred into their structure. Usage of ribbed bars, in panels of *series I*, ensured better bonding between reinforcement and the mortar, than in case of smooth bars used in elements of *series II*. This resulted in later appearance of cracking, observed at higher loading level. The using of external boundary frame with simultaneous fixing of the reinforcement to them had the significant influence on stiffness of tested elements. The elements with steel frame deformed much less than elements without external frame.

5.2. Failure modes of tested panels

Both elements of *series I* have behaved and failed in the same way. They were completely, physically destroyed. At relatively small loads some cracks, between the clinker unit and the mortar had appeared. With increasing of the load, the width of cracks was also growing up and, as a result, the external units were separated from the whole panel. In next steps of loading the reinforcement began to deform and destroy the mortar structure. Finally, the element was divided by cracks into the almost independent pieces. There was no bonding between the clinker elements and the mortar (the components could be separated without any efforts); also the reinforcement was completely separated. Some of the ceramic clinker units (located in external parts of panel – mostly at corners) were destroyed. Such a type of failure could be taken as an unsafe and danger from the safety point of view.

In figure 13 one of the specimens from tests of *series I*, just before reaching the state of failure, is shown. Figure 14 presents the view of destroyed structure of mortar and clinker masonry unit.



Figure 13. Failure of element of *series I*



Figure 14. Detail of the destroyed panel

The maximal value of the bending moment, calculated on the basis of the maximum load, reached during the whole loading of the specimen, was taken as a load-bearing capacity of elements. The values are listed in table 4.

Table 4. Maximum load and bending moment for elements from *series I*

	Maximum load [kN]	Maximum bending moment [kNm]
<i>HL-wf-rs-4×4-SPb-4.1</i>	18.26	2.33
<i>HL-wf-rs-4×4-SPb-5.1</i>	15.30	1.95

All panels of *series II* behaved practically in the same way, during all cycles of loading. As in case of panels of *series I*, but at even lower loading levels, some cracks between the ceramic filling and the mortar have appeared. With increasing of the loading, the width of cracks was growing up, but no excessive deformation of breaking was observed. The usage of smooth bars allows the slip between the reinforcement and mortar and therefore the structure of mortar was not destroyed. After realization

of a few loading cycles, followed with the relatively higher loads, the panels kept their pervious geometry. Besides many small cracks, indicated separation of ceramic units from the mortar, no components attends to falling out and even visibly getting out from the panel. Such effect was reached by using the external frame, connected with reinforcement, and glued to the panel. No real failure of panels was achieved – even in the third tested elements, where the maximum load, applied in last cycle, was the highest – because of the excessive buckling of steel supporting profiles; threaten sudden and very danger damage. In figure 15 the pattern of cracks at reaching the maximum load is shown.



Figure 15. Pattern of cracks at maximum load

5.3. Load – deflection relationship

On the basis of the values of applied load and measured deformation of panel the load-deflection relationships were determined. In figure 16 the diagrams for both elements of *series I* are presented.

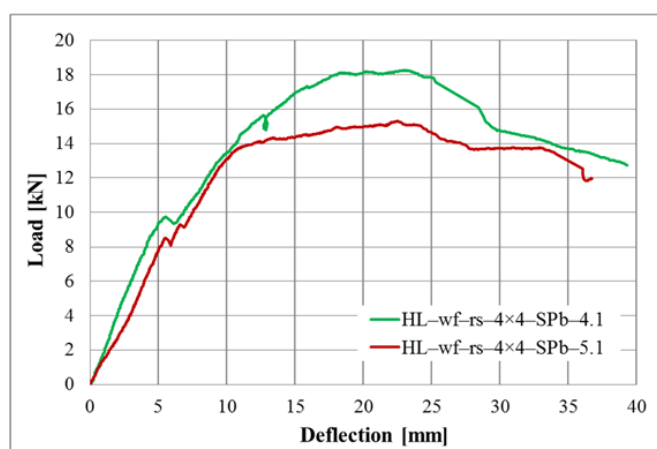


Figure 16. Load-deflection relationship in panels from the *series I*

As it is visibly on the graphs, the amount of the mixing water used in the mortar preparation (during prefabrication process of the panel) has insignificant influence on the stiffness of the panel, but resulted in changing of their load-bearing capacity. Less amount of water – 4 liters, what ranged in the values given in technical recommendation of the mortar producer – resulted in relative small increase in the capacity of the element (ca. 20%), but significantly made prefabrication difficulty and proper making of the panels, in comparison with the element, where 5 l water was used. Thus, the application of higher amount of water was reasoned from the technological point of view, even at the cost of reducing the load-bearing capacity of the element.

The deformation of the elements, just before failure, was significant. The deflection of elements at reaching the maximum load was two times smaller, than at failure (the values amounted to 40 mm).

Behaviour of the specimens tested in *series II* is visible in blue graphs, presented in figure 17. Additionally is shown, in comparison, with red colour the load-deflection relationship of panel from *series I*, featuring the same amount of mixing water (5 l), that panel from *series II*.

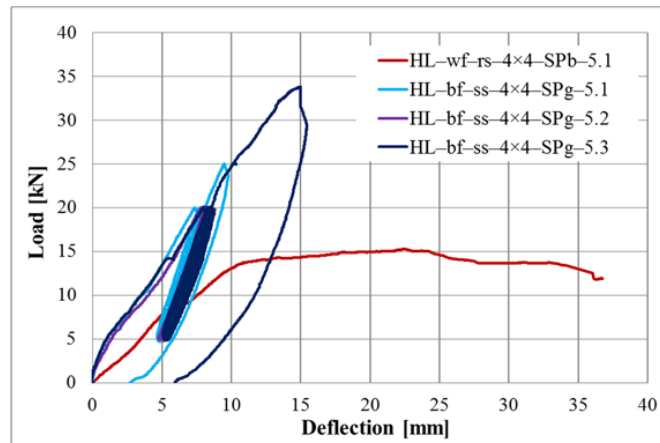


Figure 17. Load-deflection relationship of panels from the *series II* (blue) and one element from *series I* (red)

It can be easily visible, that all elements of *series II* behave almost identically. After first cycle some plastic deformations were recorded. During cyclic loading almost elastic behaviour of panels was observed, where the unloaded element shows only small increasing in deflection.

5.4. Comparison of the results for both series of tested panels

Prefabricated façade panels made according to the first solution demonstrated relatively high crack resistance but corresponding with rather low load-bearing capacity determined for the level of ultimate loads. At the loading levels close to the reaching the state of failure large cracking and local damages of masonry mortar between adjacent clinker units was observed. As the result of this situation, the falling out of the clinker units from the corners of the panel was observed. At the state of failure tested specimens was practically disintegrated. Moreover, panels of this type were characterised by not acceptable dimensional tolerances, especially in case of localisation of the steel screws for mounting of the panel to the steel supporting frame. In construction of the real front elevation of the building such situation, from the executive point of view, is not acceptable.

Prefabricated panels with the modified construction, tested in *series II*, were characterised by lower crack resistance in relation to specimens of *series I* because of using stainless steel smooth bars instead of ribbed steel bars. At the same time, they showed very high load-bearing capacity and stiffness, despite of cyclical loading. In subsequent cycles, the panels behaved almost elastic, returning after unloading to the original geometry. Beyond the cracks, appear at the contact zone between mortar and the surface of the clinker filling units, no other damages have been noted. It was also not possible to reach the state of the failure of these tested panels by the reason of the significant deformation of the supporting steel elements. Using of the steel bordering frame has guaranteed keeping the appropriate dimensional tolerance. The proposed modified construction of the panels is the appropriate solution and guarantees the high level of safety and durability of the front elevation of the building.

6. Summary and conclusions

The case of the interesting from the architectural point of view but untypically and difficult to build, masonry openwork façade is presented in this paper. The original architectural conception of the external layer of the elevation as typically masonry wall with orthogonal arrangement of the ceramic clinker masonry units (without bonding) was not realistic due to workmanship difficulties. Therefore was necessary to search of the other solution of the façade preparing, guaranteed the adequate safety level, durability and which will be also easy to construct at the building site.

Authors were recommended the construction of the elevation based on the prefabricated panels with limited overall dimensions (reduction of the self weight of the panels) and with easy fixing them to the basic (load-bearing part) of the external enclosure wall, which was fully accepted by architects. Based on this data and suggestions in design office project of the prefabricated panel was elaborated. This first solution was not so correct, characterised by low durability and unsafe. Under higher level of loading these panels (first solution) were threatened with uncontrolled disintegration, what completely is not acceptable. Carried out tests of two panels of *series I* have showed the weakness of this solution.

Therefore, authors were proposed their own solution of significantly modified prefabricated panels. Tests of these modified elements (tested specimens of *series II*) fully confirmed the correctness of the proposed solution. Modified construction of the prefabricates is safe and characterized by high durability and easy montage.

Summarising all information, aspects of analysed structural and practical problem the conclusions, listed below, and may be formulate.

- Modern masonry buildings in relation to the material and structural as well as architectonical solutions sometimes require an individual and unconventional approach, often exceeding the typical standards' cases.
- Each structural and material solution should guarantee the appropriate level of safety and durability for the long period, therefore sometimes is necessary to introduce the significant changes and modification as in presented case of external part of the elevation of the university building. The original masonry façade made as brickwork type may be replaced by the prefabricated construction guaranteed this same architectural and visible effect but more safe and durable.
- In case of lack of the standard regulations or when basis on the structural standards is impossible or there are reservations about correctness of their using it is possible to support the design process by making tests. This method is accepted and recommended in all such cases and is very useful and effective. Unfortunately it has one main vice: it is rather expensive.

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