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Masonry Construction Remedial Measures in Case of a Multi-Story Housing Facility Caused by Floor Extension Process

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Abstract

This paper presents remedial measures of a multi story housing facility located in Novi Sad, Serbia. The analyzed building was built in traditional masonry construction with precast rib ceiling including unreinforced slab. The damage was caused after the floor extension (overbuilding) process of the existing facility, expressed through cracks on constructive walls. The cause of such damage is a non-analytical approach during upgrade. The work presents a case study of the applied measures at conditions when the material incorporated in the existing part of the building is not of adequate quality. The specificity of this recovery was that during the execution the flats' users remained in their apartments.

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Keywords: masonry construction, construction failure, remedial measures Introduction

1. Introduction

Collective residential buildings with roof terraces have proved to be an important building resource which allows the construction of cheaper housing. The study shows that the building belongs to a category with a long amortization period (approximately 60 years). This categorization includes a special treatment in the upgrade, due to the amortization of all building elements (structure of the coating), which is directly caused by the ageing and its

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construction quality. The observed changes, applying to most stationary loads, become exaggerated and dangerous to the safety of users, as well as the survival of the facility.

The cause of the damage is imprecise process of rehabilitation and upgrading. It should be noted that the designers anticipate and implement some of the security measures before beginning the upgrade. This paper presents a standard mitigation measures that can be applied in cases of damage to masonry structures. In the newly created state of basic construction requirements for the room and the user's life were unsustainable and demanded a fast, reliable and safe intervention.

The paper presents improved measures for the rehabilitation of masonry in terms of simultaneous use of facilities by tenants. The main goal of the proposal is to raise the functional level of the building in which raised the requirements of thermal stability of the building, due to the time when the subject facility was built, it did not take into account the way it is today [1-5].

2. "Overbuild" as a form of rational construction

States with undeveloped economies often do not have an organized form of investments and maintenance of residential buildings with deck roof. Therefore, the end result is usually unsatisfactory with high production costs. Tenants can not finance the cost of rehabilitation of the deck roof from private sources. Upgrading of facilities is the optimal form of techno-economic intervention that achieves a better protection of buildings from weather and physical impact. This led the legislator to promote this process inside the plan of detailed regulation. For the legal validity of an upgrade it is required to provide the consent of tenants in the building and the consent of the designer of the existing building or guild associations. With the acquisition of legal validity is necessary to make the project technical documentation to meet the upgrading of technical-technological sense, which includes all elements of the upgrade:

- calculations which determine the load of the upgraded parts
- calculations which determine the capacity of the existing building structure for new loads
- measures which include preparation in order to provide security from the effects of new load

3. Masonry

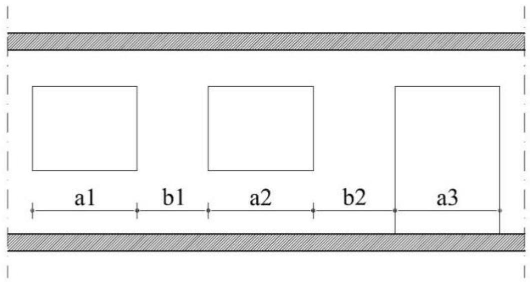
3.1. Physical and mechanical properties of the unreinforced walls

Masonry walls and columns, as supporting elements, other than its own weight, in general they may be also exposed by other loads: vertical - the stories and roof, and horizontal - the effect of wind, seismic forces and the pressure of the ground (basement walls). Masonry structural elements are usually adopted with great caution, so the reliability of masonry structures in relation to the vertical load is, always higher than in other structures. Tab. 1 shows the limits prescribed by "Regulation on technical standards for masonry walls ".

Table 1. Recommended allowable number of storeys above ground according to Serbian standard

Type of construction	Acceleration at site		
	VII	VIII	IX
Unreinforced masonry	3	2	1
Masonry masonry with vertical and horizontal ring beam	5	4	3
Reinforced	8	8	8

Masonry should be designed with simple geometries and proper form of the basis. Depending on the object function, it is a great presence of the openings of the elements of windows and doors. Given that the impact of these openings on the stress-strain state of the walls are difficult to cover by calculation (which is particularly true in the case where the walls are exposed to horizontal loads in their planes), technical regulations limit the size of the openings (Fig. 1.).



Zone VII .... aj ≤ 3,5 m, bj ≤ 1/3 x aj

Zone VIII .... aj ≤ 2,5 m, bj ≤ 2/3 x aj

Zone IX .... Same as zone VIII

Fig. 1. Requirements of opening sizes in the walls according to technical regulations

The main physical-mechanical characteristic of walls is compressive strength which depends on both; characteristics of building elements and material properties. The process of examining the walls to pressure of fracture can be defined by four characteristic phases. The first phase (2a.) suits to the normal exploitation of the wall, when the stress does not produce any damage. Transition wall to the second phase (2b.) is characterized by the appearance of small cracks in some small number of masonry, which is 60-80% of ultimate load, and if at this level stops increasing load, there will be no further development of cracks. However, if the load increases over the specified level, the wall will show vertical cracks and these cracks, linking to each other, in the third phase of the wall (2c.), will have the effect of its division into several, more or less independent parts. During the longer activity of the load suiting to this stage (even without its increase), there will be a further development of cracks, so that the wall will, actually, be separated into thin slender pillars, when losing a stability means the fourth - the last phase of the wall activity, i.e. his fracture (2d.).

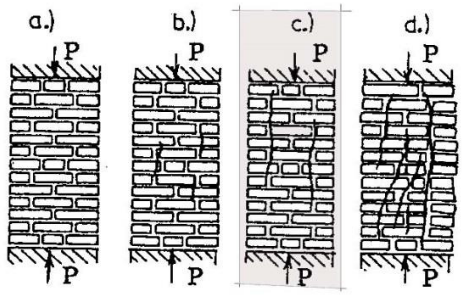


Fig. 2. Phases of activity in the wall loaded by pressure

4. Project documentation for upgrade

4.1. Geotechnical conditions

Foundation depth and dimensions, and the calculation of soil bearing capacity resulted in 191-196 kN/m2. Subsidence of the upgraded facility, for one floor and the attic, would be in the range of 2 to 5 mm, and depending on the actual size of the additional load foundations. Control of the actual subsidence was carried out by observing rappers during the upgrade. Throughout upgrading it is necessary to use lighter materials in order to less disrupt the existing stress state in the structure and soil. The load should be applied equally on existing building. It is necessary to record in detail the current state of object in build both the geometry and quality of the installed material. With these parameters perform a detailed analysis of strength and stability of structural elements of buildings.

4.2. Preliminary Upgrade Design

Upgrading predicts the construction of one storey and attic. On the part above the attic as well as the floor is foreseen half-precast ceiling. The ″Analysis of additional loads of housing″ was given by preliminary project. Within this analysis were obtained by stresses in the soil under the certain foundations of object given in Tab. 2. Schedule of floor and ceiling suspension is given in the preliminary and final design both, of the existing object (Fig. 3, a)) and the upgrading (Fi. 3, b)).

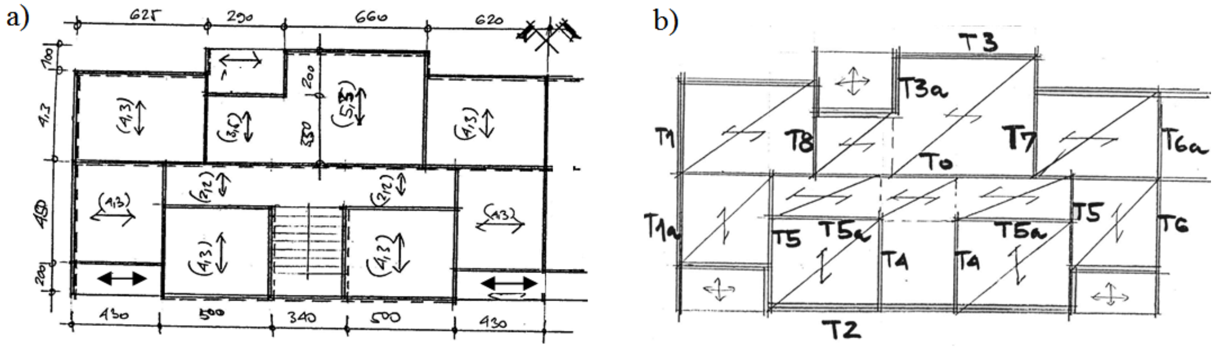


Fig. 3. a) Plan of existing positions, b) Plan the positions of the new designed part reflected on the foundations

Table 2. Stresses in the soil beneath of some foundations of the intended upgrade

Foundations	Stresses in soil, σt [kN/m2]
To	205.00
T1a i T2	191.20
T3	203.00
T4	182.00
T5a	206.00
T5	205.00
T6	205.20
T7	211.00
T8	200.90

The project provided that some foundations exceeded the allowed stress of 10%.

4.3. The main upgrade project

The main project includes the following:

- the calculation of impact and sizing of elements related to the upgrade,
- the analysis of seismic impacts which were obtained by the calculation that the stresses in the walls, due to action of seismic forces is less than the allowed
- σ_n = 6,28 N/cm^2 < σ_{ndop} = 9,00 N/cm^2
- the analysis of additional loads - taken from the Preliminary Design.

4.4. The main project of ensuring structure

The properties of the walls are considered to be of lower quality material (less baked bricks, old format), which also represents the Main Structure Project of existing facility upgrade. This description is given as an annex to the calculation of structure as evidence that, with recovery, the facility can stand the new workload. Newly designed state provides the repair of groundfloor walls by inserting pillars which would take over most of the vertical loads.

4.5. Subsidence control of the housing facility

Subsidence control of the residential building was carried out by two companies, with reports:

- rappers are defined and set on the building and made a record of absolute elevation rappers (″0″ 29.07.2002.g + 4 series) from the first company and obtained subsidence of building and it is given in Tab. 3.

Table 2. Rappers subsidence after four measurements

	1. series 04.09.2002	2. series 07.10.2002	3. series 05.11.2002	4. series 04.12.2002
R-1	0	-1	-2	-2
R-2	-1	-1	-2	-2
R-3	-2	-2	-3	-3
R-4	-2	-3	-3	-3
R-5	-2	-3	-4	-4
R-6	-2	-2	-2	-3

- the control and measurement of 5-series 21.05.2003.g. made by second company and the following results are given in Tab. 3.

Table 3. Rappers subsidence after the 5th measurement

	5. series, 21.05.2003
R-1	-2
R-2	-2
R-3	-4
R-4	-4
R-5	-3
R-6	-2

All the measured subsidence of the building (6 measuring points - rappers) are within the limits assumed in the framework set out in the Report of the geomechanical conditions, which assumes the subsidence of the building due to the upgrade from 2 to 5mm.

5. Condition of the structure after upgrade.

5.1. The analysis of generated situation

The main characteristic of the existing structure is masonry construction with longitudinal and transverse bearing walls, facade walls d = 38cm and bearing inner walls d = 25cm, mezzanine ceiling is ribbed half-assembly, type ″ Herbst ″, basement bearing walls are d = 51cm. The existing part of the building has no vertical or horizontal AB ring girder. Above the openings there are AB beams, while in the basement some arches are built up of masonry. The staircase is double (AB slab), with landing beam. Ground beams are of foundation concrete.

The upgrade built with vertical and horizontal AB ring beams, floor structures of type ″Fert ″, and wooden roof structure. The roof covering is tegola. The quality of materials of the supporting masonry walls was not experimentally determined before the design and execution of works to upgrade. It was not analyzed, as part of project documentation relating to the upgrade, the capacity of walls of the existing building for existing and additional vertical load. According to the Preliminary and Final design, the upgraded structures (floor ceiling) is unevenly redistributed to the two longitudinal and a transverse walls of the facade that is conected with them (southeast facade receives much of the load). Total increase stress in wall due to this upgrade is about 25-30%. The level of subsidence of the building according to the affordable measurements is with in the anticipated (Tab. 2 and 3), which could contribute to the emergence of additional strain in the building. However, if shows the symmetrical subsidence of both façade, and the difference in the subsidence of the ends and in the middle of the building is 2 mm, it results that the deformations of building caused due to subsidence is less unlikely. On the walls of the basement, the thickness of 51cm, no significant damage was observed. Cracks and crevices on plinths of south-east (entrance) façade are more prominent than the other facades. On the facade walls I and II floor cracks decreases with increasing levels.

5.2. Description of the structures

Damages was observed about 3 months after moving in the upgraded parts, inside and outside of the facade wall (south-east - entrance wall), and at the inner walls closer to facade wall, picture xxx. Repair of walls ground floor (the outside) according to ″design for the reparation″ was performed five months after moving in. The damage to apartments on I and II floors are relatively small in view of the small cracks and fissures in the walls, while the damages to supporting walls on ground floor are extremely large. The layouts of dameges supporting walls of ground floor are unequal. Damages were classified in: horizontal, vertical and diagonal cracks, as well as parts with extremely high damages where there were the fractures of supporting walls. it was observed that the most obvious damages to the supporting walls of the southeast facade, especialy in part of masonry ″columns ″ placed among the windows in the lower level of window edge and slightly above the breast. The damages are manifested in the form of separating and falling plaster in the form of horizontal and vertical cracks (Fig. 4). Also, extreme damages occur in the transverse walls in immediate vicinity of the longitudinal facade walls (entrance). The intensity of damages to the supporting cross walls (expressed in the form of horizontal cracks) proportionaly decline with distance from the entrance facade.

Fig. 4. a) Damages of basic material, b) Unequal thickness of the vertical and horizontal joints in wall

In areas of extreme damage and fallen plaster it was observed the following: the walls are plastered with lime mortar, the bricks which used for making the bearing wall are out of expected quality (well roasted red, yellow slightly roasted), mortar for brick cement is a flexible mortar, which is, based on visual views, can be said that good quality, uneven thickness of the vertical and horizontal joints in walls. It was performed the design and reconstruction of the walls of the entrance facade. Restoration was carried out making ten reinforced concrete, 25/25cm pillars, which are placed between the ceiling above the ground (below ″unknown″ a horizontal concrete element in the ceiling above the ground level) and the wall plinth (basement d = 51cm), with that a pillar in the place reliance the basement wall with trapezoidal enlargement (Fig. 5).

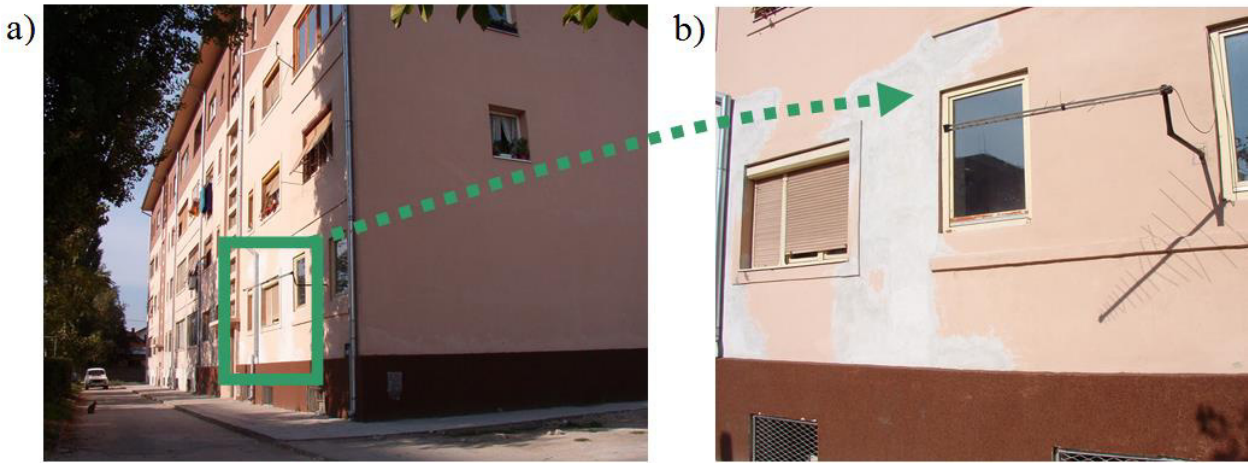


Fig. 5. a) South-east (entrance) facade, b) Partial recovery that prevented demolition of the ground floor walls

The greatest damage in the apartments are on the walls (columns among windows) south-east (entrance) facade, as well as close to them cross-walls (supporting). The level of damage to the lateral walls and the level of damage to apartments in general decreases with removal from the southeast (entrance) facade. A loggia on the northwest side of the building have a hole the size of 259cm and less than those on the south facade and on the walls of this orientation have not observed significant damage.

Southeast facade has suffered such damage because the structure of the wall: large windows (144cm) at close distance (112cm) the loggia, which opening amounts 360cm, a portion of the wall which divides the loggias is 65cm, 25cm cross walls are weakened by 65cm wide opening for the door at a 50 cm distance from the connecting with the southeast wall facade. Walls with small openings (or without them), with the same disposition as previously mentioned above, have significantly less damage to itself. Damage in the form of cracks and falling plaster (ground floor) is also occurring on the walls of which are bearing doors and windows lintel as well as the walls which connects with the parapet to a height of 90cm (from floor). At places the greatest damages to the walls where the plaster has fallen away it can be said that the walls entered the stage c.) (Fig. 2, c)) - the division of the wall vertical cracks on several more or less independent parts.

For the state of the walls where the border wall bearing capacity is exceeded, the experimental determination of mechanical properties (sampling) is recommended (in addition to their reduced capacity). Determining the quality of the wall was made to calculation model based on the load data (taken from the main project), the characteristics of the wall at floor level and the fact that due to this load has been exceeded limit state. For a typical pillar that placed among the windows (given in Fig. 6) the calculation of load capacity has been done.

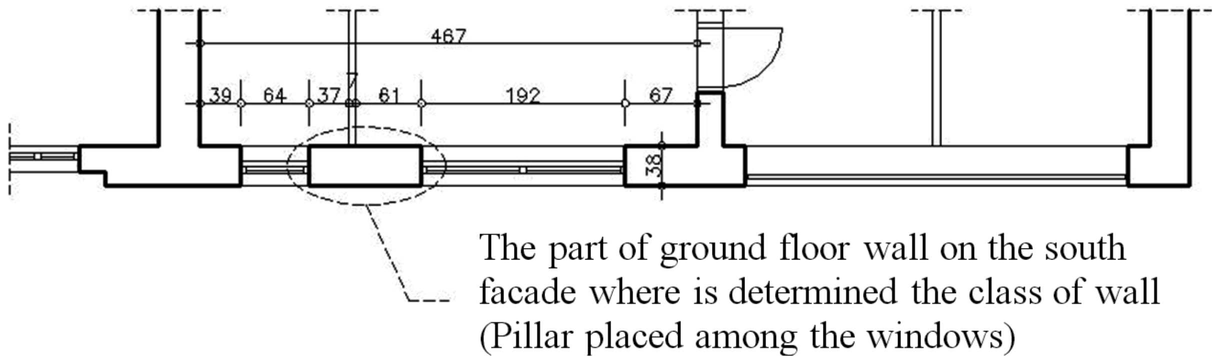


Fig. 6. Typical pillar that placed among the windows.

1. Determining the load on the pylon which placed up among the windows on the south facade, due to which there has been significant damage:

The load of the wall without openings:110.52 kN/m2, σz=290.84kN/m2

The load of the wall with openings: 89.40 kN/m2, σz=235.26kN/m2

The load on the pillar: maxQ=395.34kN

Distributed force on the observed part of the wall - distributed force at fracture in the wall on the south-east facade: 233.69 kN/m'

2. Determining the critical load of section and distributed force at fracture in the middle of the height of the wall on the south-east facade of the vertical effects the different class of the wall:

$$N_{uv} = \frac{\omega \cdot d \cdot f_k}{\gamma_m} \tag{1}$$

For the baked clay final coefficient of flow is: $\phi_\infty = 0.70$

For the observed wall buckling coefficient is: $\frac{h_{ef}}{d} = \frac{280.00}{38.00} = 7.37$

Eccentricity due to material flow is:

$$e_\infty = e_0 + \Delta e_\infty = 20.00 + 2.84 = 22.84cm \tag{2}$$

Initial eccentricity of the load force in the middle of the height of the wall: $e_0 = 0.05 \cdot 400.0 = 20.00cm$

Additional eccentricity of the load force in the middle of the height of the wall:

$$\Delta e_\infty = 0.02 \cdot \phi_\infty \cdot \frac{h_{ef}}{t} \cdot \sqrt{t \cdot e_0} = 0.02 \cdot 0.70 \cdot \frac{280.00}{38.00} \cdot \sqrt{38.00 \cdot 20.00} = 2.84cm\ , \tag{3}$$

$$\frac{e}{t} = \frac{22.84}{38.00} = 0.60 \text{ adopts the } \frac{e}{t} = 0.40$$

From the Tab. 12 ˝ Rulebook on technical standards for masonry walls ˝:

$$\omega = 0.31$$

For the lowest class wall being examined by Rulebook KW 1.5 (fk=1.2 kN/m2):

$$N_{uv(1.5)} = \frac{0.31 \cdot 0.38 \cdot 1.2}{0.85 \cdot 3} \cdot 1000 = 55.44 \frac{kN}{m} \tag{4}$$

For the highest class wall being examined by Rulebook KW 20.0 (fk=16.0 kN/m2):

$$N_{uv(20)} = \frac{0.31 \cdot 0.38 \cdot 16.0}{0.85 \cdot 3} \cdot 1000 = 739.14 \frac{kN}{m} \tag{5}$$

Dependence of critical load wall section d = 380 mm of the vertical loads on the south-east facade, as well as a dependence distributed power during the fracture of wall d = 380 mm of the class wall on the south-east facade is shown in Tab. 4. If from the equation of critical load of the wall section omitted the partial factor for materials, during the fracture of observed wall the values of distributed force were obtained [kN/m']

Table 4. Dependence of the vertical loads and distributed power during the fracture of wall.

Class wall KW	1.50	2.00	2.50	3.50	5.00	6.00
Typical compressive strength [N/mm2]	1.20	1.60	2.00	2.80	4.00	4.80
Ultimate bearing capacity of the wall [kN/m']	55.44	73.91	92.39	129.35	184.78	221.74
Distributed force at fracture of materials [kN/m']	141.36	188.48	235.60	329.84	471.20	565.44
Class wall KW	7.00	9.00	11.00	13.00	16.00	20.00
Typical compressive strength [N/mm2]	5.60	7.20	8.80	10.40	12.80	16.00
Ultimate bearing capacity of the wall [kN/m']	258.70	332.61	406.53	480.44	591.31	739.14
Distributed force at fracture of materials [kN/m']	659.68	848.16	1036.64	1225.12	1507.84	1884.80

Dependence distributed force at wall failure d = 38cm of the class wall on the south-east facade is shown in Fig. 7.

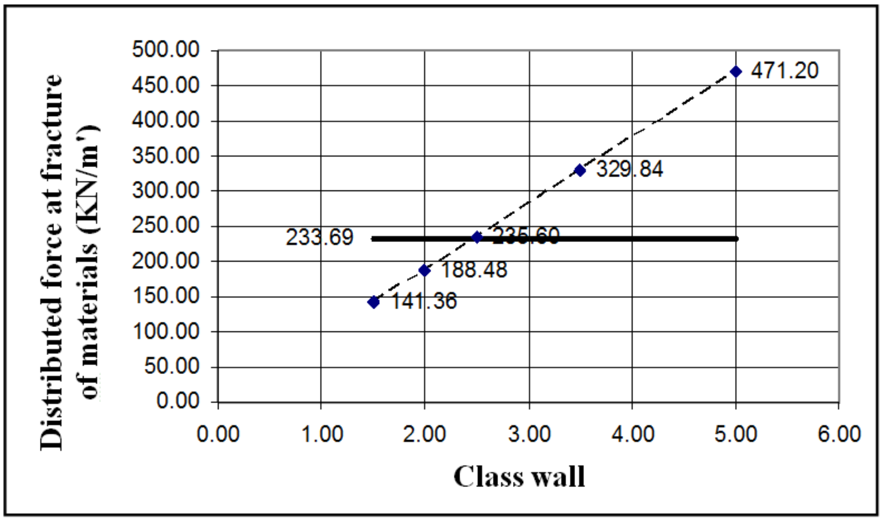


Fig. 7. Dependence of distributed force to the class wall

Based on the presented analysis can be concluded that the class of the wall on the south facade is very close to CW 2.5.

6. Repair proposal

When performing the repair facility is necessary to carry out continuous monitoring of its behavior. Rehabilitation works to be carried out (to predict rehabilitation project) at the facility:

- It is necessary to repair masonry arches in the basement of the addition of AB or steel elements (beams).
- It is necessary to repair the support parts of stairs.
- Remediate damaged ribs ″ Herbst ″ ceiling in the basement, cleaning corroded reinforcement and add the missing part of the concrete.
- It is necessary to repair the damaged walls of the ground foor. Condition of structural walls south-east facade is such that the repair should avoid using destructive methods that would further reduced bearing capacity and jeopardize good stability. In all measures of rehabilitation is necessary to monitor the behavior of the walls in the rehabilitation and repair itself in a way to predict the position of necessary supporting.

7. Repair of ground floor walls

7.1. The most frequently used measures of rehabilitation of damaged structures in practice

In practice are using already established methods for the recovery of damages on buildings (Fig. 8, 9). Such measures require a well-prepared work environment. However, the nature of the intervention on the subject property is such that it can not disturb the work and life processes of their users, so that none of these methods can be fully applied in an appropriate manner. Also, the nature of the damage is no such to a request for implementation a systematic recovery (Fig. 8), which would further increases costs.

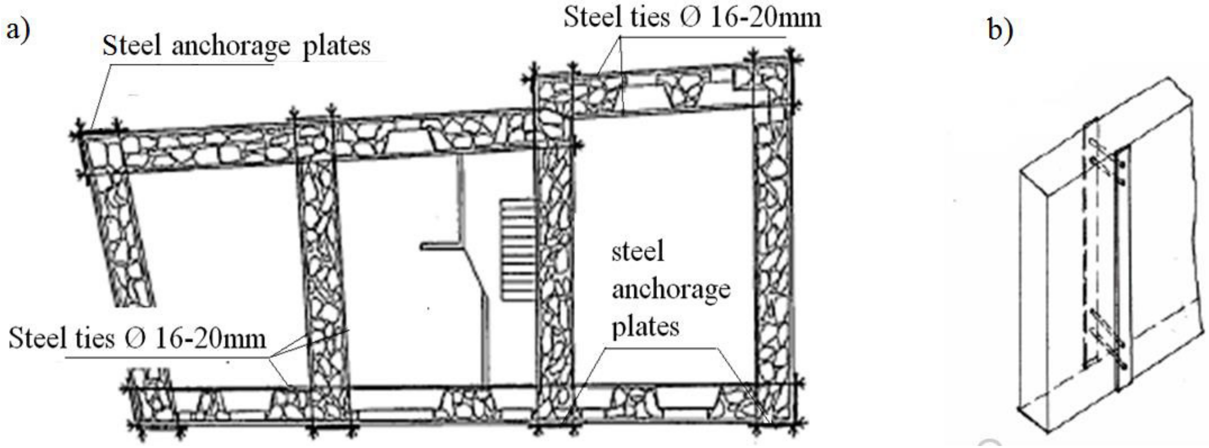


Fig. 8. a) Repair method of walls compound with steel ties, b) The method where the repair of walls is performed with a flat anchored in a wall (rarely in use).

The method where the repair of walls is performed with a reinforced cement coating Fig. 9.

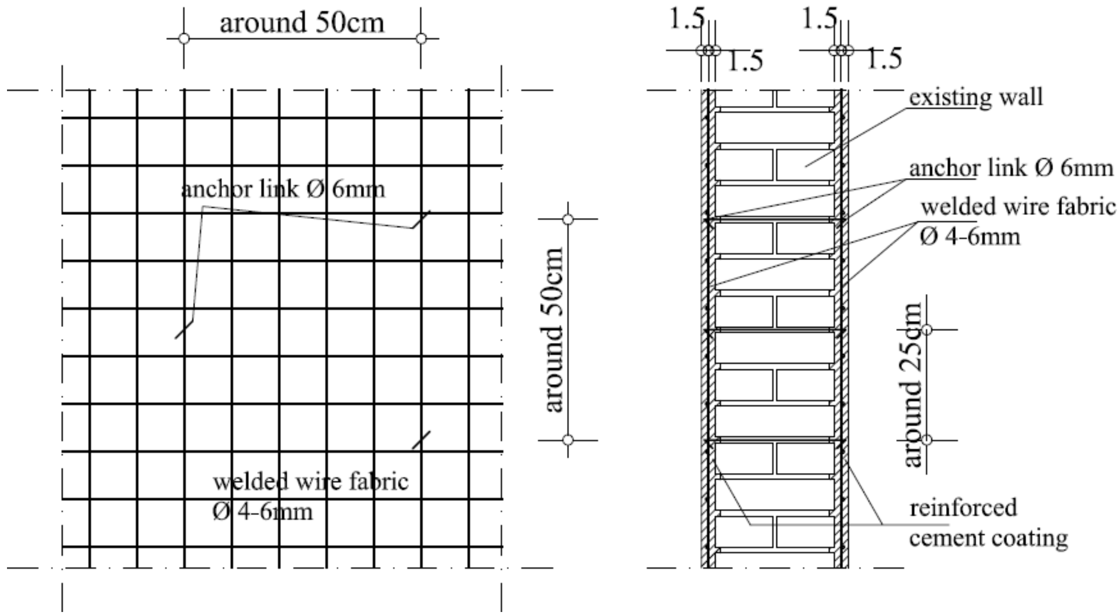


Fig. 9. The method where the repair of walls is performed with a reinforced cement coating.

7.2. Proposed methods for repairing damage

Rehabilitation predicts new reinforced concrete pylons (25x25cm), by cutting into the facade wall, at a level of ground, between the slabs of ground-floor and floor. Eleven pylons will be put on the south-east facade (Fig. 10). At the first floor level are cutted horizontal ring beams that are anchored into the new pylons and existing slab. Along the southeast facade, at the parapet wall level (about 90cm) AB canvas were derived, a thickness of 10 cm, with only 5 cm into the wall mass. Most of the work in outer space tried to make it at this way. Given the projected pylons in the walls (dwall = 38cm), which weakens the intersection of cracked walls, while not being tied concrete needed to predict scaffold that will downloaded the load. Scaffolding practically strutting two plates between which are inserted new RC pylons, in duration of seven days as it is considered necessary for the concrete setting.

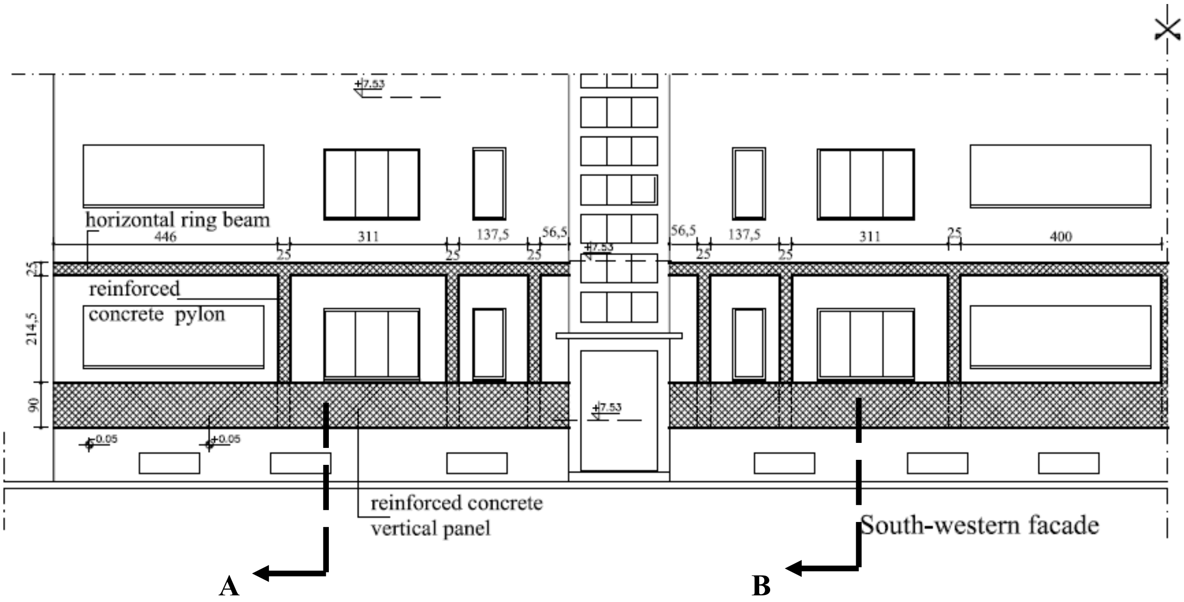


Fig. 10. Elements of reinforcement on the southeast facade – elevation.

Places damage to walls and the position of pillars to strengthen the structure shown in Fig. 11.

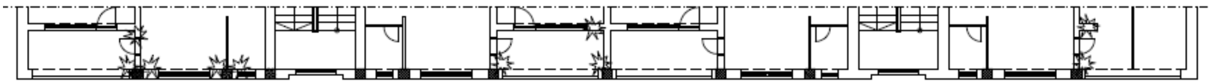


Fig. 11. Places damage to walls and the position of pillars to strengthen the structure

Final treatment of the façade wall is made of polystyrene plates that allowing a double benefit:

- facade coating is applied to a stable surface which increases its durability (Fig. 12),
- increases the energy efficiency of buildings (Fig. 13)

a) 

b) 

Fig. 12. Elements of reinforcement on the southeast facade – elevation

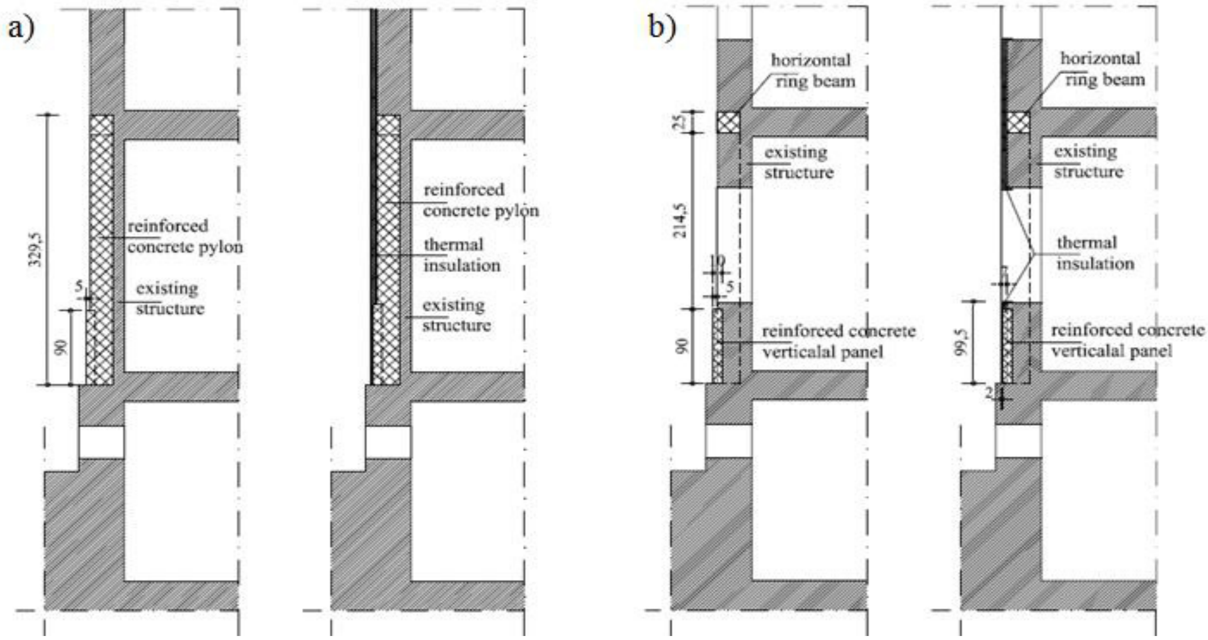


Fig. 13. a) Section A, b) Section B

Republic of Serbia has not yet adopted a regulation on energy efficiency in buildings, but it's just a matter of time. Under the Regulations all buildings will be energy efficient, with energy passports. A benefit from the savings of energy consumption can be seen from the southeast wall calculation, for cases with and without insulation. Benefit is the greater because the isolation procedure is not too expensive. On the basis of the calculation the values were determined for the overall coefficient of heat transmission construction parts of south-east facade. For section A, without isolation Uc = 2.322 W/m2K, while the addition of polystyrene layer thickness d = 9cm Uc = 0.333 W/m2K. For the B-section without insulation Uc = 1.597 W/m2K, while the addition of polystyrene layer thickness d = 4cm Uc = 0.487 W/m2K. The calculation was performed by HRN EN ISO 14683:2000, which is the maximum allowed Umax = 0.500 W/m2K.

8. Summary

Cheap housing is one of the most constant needs in cities. One solution is an upgrade of deck roofs of existing buildings. Building upgrades must be approached with more attention. The reasons are deterioration of elements and parts of structure. This is especially important for facilities aged approximately 60 years, when the seismic force is not significantly taken into account. During designing and calculating, relevant impacts were not sufficiently taken into account, which resulted in an upgrade with dramatic damages to structural elements of the ground floor in the form of base material (brick) destruction. The designers proposed solutions of short-term character, inserting columns in the ground floor with trapezoidal extensions. The new solution includes a more complete recovery with concrete columns associated with the newly formed horizontal tie beams in the height of the first floor slab. In the lower part of column horizontal tie beams could not be installed because of the large damaged walls. Due to that the projected concrete vertical panel d = 10 cm thick is only 5cm cut into the wall. In order to improve the finishing of the facade and improve the energy efficiency of the building, the facade is further coated with polystyrene plates. Such interventions over a longer period of time gives a high quality facility for a relatively reasonable investment.

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