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Comprehensive Reconstruction of the *Hrvatski dom* Building in Petrinja

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Professional paper

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Comprehensive Reconstruction of the “Hrvatski dom“ Building in Petrinja

After the 2020 earthquake, the building of the Croatian Home in Petrinja was damaged, and a comprehensive reconstruction was carried out. This paper presents the entire restoration process - from the initial inspection and investigation works, through design development and selection of the strengthening concept, to the implementation of works within the framework of the structural strengthening supervision. The reconstruction of the Croatian Home building represents an example of a successfully completed comprehensive restoration and contributes to the revitalization of life in Petrinja, as the building holds particular importance for the city’s cultural life.

Key words:

earthquake, post-earthquake reconstruction, Petrinja, numerical model, strengthening methods

Stručni rad

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Cjelovita obnova zgrade Hrvatskog doma u Petrinji

Nakon potresa 2020. godine oštećena je zgrada Hrvatskog doma u Petrinji te je provedena njezina cjelovita obnova. U ovom radu prikazan je ukupan proces obnove, od početnog pregleda i istražnih radova, projektiranja, odabir koncepta pojačanja do provedbe radova u okviru projektantskog nadzora nad pojačanjem konstrukcije. Obnova zgrade Hrvatskog doma primjer je uspješne cjelovite obnove i doprinosi revitalizaciji života u Petrinji budući da je riječ o građevini od osobite važnosti za kulturni život grada.

Ključne riječi:

potres, potresna obnova, Petrinja, numerički model, metode pojačanja

1. Introduction

The earthquake that occurred on 29 December 2020 at 12:19 local time, with a moment magnitude of 6.2, affected the wider Petrinja area and resulted in extensive structural damage to the building stock, including the Croatian Home in Petrinja [1]. Following the severe earthquake-induced damage, a comprehensive renovation design was required to enable the full rehabilitation of the building, in accordance with the client's technical requirements and project brief [2].

The project scope encompassed the preparation of complete project documentation across all design phases, as well as monitoring the implementation of the adopted design solutions through design supervision of the structural strengthening works, which included the following:

Figure 1. Southern and northern facades (towards the atrium [3]

Figure 2. Eastern and western facades [3]

- collection and analysis of archival documentation, existing design drawings, relevant technical literature, and other available data required to determine the structural geometry and material properties necessary for the preparation of the project documentation
- identification of the existing building's structural system based on a visual inspection of the structure
- determination of the building geometry through on-site inspection and measurements for the preparation of the as-built documentation, including the identification of any deviations from the available reference drawings
- documentation of load-bearing and non-load-bearing structural elements
- identification and documentation of damage to both load-bearing and non-load-bearing structural elements observed during the inspection, with graphical representation in the drawings and preparation of the corresponding photographic documentation
- execution of investigative works on the structure
- preparation of a structural condition assessment report and damage analysis based on all the aforementioned data, providing the client with recommendations for further actions
- development of complete renovation project documentation, encompassing a separate project folder for the structural strengthening of the building structure
- the structural strengthening project for the building shall be developed at both the main design and detailed design levels

provision of continuous design-stage supervision during the execution of the building structural strengthening works

All buildings built before 1964, when the first serious seismic regulations were passed in Croatia, represent a significant engineering challenge in post-earthquake reconstruction. Erected in the 1950s (1951), the structure was designed to accommodate cultural functions together with related ancillary facilities. Over time, the building has accommodated a range of cultural and social functions, including a cinema, a municipal library, and hospitality facilities such as a city restaurant.

The paper outlines the full scope of the renovation process, including structural inspection and investigation, design development, evaluation and selection of the strengthening strategy, and its realization through design-stage supervision of the structural reinforcement works. The project solution was executed by tendering consortia composed of Toding d.o.o. and ing4studio d.o.o. Figures 1 and 2 present the condition of the building facade before the initiation of the renovation works.

2. Intended use and location of the existing structure

The existing building serves a public function, extends over a full city block, and is delineated by four urban traffic corridors together with the adjoining public pedestrian and parking areas. The building of the Croatian Open University "Hrvatski dom" is located in the city center in the immediate vicinity of the central city park, at Matije Gupca 2 (Figure 3).

Figure 3. Ground plan position of the building [4]

Figure 4. Condition of the great hall at the commencement of project implementation [3]

Figure 5. Condition of load-bearing and non-load-bearing walls resulting from loss of out-of-plane stability [3]

At the commencement of the project assignment in 2022, the building was not in operation due to the condition resulting from the earthquake. Certain load-bearing and non-load-bearing elements have completely lost their structural stability, rendering the building unfit for use.

The basement floor remains in relatively good structural condition; however, significant damage to both structural and non-structural components is present at the ground floor and first-floor levels, where a risk of local or progressive collapse has been identified. The great hall represents the most critical structural zone of the building due to the extent of damage sustained. In addition, partial failure of the roof structure following the earthquake allowed the ingress of atmospheric agents, leading to pronounced deterioration of both structural and non-structural elements. Moisture exposure and water infiltration have saturated the elements of the hall, resulting in uncontrolled structural failures within the hall space. Owing to the absence of protective measures, the resulting deterioration is progressive and accelerates the spread of damage throughout the remainder of the building.

The building was originally designed and constructed as a public facility intended to accommodate a variety of cultural and social functions, comprising numerous spaces designed for gatherings of large numbers of users. The building contains several halls with a capacity of over 300 users/seats, as well as a large multi-purpose hall in the southern extension with around 500 seats spread over two floors (ground floor and gallery). Beyond the previously mentioned facilities, the structure incorporates various public-use functions such as counseling and speech therapy services, as well as the necessary administrative, technical, and storage areas supporting their operation. In its current state, the building is formed as a free-standing atrium-type structure, with a central inner courtyard. Individual units differ in the number of floors, overall dimensions, and the configuration of their structural and architectural elements. The building is built on a gentle slope, with the basement level partially embedded into the surrounding terrain. The building is a free-standing structure with an open atrium in the center of the building, and consists of five interconnected units. The number of floors and plan dimensions of the building [1], as illustrated in Figure 6, are as follows:

1. Northern unit A – floor configuration: B + GF + 1, with plan dimensions of approximately 35.00 × 11.00 m
2. Northern unit B – floor configuration: B + GF, with plan dimensions of approximately 16.00 × 6.00 m
3. Western unit – floor configuration: B + GF, with plan dimensions of approximately 25.00 × 9.00 m
4. Southern unit – floor configuration: B + GF + gallery floor, with plan dimensions of approximately 40.00 × 19.00 m
5. Eastern unit – floor configuration: B + GF + attic, with plan dimensions of approximately 25.00 × 10.00 m; the open central atrium has plan dimensions of approximately 19.00 × 16.00 m.

The building is situated on a cadastral plot with a total area of 2,667 m². The building, including its annexes, external access stairs, and landings, occupies the cadastral plot along all neighboring boundaries, except for the southeastern side, which borders a public traffic surface. The building floor plan area, as recorded in the land registry, amounts to 1,870 m². All units are structurally interconnected; however, they differ in floor configuration and floor heights, as well as in their structural systems and the construction materials used.

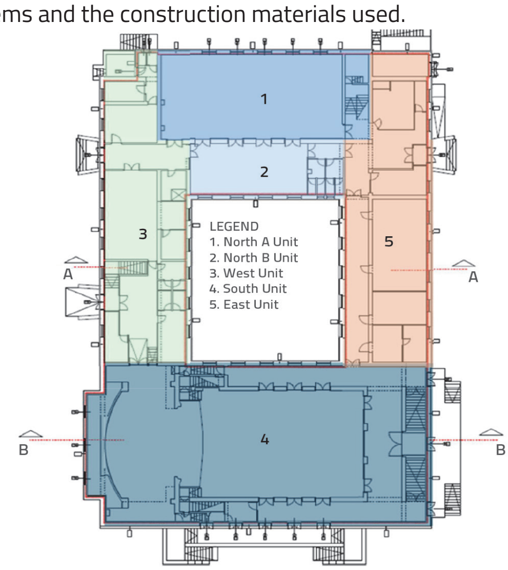


Figure 6. Plan view of the building units[1]

3. Description of the existing building structure

Structurally, the building is designed as a single dilatation, centered on a main atrium and surrounded by building segments exhibiting variations in structural systems, materials, and floor levels. Based on a portion of the available archival records, the building was initially conceived and executed through multiple construction phases. The basic documentation envisages 3 construction stages. The first phase (stage) was designed in 1951. This phase envisaged the realization of the northern building segment, incorporating the current library facilities as well as portions of the western and eastern annexes. In the second phase within the same year, the construction of the remaining parts of the building was envisaged, including the great hall and additional portions of the western and eastern annexes. Figure 7 presents the ground floor plan of the building, including all its constituent parts.

As early as 1952, the reconstruction of the basement and shelter areas was envisaged, and was partially implemented in accordance with the existing condition at the time. Archival records also include conceptual designs and sketches related to the reconstruction of individual building parts dating from 1959 and 1960. Due to the absence of complete documentation, it is not possible to reliably determine the extent to which these solutions were formally designed or subsequently implemented.

In the course of the building's continued use, additional reconstruction interventions were implemented, for which project documentation prepared in 1985 is partially preserved. These interventions included the design of portions of the slab elements in the basement and stage zones, the addition of a new ceiling slab currently functioning as the library floor, and the introduction of an additional stairway as a vertical communication element. It may be concluded that the reconstruction was not fully executed in accordance with the original project documentation, or that certain modifications were introduced during construction for which no corresponding design records are available.

Following the aforementioned reconstruction, documentation (architectural design) is available for the executed intervention, namely the adaptation of the attic into commercial/office space in the eastern annex. In addition to the construction or adaptation of the communication to this area, a reinforced concrete ceiling slab was executed on top of the existing timber soffit structure above the ground floor. No records of the structural analysis or construction detailing of the slab are available, with the exception of information regarding its thickness.

Based on the documented existing condition of the building, it can be reasonably assumed that additional minor structural interventions were carried out during its use, primarily to enable functional adjustments of the spaces in response to immediate needs.

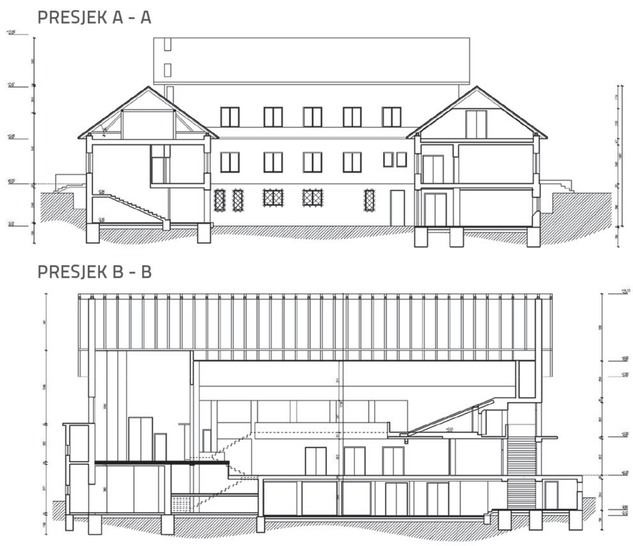


Figure 8. Section A-A and section B-B [5]

The main load-bearing vertical structure consists mainly of longitudinal and transverse walls of varying thickness along the building height, constructed of solid clay bricks of traditional format bonded with lime mortar. The columns in the basement in the great and small hall areas were constructed as reinforced concrete, probably during the aforementioned reconstructions. Above the columns, steel beams introduced at a later stage are also likely present, as indicated by the findings of the investigative works. Sections of the western, eastern, and southern units are seen in Figure 8.

The majority of the ceiling structure above the basement was built as a semi-prefabricated ceiling, most likely in the old format ("monta"). Portions of the basement ceiling are realized as timber beam structures, as demonstrated in the following elaborations. The basement area was subsequently reconstructed, where part of the ceiling was constructed

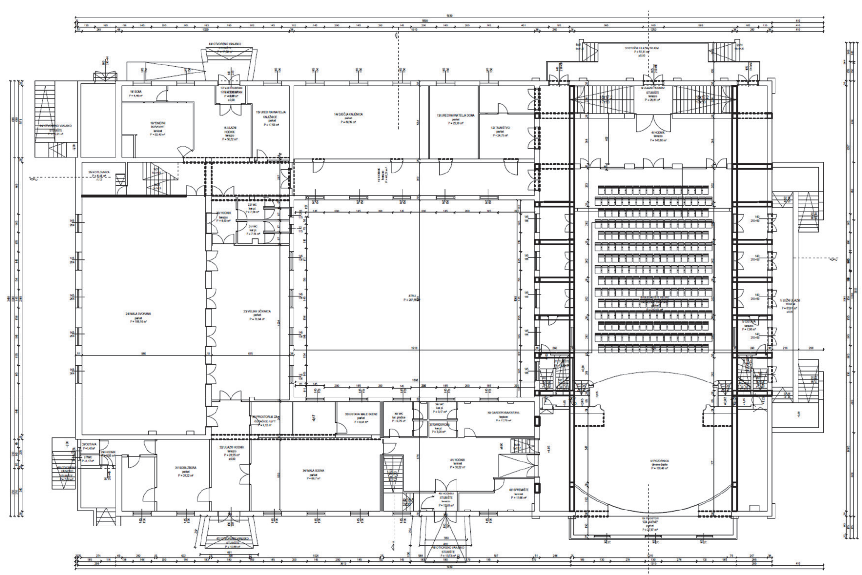


Figure 7. Ground floor plan [5]

as a semi-prefabricated "monta" ceiling in the storage area, and part was constructed as a reinforced concrete slab.

In the course of the ground-floor reconstruction, a reinforced concrete grillage beam system was additionally constructed to support the library floor, with corresponding reinforcement drawings available. Above the western and eastern annexes, the original structural solution comprised timber hanging roof trusses, with the ceiling soffit realized through a timber secondary framing attached to the truss tie beams. Portions of the gallery ceiling structure in the great hall are constructed as a timber floor system supported on a reinforced concrete beam grillage. The ceiling structure behind the stage is made of reinforced concrete panels, and the sloping panel of the auditorium is made entirely of wooden construction.

The basic documentation indicates that the ground floor of the hall was realized as a reinforced concrete column-and-beam frame system, incorporating infill walls of consistent thickness, a configuration that has been partially verified by investigative investigations. A timber structure was constructed on top of a reinforced concrete grillage structure that served as the floor and ceiling above the ground floor (under the first-floor gallery and auditorium). This structural system was repeated on the gallery all the way to the roof structure. The entire roof structure was constructed of wood, which is today in extremely bad shape because of leaks. The foundation structure is unknown. Basic historical dossiers indicate that the foundation structure's outlines can be seen as foundation strips that are wider than the wall's thickness.

4. Preparation of the assessment report on the existing atructural condition

4.1. Historical dossier

The assessment of the existing condition was carried out using all available historical project dossier obtained during the data collection process. Since the building is located in a city that experienced severe earthquake damage in addition to wartime devastation, it is possible that a considerable amount of the historical dossier has been lost. The following historical dossier was collected:

- Architectural drawings of stages I, II and III from 1951, 1952, 1959 and 1960.
- Structural analysis and rebar plans of the 1985 reconstruction
- Cost estimate for the 1985 reconstruction works
- Architectural drawings of the 2005 attic reconstruction

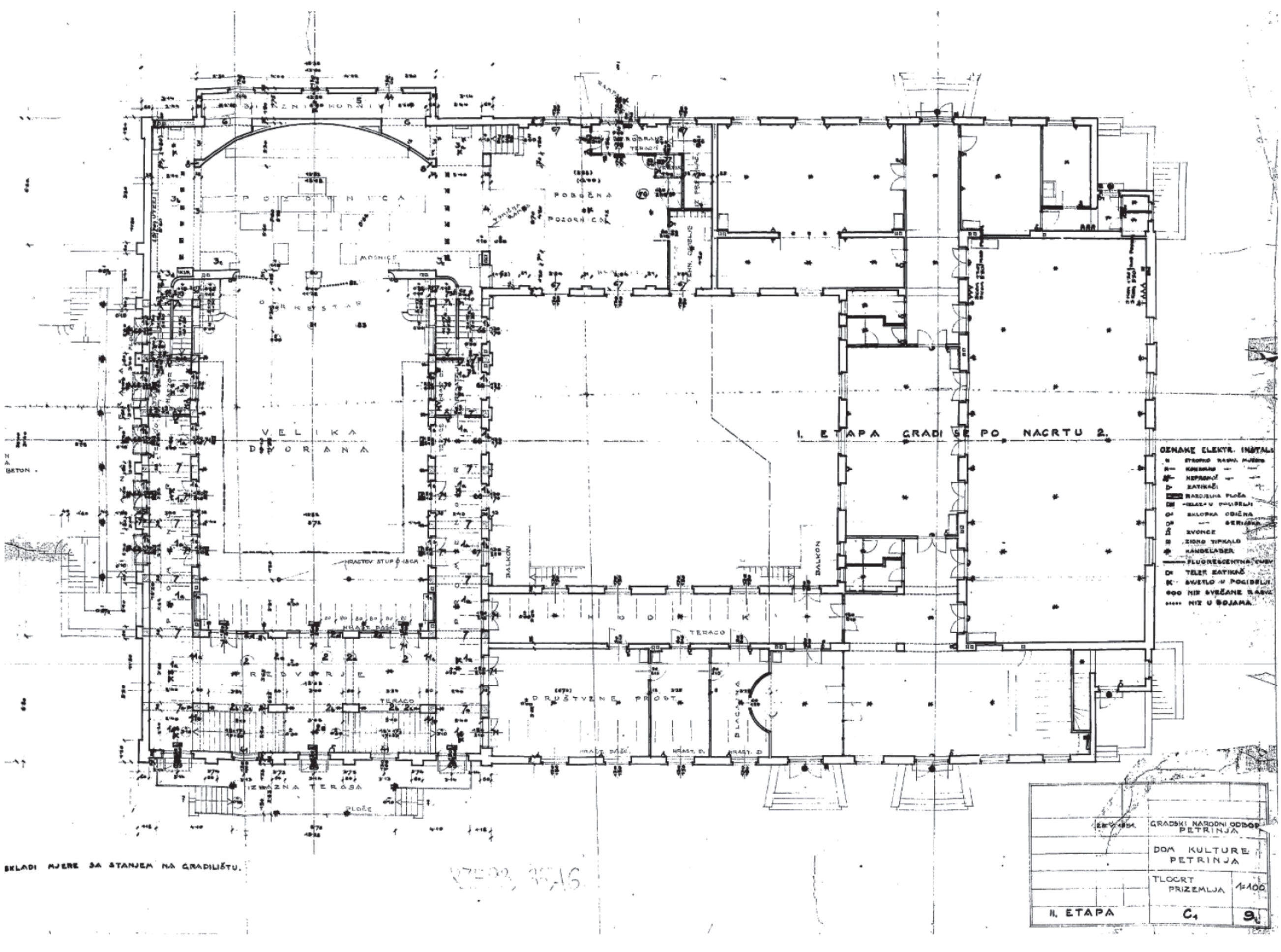


Figure 9. Basement floor plan – Phase II – historical dossier – 1951 [6]

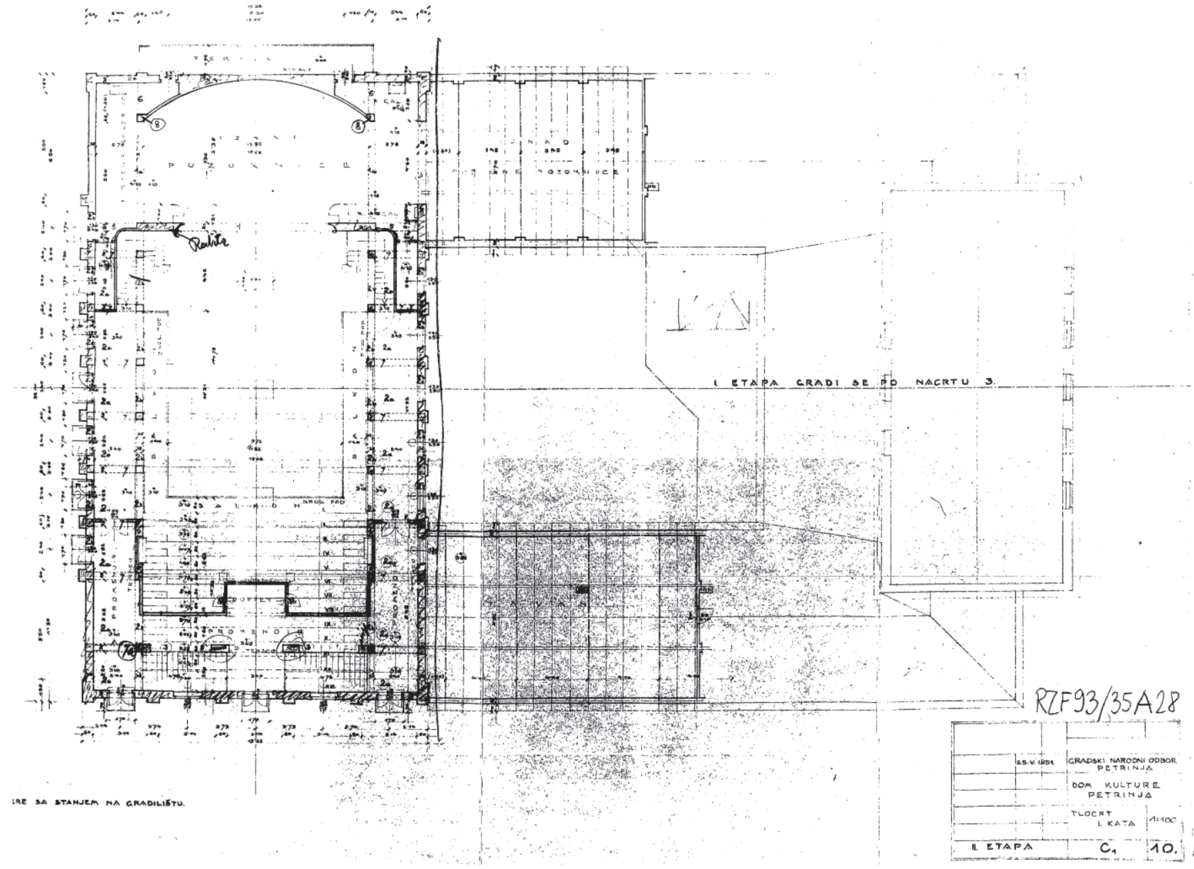


Figure 10. Ground floor plan – Phase II – historical dossier – 1951 [6]

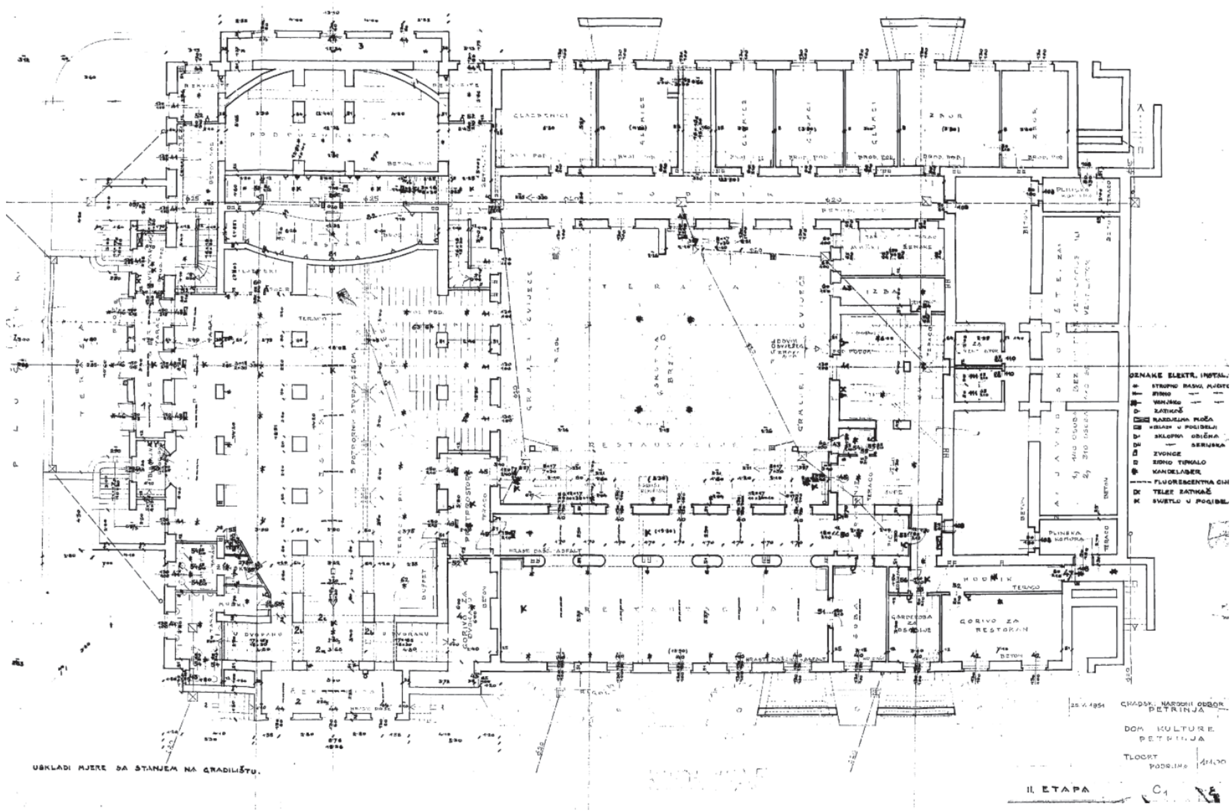


Figure 11. Floor plan of the 1st floor – Phase II – historical dossier – 1951 [6]

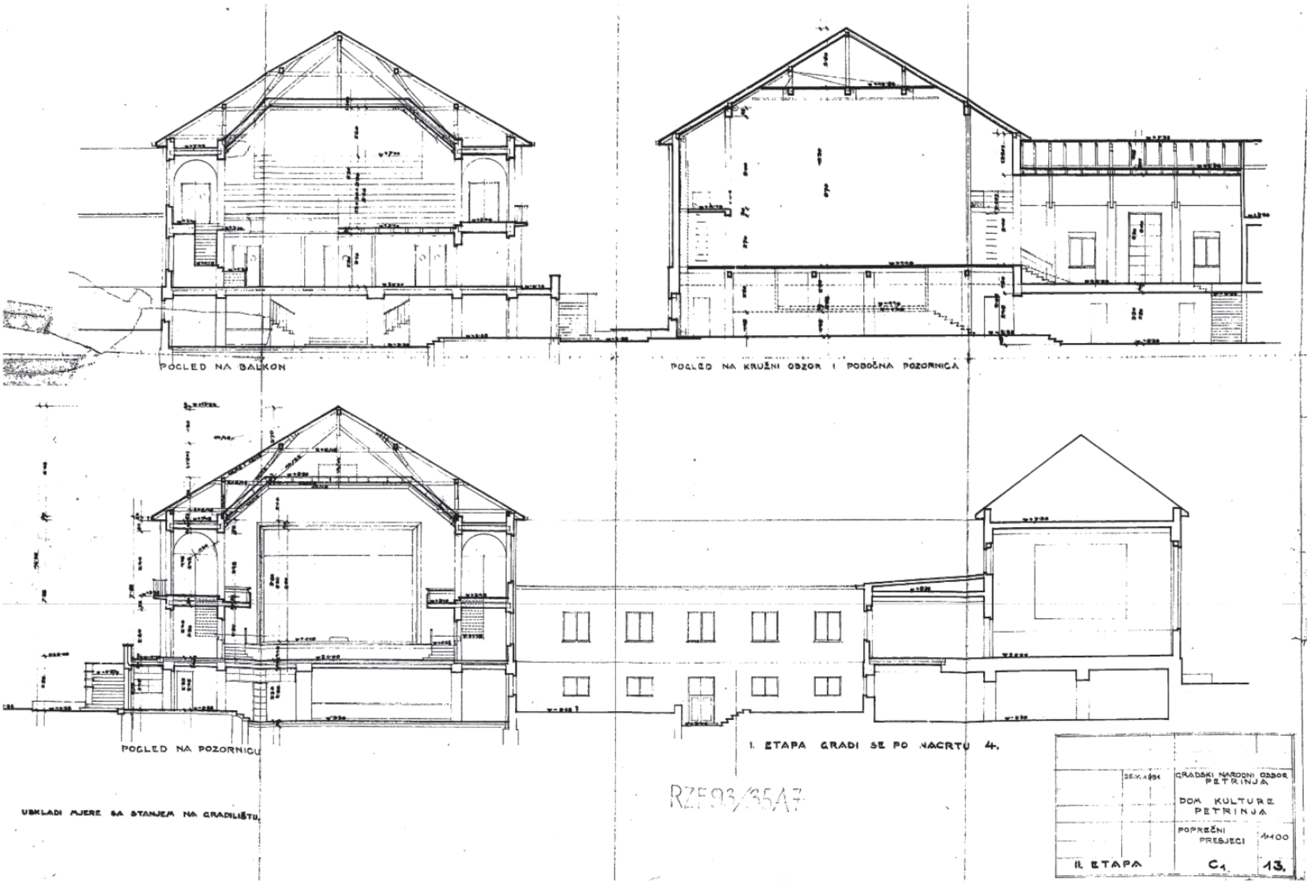


Figure 12. Sections – Phase II – historical dossier – 1951 [6]

- Assessment Report on the Existing Structural Condition prepared in August 2021, TD 110-21
- File I, II and III – Energy renovations (Architectural design, energy efficiency and thermal protection design, electrical design), ZOP 117-20.

analysis records are available that would allow the load-bearing system to be clearly determined.

The reconstructions that have been carried out (e.g. from 1985) have been partially completed, but an on-site inspection of the building reveals deviations from the dossier. The available structural analysis

Selected examples from historical dossier of the Ministry of Culture and Media – Cultural Heritage Microfilm Archive are presented in several figures.

Although the presented documentation appears relatively limited and illegible, it is nevertheless valuable and of considerable importance for the analysis of the building's existing structural system.

By analyzing existing historical dossiers, it can be concluded that the building has undergone multiple interventions throughout its history, both to the load-bearing structure and through various changes due to the repurposing of individual building parts. The basic historical dossier identifies the outlines of the load-bearing structural system (brick, concrete, beam grid, etc.), but no structural

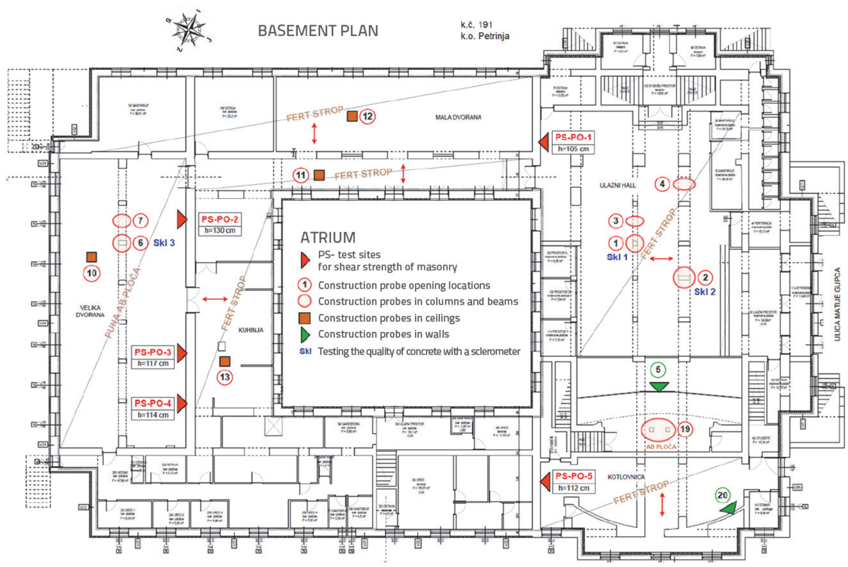


Figure 13. Example of the layout of basement investigation plans [7]

documentation, limited to the cover page, lacks the corresponding position plans and formwork drawings, which precludes the identification of individual structural elements based solely on the calculation component. Rebar plans can be partially read or logically concluded where individual positions are located, but given that the formwork plan is not available, it is also not entirely clear where individual load-bearing elements are planned.

4.2. Investigation works on the structure

Structural investigation works were conducted for the building in question, resulting in a standalone report prepared by the Laboratory for Structural Testing at the Faculty of Civil Engineering, University of Zagreb [7], which contains comprehensive results and analyses. Prior to the investigation, an analysis of existing documentation was conducted, and unknown parts were marked as sites for investigative work. As part of the investigation work, trial pits were constructed to determine the thickness of individual load-bearing walls, ceilings were opened to determine the composition of ceiling structures, and the shear strength of masonry walls was tested at individual positions of the building. Given that there is a reinforced concrete structure filled with masonry in some areas of the building, zones of the reinforced concrete structure with built-in reinforcement were also examined. The scope of the investigation included trial pits for determining load-bearing wall thicknesses, exposure of floor structures to establish their composition, and in-situ shear strength testing of masonry at designated positions throughout the building.

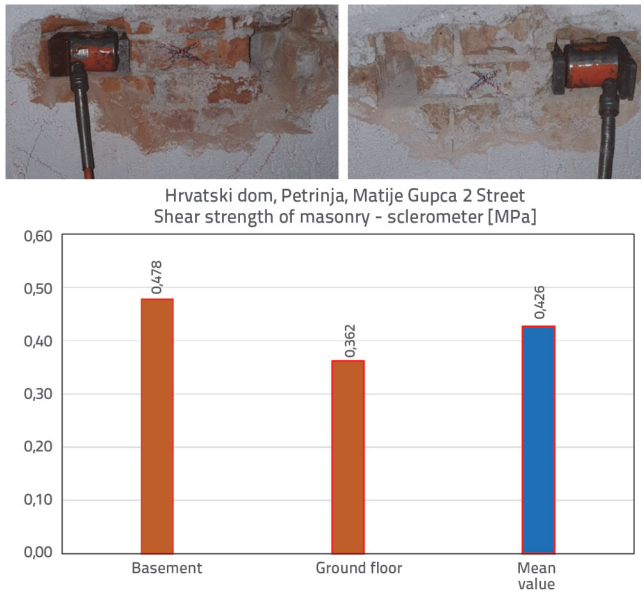


Figure 14. Examples of shear strength testing of masonry walls [7]

Investigative works are planned on all structural elements for which, based on previous work, the material or their role in the structural system of the building could not be reliably determined. The goal of these works is to create the most accurate calculation model possible, which will be used to assess the current state of the structure. Examples of the implementation of investigation work are shown in Figures 15 and 16.

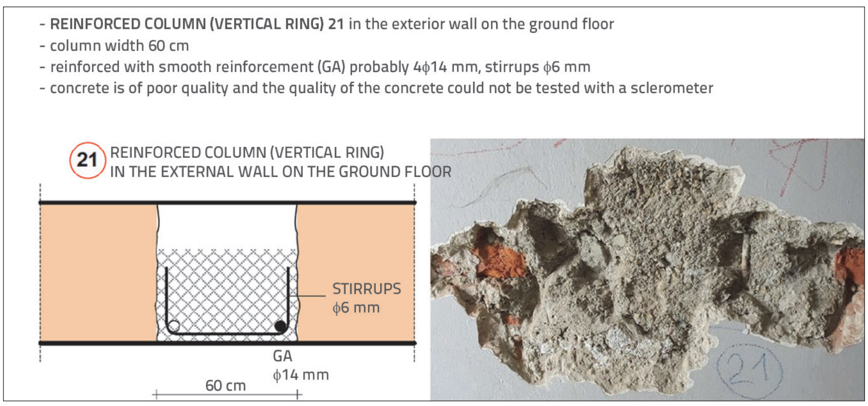


Figure 15. Examples of conducting investigation works on load-bearing elements of concrete elements within masonry walls [7]

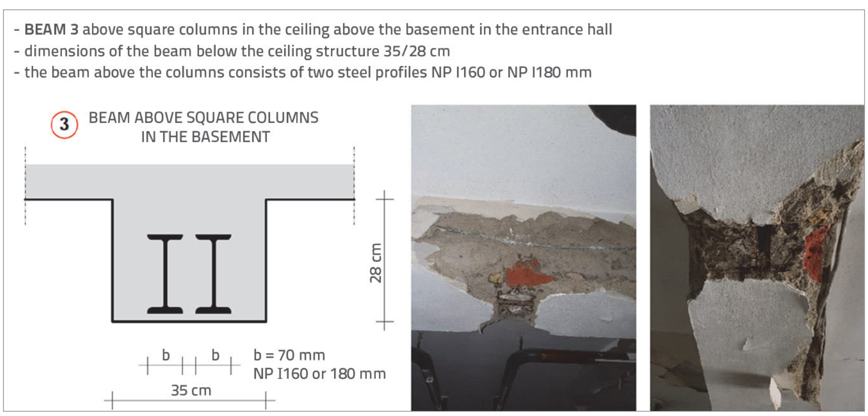


Figure 16. Examples of conducting investigation works on load-bearing elements of steel beams concealed by cladding [7]

4.3. Target level of building structure reinforcement

In accordance with the Technical Regulation for Building Structures and the Regulation amending and supplementing the Technical Regulation for Building Structures [8], repair and strengthening measures are intended to achieve the required mechanical resistance and stability of the building under seismic action corresponding to the significant damage performance level. HRN EN 1998-3 [9] defines three limit states (LS) corresponding to different levels of structural damage:

- Near collapse (NC)
- Significant damage (SD)
- Limited damage (LD).

According to the Croatian National Annex to HRN EN 1998-3, the assessment and renovation of buildings require verification of the Significant Damage (SD) and Damage Limitation (DL) limit states. Furthermore, the amendment to the Technical Regulation for Building Structures defines the levels of post-earthquake rehabilitation of building structures. In accordance with the aforementioned Regulation, due to the building's public use, renovation is required to be carried out at Level III – structural strengthening, which in this specific case corresponds to a peak ground acceleration of agR = 0.114 g.

Table 1. Parameters used for the computational model

A_g/g	0,114
Ground type	C
Behavior factor (q)	1.5
Importance factor of a building	1.2 (class III)
Knowledge level (KL)	2
Trust factor (TF)	1.2
$F_{vk,0}$ [N/mm²]	0.18

The implementation of renovation level III requires achieving a significant damage index (SDI) not less than 0.75. At this level of renovation, in addition to verification of the significant damage (SD) limit state, verification of the damage limitation (DL) limit state is also required, in accordance with HRN EN 1998-3. The seismic action is defined for an earthquake with a reference probability of exceedance of 10 % in 10 years (corresponding to a reference return period of 95 years), taking into account the importance factor for buildings as defined in HRN EN 1998-1 [10]. The significant structural damage index (SDI) is the ratio of the design seismic resistance to the structural requirements for the significant damage limit state. The design seismic resistance is the value of the seismic action expressed as a peak ground acceleration of type A for which the structure reaches the limit state of significant damage. The structural requirement for the significant damage limit state is defined by the comparative seismic action, expressed in terms of the reference peak ground acceleration of soil type A for a reference return period of 475 years (corresponding to a probability of exceedance of 10 % in 50 years). The results of the investigative works, and a verification calculation of the seismic resistance of the existing condition, it was concluded that the building is suitable for rehabilitation to Level III in accordance with the Technical Regulation for Building Structures.

4.4. Computational model and in-plane seismic verification of load-bearing masonry elements

It is necessary to make the computational model as realistic as possible based on all previously collected data. The selection of model boundary conditions, the method of modeling ceiling structures, as well as the definition of the seismic calculation method with all parameters greatly affect the obtained results.

Semi-precast Monta floor structures and timber joist floors were modeled as orthotropic, with the corresponding stiffness parameters of such structures (Ex, Ey, G) being determined. Partition walls in the design model were not modeled in the same way as the roof structure. They were taken as a load according to the load analysis. The influence of partition walls has a positive effect on the structural resistance, therefore, their omission in the analysis is on the safe side and justified.

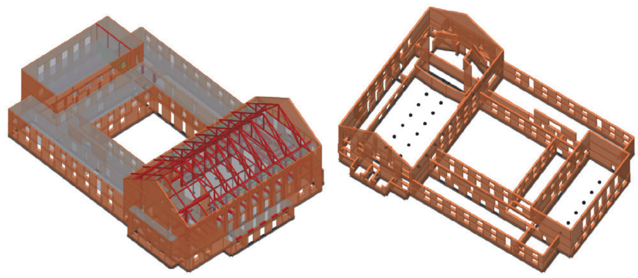
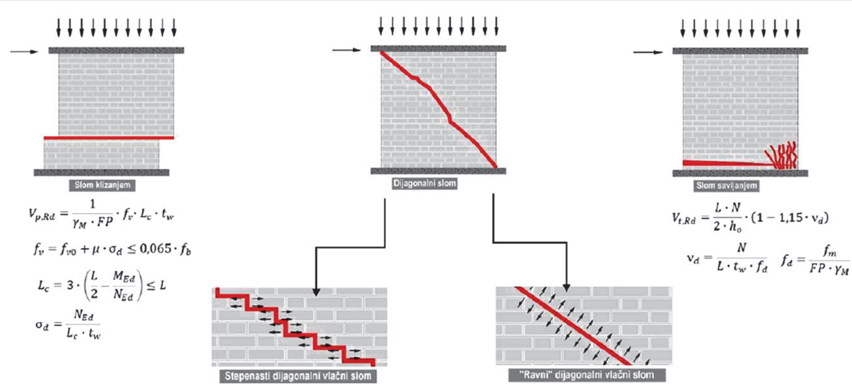


Figure 17. 3D computational model for the development of the EOPS [1]

Modal analysis using the response spectrum was carried out in accordance with the applicable regulations for seismic design (HRN EN 1998-1), using the design response spectrum. CQC (*Complete Quadratic Combination*) modal combination method was used, satisfying the criterion of 90 % structural mass participation. An eccentricity of 5 % was taken into account in the calculation, and the behavior factor was assumed to have



$$V_{p,Rd} = \frac{1}{\gamma_M \cdot FP} \cdot f_v \cdot L_c \cdot t_w$$

$$f_v = f_{v0} + \mu \cdot \sigma_d \leq 0{,}065 \cdot f_b$$

$$L_c = 3 \cdot \left(\frac{L}{2} - \frac{M_{Ed}}{N_{Ed}}\right) \leq L$$

$$\sigma_d = \frac{N_{Ed}}{L_c \cdot t_w}$$

$$V_{t,Rd} = \frac{L \cdot N}{2 \cdot h_o} \cdot (1 - 1{,}15 \cdot \nu_d)$$

$$\nu_d = \frac{N}{L \cdot t_w \cdot f_d} \quad f_d = \frac{f_m}{FP \cdot \gamma_M}$$

$$V_{t2,Rd} = \frac{L \cdot t_w}{b} \cdot \left(\frac{f_{v0}}{\gamma_M \cdot FP \cdot (1 + \mu_j \cdot \phi)} + \frac{\mu_j}{1 + \mu_j \cdot \phi} \cdot \sigma_0\right) \leq V_{d,lim}$$

$$V_{t1,Rd} = L \cdot t_w \cdot \frac{f_t}{\gamma_M \cdot FP \cdot b} \cdot \sqrt{1 + \frac{\gamma_M \cdot FP \cdot \sigma_0}{f_t}}$$

$$V_{d,lim} = L \cdot t_w \cdot \frac{f_{bt}}{\gamma_M \cdot FP \cdot 2{,}3 \cdot b} \cdot \sqrt{1 + \frac{\gamma_M \cdot FP \cdot \sigma_0}{f_{bt}}}$$

$$\sigma_0 = \frac{N_{Ed}}{L \cdot t_w}$$

Legenda:

N_{Ed} - design longitudinal force
M_{Ed} - design moment
V_{Ed} - design shear force
L - wall length
t_w - wall thickness
L_c - length of the compressive part of the wall
S_d - design compressive stress
f_{vk} - characteristic shear strength of the masonry
S_0 - mean compressive stress
n_d - normalized longitudinal force
h - wall height
Ø - clamping coefficient
V_{Ed}*IZO - reduced shear force
authoritative V_{Rd} - relevant value of wall bearing capacity
$V_{t,Rd}$ - wall bearing capacity for bending failure
$V_{p,Rd}$ - wall bearing capacity for sliding failure
$V_{t1,Rd}$ - wall bearing capacity for flat diagonal tensile failure
$V_{t2,Rd}$ - wall bearing capacity for stepped diagonal tensile failure
$V_{t3,Rd}$ - limit value of bearing capacity $V_{t2,Rd}$

Figure 18. Expressions for in-plane failure modes of existing masonry [11]

a value of 1.5 in accordance with HRN EN 1998-3. The cracked state of the structure was taken into account, as it was assumed that the building will reach a state of significant damage (SD). For the modal analysis, the in-plane longitudinal stiffness of walls was taken as 50 % of the elastic stiffness, and the flexural stiffness of floor slabs, columns, and beams was taken as 50 % of the elastic stiffness. The importance factor of the building is 1.2 in accordance with HRN EN 1998-3, as it is classified as a public building.

Table 2. Parameters of existing masonry walls [11]

Type of masonry	Solid brick wall	
E	1500	Young's modulus of elasticity [MPa]
G	500	Shear modulus [MPa]
γ	18	Unit weight [kN/m^3]
$f_{vk,0}$	0.18	Characteristic initial shear strength [kN/m^3]
f_b	15.0	Mean compressive strength of wall elements [N/mm^2]
f_m	3.40	Mean value of masonry compressive strength [N/mm^2]
f_{bt}	1.50	Mean tensile strength of the wall element (0.1 fb) [N/mm^2]
f_t	0.11	Mean value of tensile strength of masonry [N/mm^2]
μ	0.50	Internal friction coefficient (0.5 for old masonry)
f_d	1.89	Design compressive strength of masonry [N/mm^2]

The load-bearing capacity of walls and lintels was determined based on failure mechanisms, which may be diverse. For the assessment of the resistance of a given load-bearing wall, all possible failure mechanisms were considered, and the governing mechanism was selected. The resistances of walls, lintels, and beams were taken in accordance with HRN EN 1998-3. The load-bearing capacities of reinforced concrete beam sections were calculated based on the concrete strength and the assumed minimum reinforcement in accordance with HRN EN 1998-3 and HRN EN 1992-1-1.

It is important to note that certain modeled elements were defined parametrically in order to obtain the most accurate possible results for vertical wall elements, reinforced concrete columns, and beams. This means that the ceiling structures were modeled to simulate the stiffness of the ceiling diaphragm and its weight. The effects in the substitute plate elements are not governing and are not used for dimensioning under static (vertical) actions.

In the seismic computational models, the roof structures, except for the great hall, were not modeled, as they do not have a significant influence on the overall seismic behavior of the building, apart from the load, which was applied as an equivalent line load.

All wooden diaphragms in the computational model were assumed to be connected to the exterior walls, although this condition is not present in reality. Given that the facade walls in these areas have separated from the subfloor by several centimeters, there are clear indications of out-of-plane failure. Additionally, the stiffness of the wooden diaphragms must be assumed for the purposes of the calculations, but as these are wooden beams with planks, it is impossible to precisely estimate their stiffness in both orthogonal directions.

The computational model is used exclusively for the analysis of vertical elements, masonry walls in plane, and for the design of reinforced concrete columns and beams. The out-of-plane stability of walls will be analyzed separately using a realistic support model, since this mechanism represents the governing failure criterion under seismic action, as confirmed on the structure by recorded displacements after the earthquake.

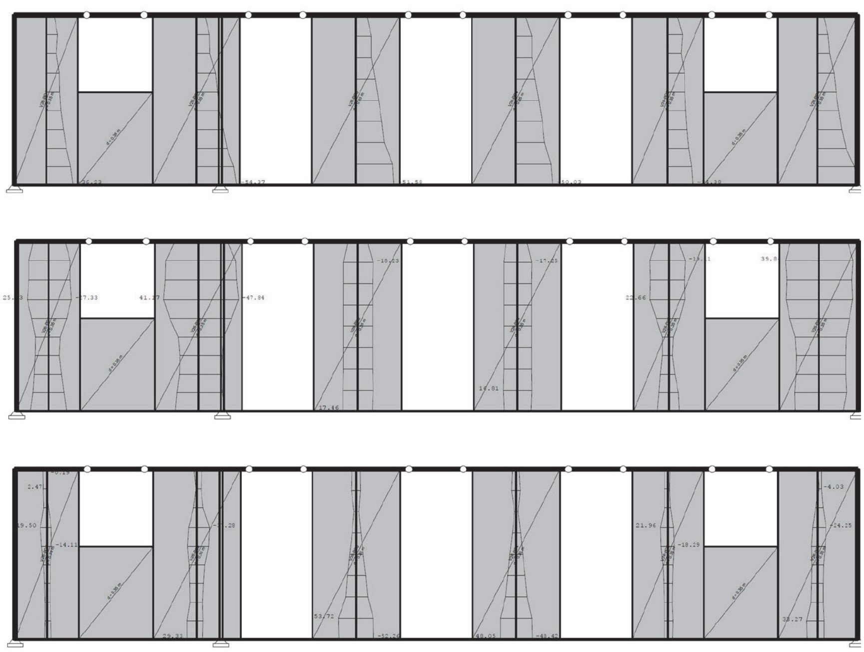


Figure 19. Example of N_{Ed}, T_{Ed} and M_{Ed} forces in walls, based on which the calculation is performed[11]

Due to the complexity of modeling and checking the load-bearing capacity of lintels and parapets in walls with openings, they will not be considered. Cracking, that is, failure beyond the level of elastic load-bearing capacity of lintels and parapets, may occur, however, their complete failure must not occur, as this could potentially compromise the stability of the floor structure and lead to the collapse of part of the building. For this reason, it is assumed that lintels and parapets will crack or fail prior to the vertical wall piers. In order to reduce the influence of masonry lintels and parapets and to take into account the effect described above, they were modeled so that 30 % of the axial stiffness and bending stiffness are taken for the seismic combination.

Based on these expressions, all walls were tabulated and calculated using

CALCULATION OF THE SHEAR LOAD CAPACITY OF EXISTING WALLS IN THE PLANE - IZO < THAT THE WALLS SATISFY

Input parameters:

$f_{vk,0}$ =	0,18	N/mm²	characteristic initial shear strength
f_b =	15,00	N/mm²	mean compressive strength of masonry elements
f_m =	3,40	N/mm²	mean compressive strength of masonry
f_{bt} =	1,50	N/mm²	mean tensile strength of a wall element (0.1 fb)
f_t =	0,11	N/mm²	mean tensile strength of masonry
μ =	0,50	N/mm²	coefficient of internal friction (0.5 for old masonry)
γ_M =	1,50	N/mm²	material safety coefficient
FP =	1,20	N/mm²	confidence factor
IZO=	0,25		IZO coefficient
f_d =	1,89	N/mm²	design compressive strength of masonry:
a_{gR} =	0,15 g		peak ground acceleration

WALLS - DIRECTION X

INPUT DATA								SLIDING FAILURE					DIAGONAL TENSILE FRACTURE				BENDING FRACTURE		CONTROL		
WALL	N_{Ed} [kN]	M_{Ed} [kNm]	V_{Ed} [kN]	L [cm]	t_w [cm]	h [cm]	ø	V_{Ed} * IZO [kN]	Lc [cm]	σd [N/mm²]	fvk [N/mm²]	$V_{p,Rd}$ [kN]	σ_o [N/mm²]	$V_{t1,Rd}$ [kN]	$V_{d,lim}$ [kN]	$V_{t2,Rd}$ [kN]	ν_d ($N_{Ed}/L\ t_w\ f_d$)	$V_{f,Rd}$ [kN]	Relevant V_{Rd} [kN]	Usability	CONDITION VRd > VEd
H2-PR1	45,00	28,00	70,00	112,00	40,00	330,00	1,00	17,5	112,00	0,0100	0,1850	46,1	0,0100	15,1	58,3	15,2	0,05	14,3	14,3	122,05%	✗
H2-PR2	103,00	68,00	102,00	208,00	40,00	330,00	1,00	25,5	208,00	0,0124	0,1862	86,1	0,0124	55,7	203,6	57,1	0,07	60,0	55,7	45,74%	✓
H2-PR3	95,00	80,00	76,00	190,00	40,00	330,00	1,00	19,0	190,00	0,0125	0,1863	78,6	0,0125	46,7	170,0	47,9	0,07	50,5	46,7	40,71%	✓
H2-PR4	106,00	86,00	74,00	218,00	40,00	330,00	1,00	18,5	218,00	0,0122	0,1861	90,1	0,0122	60,9	223,4	62,3	0,06	64,8	60,9	30,40%	✓
H2-PR5	106,00	40,00	75,00	195,00	40,00	330,00	1,00	18,8	195,00	0,0136	0,1868	80,9	0,0136	50,6	180,1	52,3	0,07	57,5	50,6	37,08%	✓
H2-PR6	90,00	40,00	65,00	185,00	40,00	330,00	1,00	16,3	185,00	0,0122	0,1861	76,5	0,0122	43,8	160,9	44,8	0,06	46,7	43,8	37,07%	✓
H2-PR7	102,00	43,00	60,00	220,00	40,00	330,00	1,00	15,0	220,00	0,0116	0,1858	90,8	0,0116	61,0	226,9	62,2	0,06	63,2	61,0	24,58%	✓
H2-PR8	100,00	47,00	60,00	200,00	40,00	330,00	1,00	15,0	200,00	0,0125	0,1863	82,8	0,0125	51,7	188,4	53,0	0,07	56,0	51,7	29,01%	✓
H2-PR9	100,00	53,00	45,00	191,00	40,00	330,00	1,00	11,3	191,00	0,0131	0,1865	79,2	0,0131	47,9	172,3	49,3	0,07	53,3	47,9	23,49%	✓
H2-PR10	160,00	107,00	152,00	315,00	40,00	330,00	1,00	38,0	315,00	0,0127	0,1863	130,4	0,0127	128,9	467,8	132,4	0,07	140,9	128,9	29,47%	✓
H2-PR11	204,00	127,00	229,00	144,00	40,00	230,00	1,00	57,3	144,00	0,0354	0,1977	63,3	0,0354	57,4	156,0	70,4	0,19	100,2	57,4	99,65%	✓
H2-PR12	546,00	675,00	284,00	213,00	40,00	230,00	1,00	71,0	213,00	0,0641	0,2120	100,4	0,0641	163,4	380,2	238,9	0,34	308,4	100,4	70,74%	✓
H2-PR13	195,00	203,00	101,00	328,00	40,00	230,00	1,00	25,3	328,00	0,0149	0,1874	136,6	0,0149	211,8	735,9	221,2	0,08	252,9	136,6	18,48%	✓
K3-PO1	353,00	300,00	307,00	542,00	35,00	430,00	1,00	76,8	542,00	0,0186	0,1893	199,5	0,0186	293,9	958,2	316,3	0,10	394,5	199,5	38,47%	✓
K3-PR1	250,00	195,00	270,00	542,00	25,00	230,00	1,00	67,5	542,00	0,0185	0,1892	142,4	0,0185	391,2	1278,6	420,5	0,10	523,0	142,4	47,39%	✓
K2-PO1	463,00	216,00	172,00	500,00	35,00	430,00	1,00	43,0	500,00	0,0265	0,1932	187,9	0,0265	287,1	846,3	329,1	0,14	451,7	187,9	22,89%	✓
K2-PR1	250,00	136,00	172,00	500,00	25,00	230,00	1,00	43,0	500,00	0,0200	0,1900	131,9	0,0200	343,3	1096,4	373,6	0,11	477,3	131,9	32,59%	✓
H7-PO1	660,00	1437,00	441,00	1289,00	35,00	430,00	1,00	110,3	1289,00	0,0146	0,1873	469,5	0,0146	1522,6	5312,7	1587,2	0,08	1802,3	469,5	23,48%	✓
H7-PO2	110,00	31,00	30,00	173,00	35,00	430,00	1,00	7,5	173,00	0,0182	0,1891	63,6	0,0182	29,7	97,4	31,8	0,10	39,4	29,7	25,28%	✓
H8-PO1	712,00	638,00	464,00	354,00	50,00	330,00	1,00	116,0	354,00	0,0402	0,2001	196,8	0,0402	319,5	837,7	405,1	0,21	576,7	196,8	58,95%	✓
H8-PO2	1144,00	1681,00	608,00	556,00	50,00	330,00	1,00	152,0	556,00	0,0412	0,2006	309,8	0,0412	796,0	2074,2	1015,5	0,22	1444,6	309,8	49,07%	✓
H8-PO3	305,00	302,00	190,00	236,00	50,00	330,00	1,00	47,5	236,00	0,0258	0,1929	126,5	0,0258	117,9	350,0	134,5	0,14	183,8	117,9	40,28%	✓
H8-PO4	286,00	205,00	140,00	221,00	50,00	330,00	1,00	35,0	221,00	0,0259	0,1929	118,4	0,0259	103,5	306,9	118,1	0,14	161,4	103,5	33,82%	✓
H8-PO5	272,00	260,00	141,00	250,00	50,00	330,00	1,00	35,3	250,00	0,0218	0,1909	132,6	0,0218	123,6	385,3	136,5	0,12	178,8	123,6	28,52%	✓
H8-PO6	228,00	76,00	47,00	165,00	50,00	330,00	1,00	11,8	165,00	0,0276	0,1938	88,8	0,0276	59,2	172,5	68,5	0,15	94,8	59,2	19,84%	✓
H8-PO7	720,00	763,00	464,00	1055,00	50,00	330,00	1,00	116,0	1055,00	0,0136	0,1868	547,5	0,0136	1853,2	6591,6	1917,2	0,07	2110,5	547,5	21,19%	✓
							Σ	1209,75										Σ	3475,23	34,81%	

#DIV/0!	NO PRESSURE ZONE

Legend:

N_{Ed} - design longitudinal force
M_{Ed} - design moment
V_{Ed} - design shear force
L - wall length
t_w - wall thickness
h - wall height
Ø - clamping coefficient
V_{Ed}*IZO - reduced shear force
authoritative V_{Rd} - relevant value of wall bearing capacity
L_c - length of the compressive part of the masonry
σ_d - design compressive stress
f_{vk} - characteristic shear strength of the masonry
σ_0 - mean compressive stress
ν_d - normalized longitudinal force
$V_{p,Rd}$ - load-bearing capacity of a wall against sliding failure
$V_{t1,Rd}$ - load-bearing capacity of a wall against flat diagonal tensile failure
$V_{t2,Rd}$ - load-bearing capacity of a wall against stepped diagonal tensile failure
$V_{t3,Rd}$ - limit value of load-bearing capacity Vt2,Rd
$V_{t,Rd}$ - load-bearing capacity of a wall against bending failure

Figure 20. Example of the computational assessment of individual walls according to the failure modes of existing masonry [11]

the forces obtained from the spatial computational model with respect to the in-plane failure criteria, and the corresponding utilization ratios are presented for the peak ground acceleration with SDI = 1.0.

4.5. Calculation of out-of-plane wall failure mechanisms

The currently valid HRN EN 1998-3 does not include recommendations for the assessment of local failure mechanisms, so the procedures and requirements on the basis of which the wall overturning checks were performed are based on the Italian technical code NTC 2008. However, it is known that the next revision of Eurocode 1998-3 will include a similar procedure for the verification of out-of-plane wall failure in existing buildings.

The out-of-plane failure analysis is performed using kinematic analysis of a single-degree-of-freedom mechanism. In this case, structural components, i.e., walls, are modeled as rigid or partially rigid blocks. The geometry of the blocks, as well as the boundary conditions, are defined based on actual or potential crack locations. Accordingly, the first step in the analysis is the selection of the mechanism type allowing for relative rotation and/or sliding, with the inclusion of all forces acting on it. Among several possible mechanisms, the governing one is that with the lowest activation acceleration, or alternatively, the one with the lowest displacement capacity. For this purpose, an equivalent linear-elastic single-degree-of-freedom system is defined to replace the actual mechanism in the limit state verifications.

The kinematic analysis can be carried out using the following two approaches:

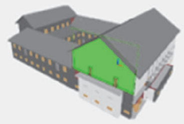
1. Linear kinematic analysis – the activation factor of the mechanism is calculated, and the verification of the corresponding demand is carried out by comparing the values of spectral acceleration or shear force for the relevant limit state. This analysis is based on force values (*Force-Based Design*) and, as such, represents a linear type of analysis.
2. Nonlinear kinematic analysis – this calculation takes into account the displacement capacity of the mechanism after its activation. The analysis considers the variation of the activation factor (or shear force) with respect to the control displacement up to the complete loss of stability of the mechanism. The verifications are performed by comparing the values of the control displacement for the corresponding limit state (*Displacement Based Design*).

Presented below are the input data and the results of the linear kinematic analysis of a single wall with respect to out-of-plane failure. The calculation parameters were obtained from the geometry of the volumetric model, with additional inertial effects due to the roof weight and possible thrust actions from the roof surfaces being neglected.

Local calculations performed for several local wall failure mechanisms showed that they do not satisfy the safety check for a return period of 475 years. The calculation of selected walls that are not laterally restrained within their

Parameter input values: Mechanism 1	
Thickness of blocks	b = 40 cm
Weight of blocks	$G_1=G_2=212{,}84$ kN $G_3= 103{,}22$ kN $G_4= G_6=20{,}52$ kN $G_5= 140{,}77$ kN $G_7= 98{,}09$ kN $G_8= 76{,}46$ kN $G_9= 104{,}27$ kN $G_{10}= 72{,}66$ kN
Value of virtual displacements	$\delta_{x,1}=\delta_{x,2}=0{,}623$ $\delta_{x,3}=\delta_{x,4}=\delta_{x,5}=\delta_{x,6}=\delta_{x,7}=0{,}312$ $\delta_{x,8}=\delta_{x,9}=\delta_{x,10}=0{,}093$ $\delta_{y,1}=\delta_{y,2}=\delta_{y,3}=\delta_{y,4}=\delta_{y,5}=\delta_{y,6}=\delta_{y,7}=\delta_{y,8}=\delta_{y,9}=\delta_{y,10}=0{,}0186$
Altitude	Z = 6,60 m
Mechanism calculation	
Mechanism activation factor	$\alpha_0=0{,}048$
Spectral acceleration	$a_0^* = 0{,}50\ m/s^2$
Required spectral acceleration 475 yr. p.p.	$a_{0,min}^* = 4{,}31\ m/s^2$
Condition $a_0 \geq a^*_{0min}$	NOT SATISFACTORY
Security Index	0,11

Wall geometry and assumed mechanism



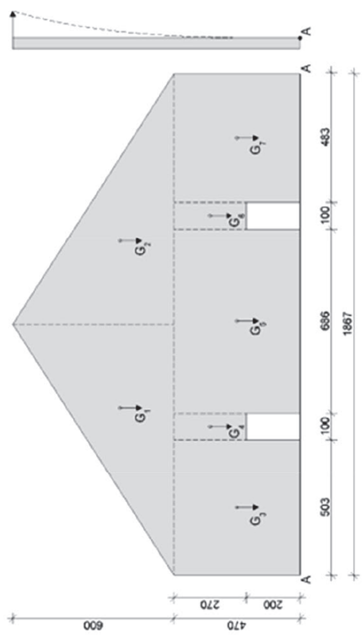


Figure 21. Calculation of the out-of-plane wall failure mechanism [11]

span has shown that they can be activated at a peak ground acceleration of 0.152 g. Furthermore, the wall damage that occurred after the earthquake indicates partial separation and out-of-plane failure of the observed walls. Mechanism 01, a dovetail mechanism with the gable wall, satisfied 11 % of the demand, while Mechanism 02, the unrestrained facade wall, satisfied 85 % of the demand based on linear analysis. It is important to emphasize that the verification of out-of-plane failure is primarily necessary because the analysis of the global model is valid only under the assumption that the structure responds as an integral whole.

4.6. Description of the expected structural interventions with technical solutions for the building's structural renovation

By adhering, on the one hand, to the guidelines and conclusions set out in the conservation requirements and, on the other hand, to the conditions for achieving the level of renovation prescribed by regulations, it is necessary to prepare a renovation design in which the required structural interventions will be determined on the basis of computational analyses in order to achieve the required condition. The building, in its current condition, is completely unusable, primarily due to non-structural elements that prevent its use because of the risk of collapse.

Based on computational methods and according to the criteria of local out-of-plane wall failure mechanisms, the existing condition of the building corresponds to an index of significant structural damage (SDI) equal to IZO = 0.11. For renovation level III, it was necessary to achieve an index of significant damage (IZO) of at least IZO = 0.75; therefore, the building currently exhibits only 15 % of the required seismic resistance. Therefore, the measures to be implemented must ensure the horizontal stiffness of the building as a whole. Parts of the building that must be completely removed because they are damaged to an extent that makes renovation impossible are:

- Gable walls of the great hall
- Roof structures of the great hall
- Partition walls
- Ceiling soffits
- Timber structures of auditoriums and galleries.

The existing masonry must be cleaned down to the load-bearing structure, inspected, and the existing crack damage consolidated, after which it must be structurally strengthened using one of the reinforcement methods. In order to ensure a uniform distribution of horizontal seismic forces to the masonry, the ceiling diaphragms must be strengthened, as they play a key role in load redistribution as well as in preventing out-of-plane wall overturning.

All surface layers must be removed to provide access to the reinforced concrete load-bearing elements (columns, beams, and arches), enabling inspection and consolidation of existing crack damage, after which they must be structurally strengthened using one of the strengthening methods. Considering other methods of strengthening stiffening elements, it will be determined which of the strengthening methods will be applied to RC elements.

Conceptual sketches of structural solutions were created, some of which are shown in Figure 22, in order to achieve the level of seismic resistance required by the regulation while also keeping in mind the need to preserve valuable construction elements in this building. The conservation study's recommendations were given top priority during the analysis and development of the solution.

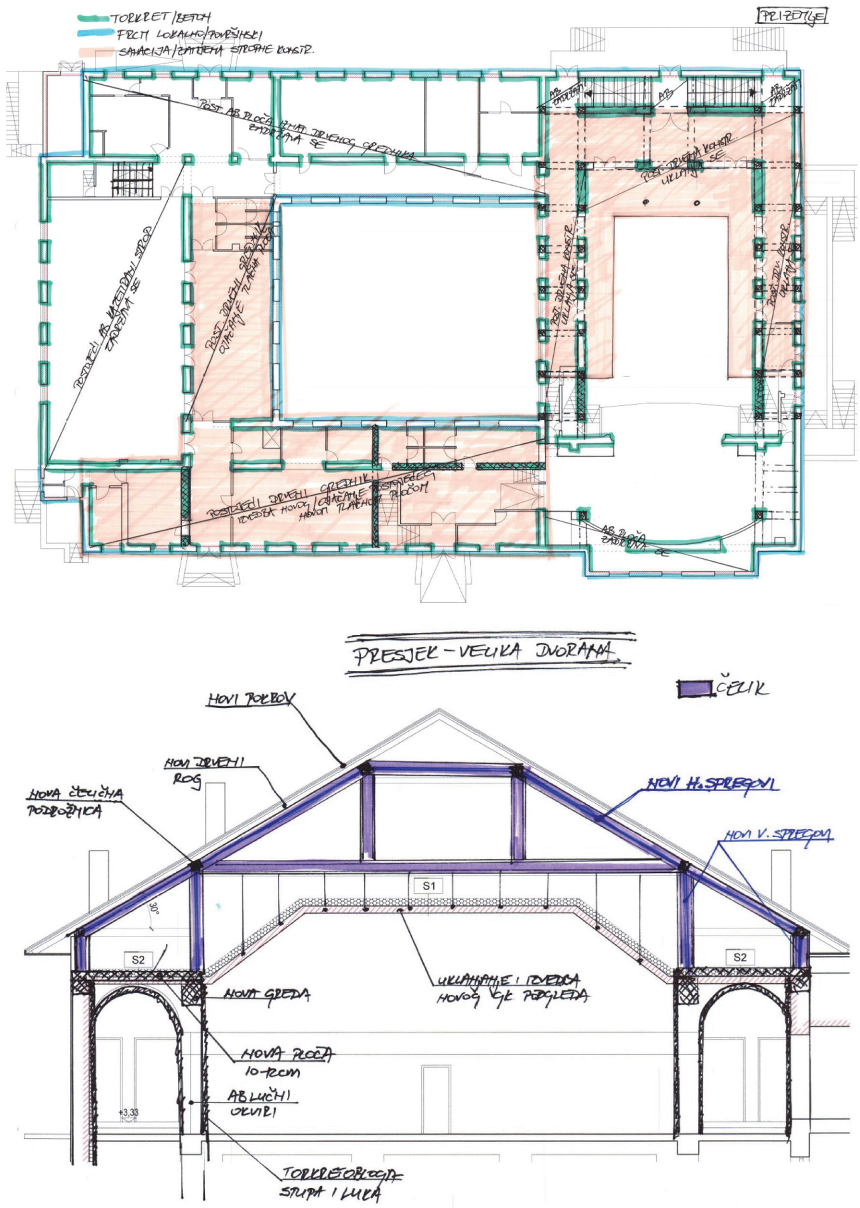


Figure 22. Sketches of concept solutions for structural reinforcement [3]

5.1. Methods of analysis and computational models

For the purposes of calculating the reinforcement of the existing building, a modal response spectrum analysis with a behavior factor was used. Since the calculation was carried out using the linear method, it is necessary to determine a behavior factor that takes into account the actual nonlinear behavior during seismic action on the structure. The value q = 2.0 was chosen for the behavior factor in consideration of the chosen ways to strengthen the current structure, which will increase its ductility and load-bearing capability. The modal analysis was performed on a bulk model incorporating the load-bearing structural elements of the building in order to determine the structural behavior as accurately as possible, as well as the sum of effective modal masses for the considered natural modes of vibration. Masonry walls reinforced with reinforced mortar are designed with an average thickness of d = 8 cm. The technical properties of the mortar are specified according to Table 3. Some walls are reinforced on one side, and some on both sides. In the computational model, such a composite wall is represented by determining the equivalent stiffness of a masonry wall strengthened with a reinforced mortar layer. In the model, this type of wall is modeled as a reinforced concrete element with an overall thickness, with a reduction in unit weight and stiffness applied. The input data provide a detailed description of the walls, including the method of reinforcement and their total thickness.

5. Structural reinforcement of the existing building as part of the comprehensive building renovation project

The analysis of the resistance of the existing structure was carried out as part of the Existing Condition Report, in which it was concluded that the structure does not have sufficient load-bearing capacity for seismic actions corresponding to the required renovation level. The comprehensive building renovation project was developed in collaboration with the architectural firm ing4studio d.o.o., and the works were executed by the contractor Strabag d.o.o. in cooperation with construction supervision provided by Investinženjering d.o.o.

Table 3. Technical characteristics of mortar

Intended use	Specification of mortar for masonry strengthening in accordance with EN 998-1 and EN 998-2
Compressive strength	M30 in accordance with EN 998-2
Classification in accordance with EN 998-1	GP CS IV
Grading	3 mm
Density of composite material	1900 kg/m^3
Compressive strength	>30 MPa according EN 1015-11
Shear strength	0,15 MPa according EN 998-2

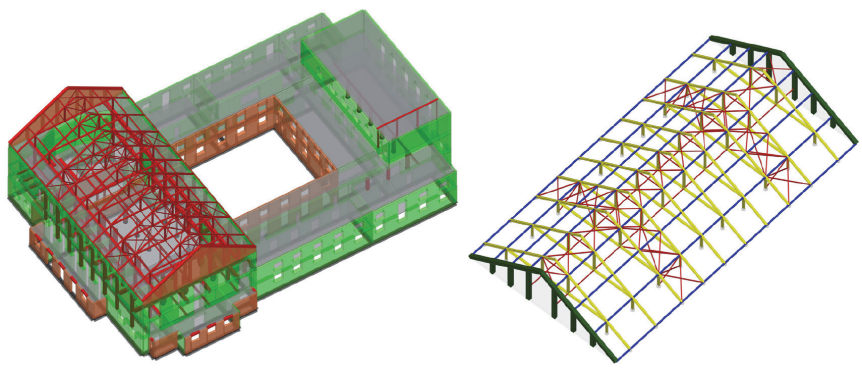


Figure 23. Computational 3D models [11]

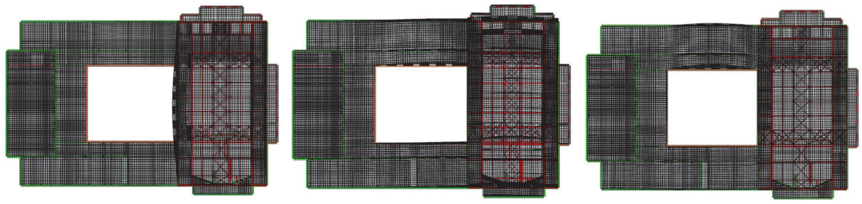


Figure 24. Natural periods of vibration of the reinforced structure of the entire building [11]

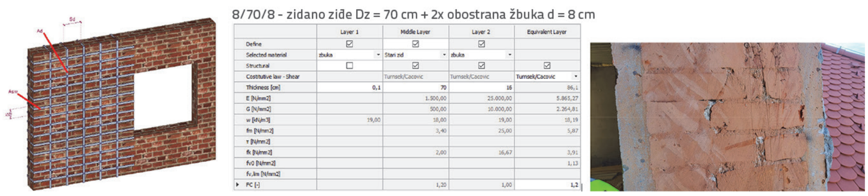


Figure 25. Calculation of parameters of reinforced masonry with reinforced mortar [11]

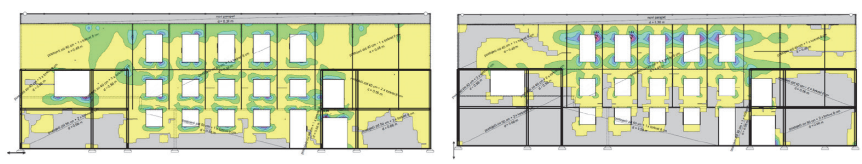


Figure 26. Results of the analysis of masonry reinforced with reinforced mortar [11]

The portions of masonry for which no reinforcement was planned were verified for in-plane resistance in accordance with the methodology described in detail in Section 4.4.

5.2. Solutions for reinforcing the existing building structure

Within the comprehensive building renovation project [11], prepared in accordance with all conservation guidelines, the concept of structural stiffening is described, based on strengthening the existing walls with additional layers of reinforced mortar applied on one or both sides of the wall.
In addition to reinforcing the existing walls, some partition walls will be removed and additional reinforced concrete walls will be constructed to increase the rigidity of the entire structure. For these measures to be effective, the construction of floor diaphragms is necessarily envisaged in areas where they do not currently exist. This applies in particular to:

- parts of the ceiling structure above the basement
- parts of the ceiling structure above the ground floor
- ceiling structure above the floor of the great hall
- roof structures (reinforcement of existing roof structures).

In order to lower the bulk and, consequently, the seismic load, the partition brick walls will be replaced by walls composed of lightweight materials. Depending on the state of the current structure, various methods will be used to manufacture ceiling diaphragms:

- by implementing a new reinforced concrete ceiling slab above the existing wooden structure as a fully load-bearing structure
- by implementing a flexible wooden diaphragm screwed to the existing and new wooden structure
- by building new slabs of reinforced concrete over pre-existing beams, as long as they are connected
- by constructing a composite steel–concrete structure.

All new diaphragms will be connected to the interior and perimeter walls with appropriate connections.
Every newly built wall must be supported by new foundations and have a consistent height. The reinforcement of the mortar layer must be continuous throughout the floor structure, and existing masonry enhanced with reinforced mortar must have sufficient anchorage to the foundation structure. A new reinforced concrete edge beam will be built and attached to the existing foundation using reinforced concrete dowels in order to implement the anchoring detail of all reinforced plaster walls. In addition to providing anchorage for the reinforcement of the mortar layer, the edge reinforced concrete beam will increase the overall width of the existing foundations, thereby reducing the bearing pressure on the foundation soil. The aforementioned beam will also distribute the additional mass load resulting from the reinforced plaster coating on the foundation structure onto the foundation soil.
Since the existing foundations are anticipated to be unreinforced, the edge beam's connection to the existing foundation increases the foundation strip's resistance to bending.

The part of the structure above the ground floor of the large hall (including the new floor diaphragm above the existing beams) will be constructed as a new structure, thereby forming a rectangular ring at the level above the ground floor and connecting it to the annex structures.

Above the floor level of the great hall, the construction of a new floor slab is also envisaged, designed as a reinforced concrete rectangular ring within the hall walls on the existing reinforced concrete beam structure. In order to link to the existing reinforced concrete section, the existing beams and arches will be covered with additional cladding that will have the required design reinforcement installed.

In order to prevent damage to the partition walls and drastically limit displacements, the ceiling structures will be attached to both the outer and interior walls. The loss of stability of the walls out of plane, which has recently been an issue in some places, will be prevented by keeping the walls at the levels of the ceiling diaphragms.

The proposed reinforcement and renovation concept will keep structural displacements at relatively small values and make them uniform.

The existing triangular gable walls (dovetail) of the large hall are envisaged to be removed down to the upper elevation of the reinforced concrete arches. The existing parapet wall is envisaged to be removed, after which a new reinforced concrete element anchored to the existing exterior walls will be constructed. Within this new reinforced concrete ring beam, anchor plates are envisaged to be installed for the construction of new steel frames.

The new roof structure is envisaged to be constructed as a steel structure with frames pinned to four supports. The supports on the exterior walls are planned to be executed as previously described, by installing anchor plates in the new reinforced concrete parapet wall, while the interior supports are planned to be anchored into new reinforced concrete encasements around the existing arches.

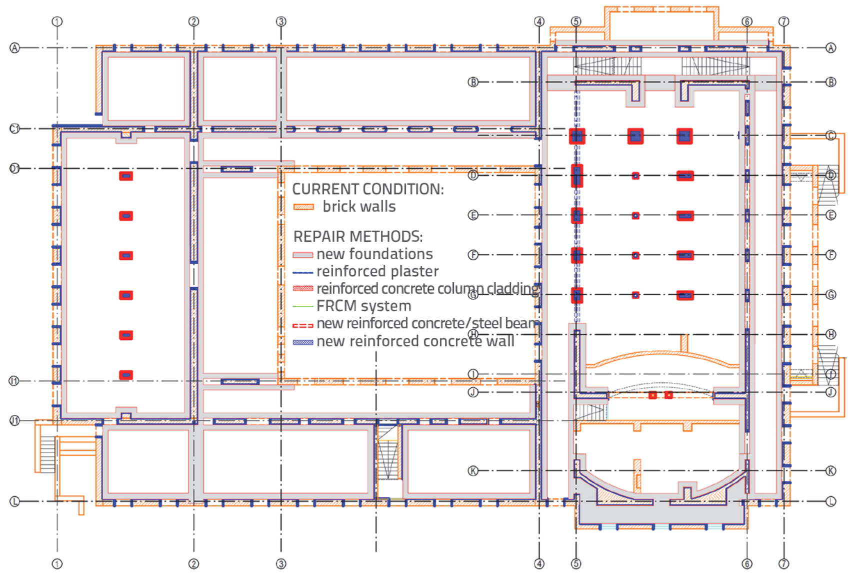


Figure 27. Reinforcement layout of the foundation structure [11]

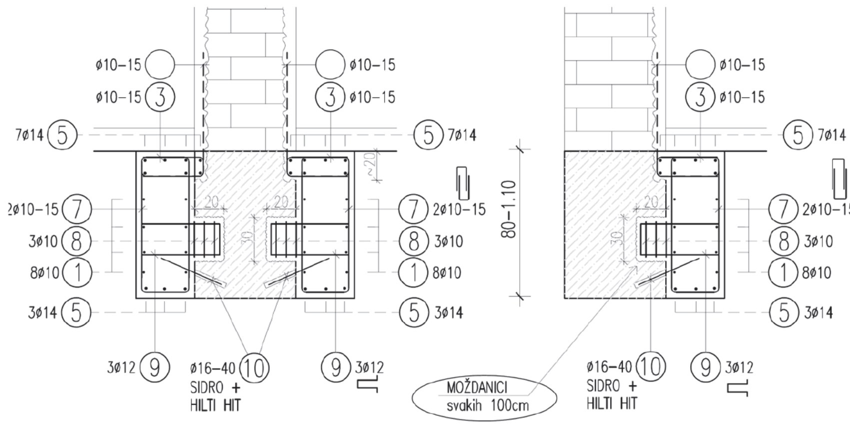


Figure 28. Characteristic details of foundation reinforcement [11]

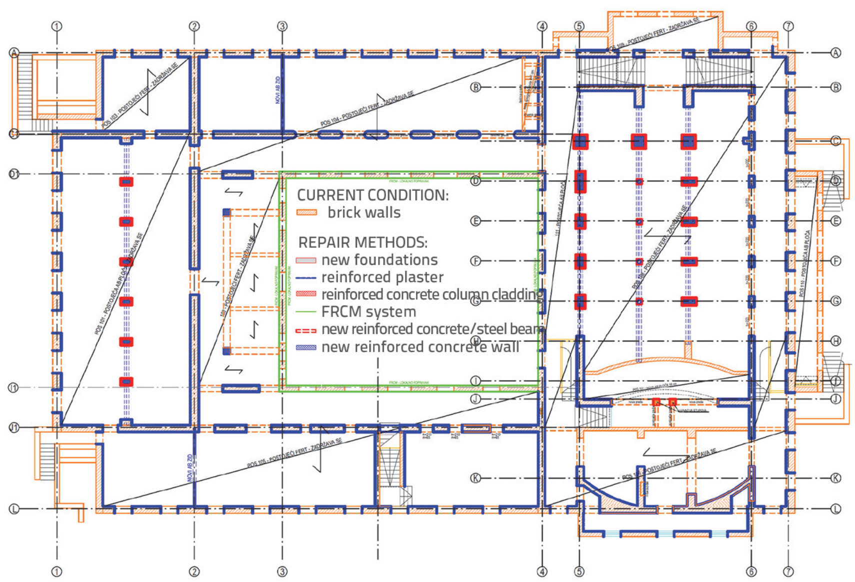


Figure 29. Layout of the basement structural strengthening [11]

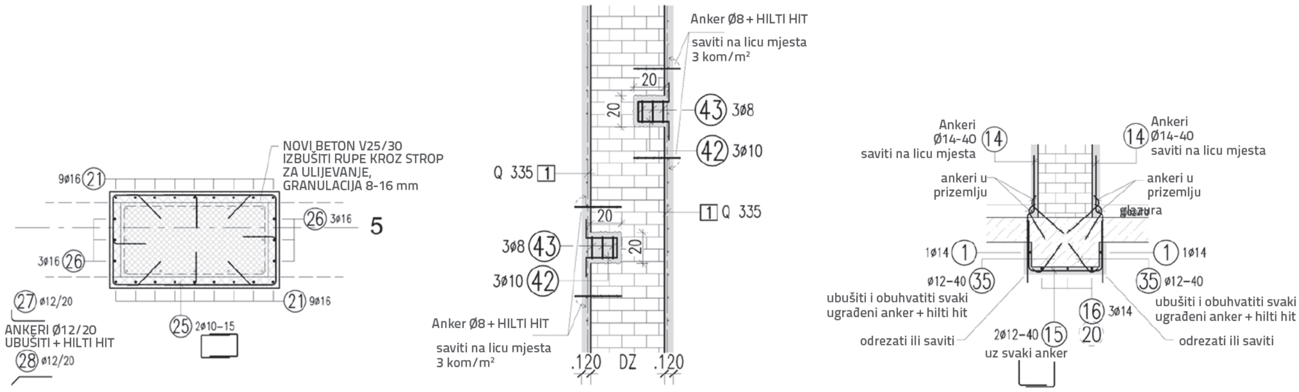


Figure 30. Characteristic details of the strengthening of the existing basement structure [12]

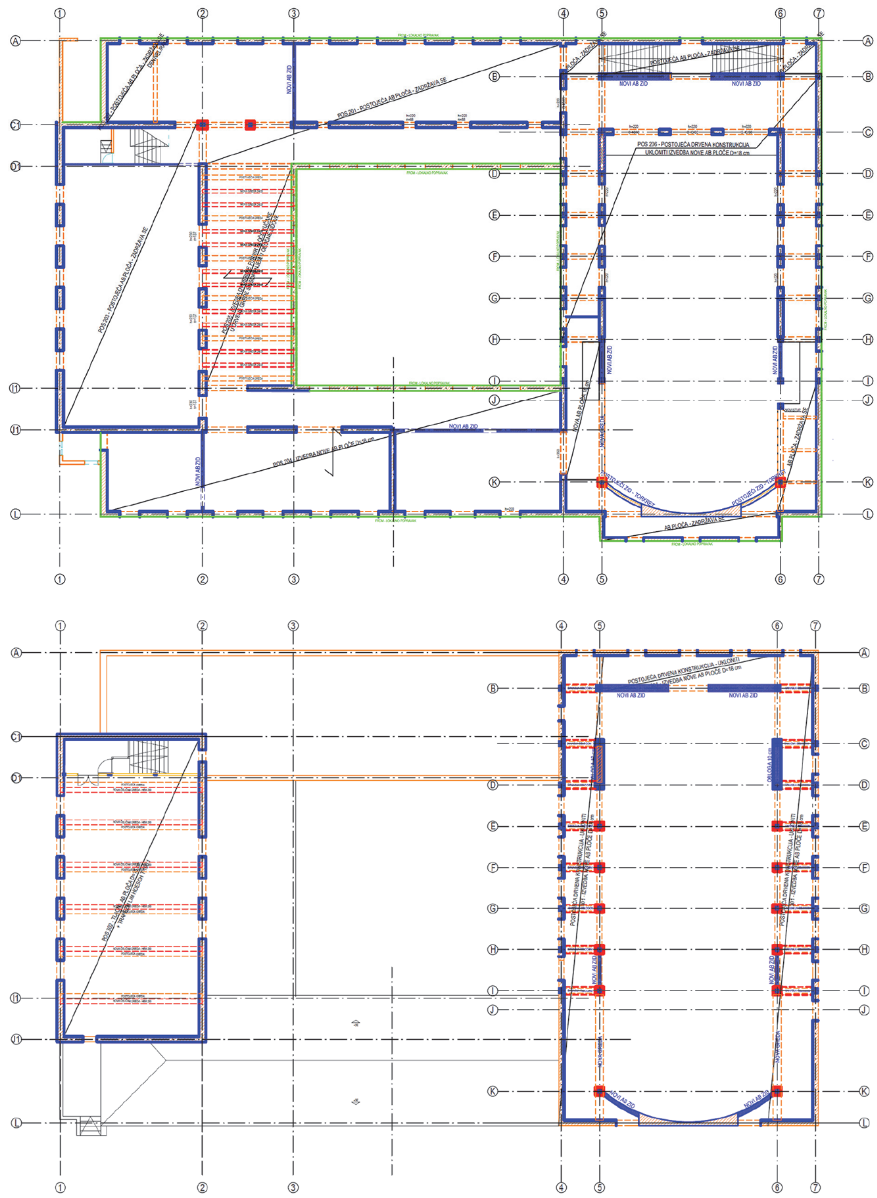


Figure 31. Layout of the strengthening of the ground floor and first-floor structure [11]

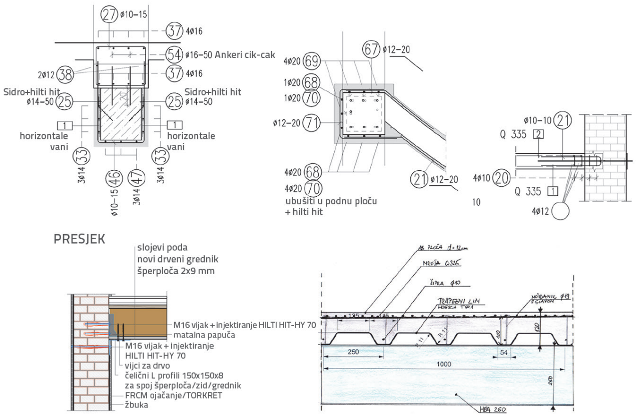


Figure 32. Characteristic details of the reinforcement of the existing ground floor structure [12]

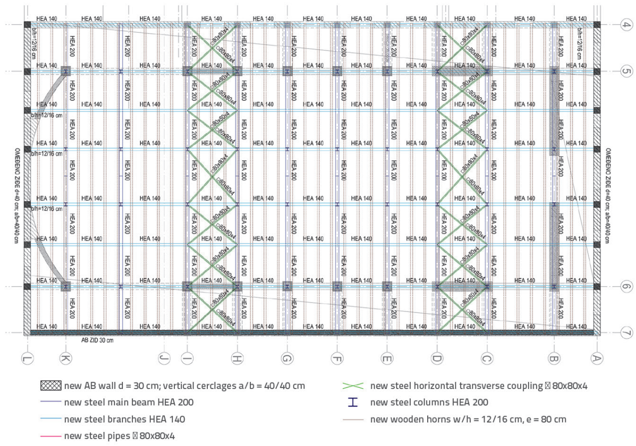


Figure 35. Construction of the upper belt of the new steel structure of the hall roof [11]

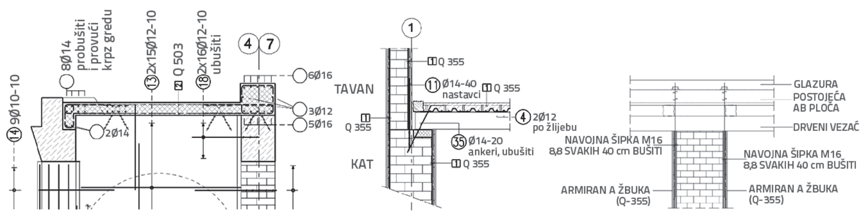


Figure 33. Characteristic details of strengthening the existing floor structure [12]

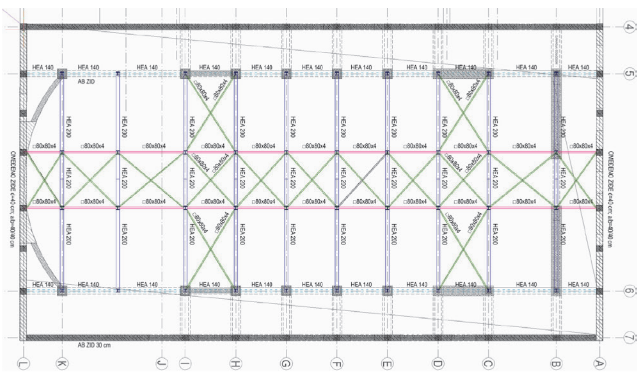


Figure 34. Construction of the lower belt of the new steel structure of the hall roof [11]

The stabilization of the frame in the transverse direction is achieved by tensioned frame connections. Structurally, a single longitudinal horizontal roof brace is provided to equalize movements along the frames. Two horizontal roof braces and two vertical stabilizing braces are proposed in the same spot in the transverse direction. Secondary steel members that will support timber rafters with plank sheathing are intended to be put on the main steel load-bearing structure.

Figure 37 shows some of the characteristic details used for the calculation of steel structure connections.

No renovation of the timber roof structure is planned in the annex zone and above the library. It is necessary to retain

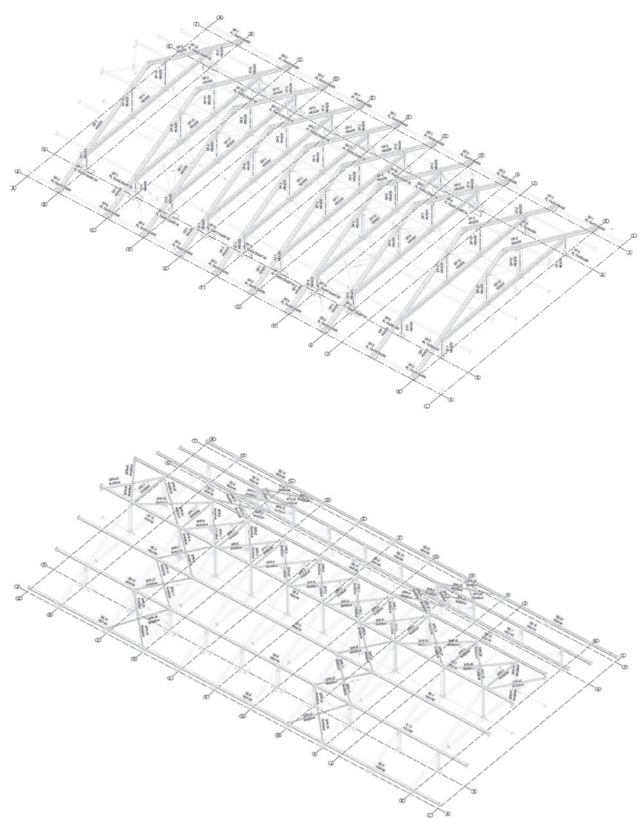


Figure 36. Shop drawings of the steel roof structure of the great hall [12]

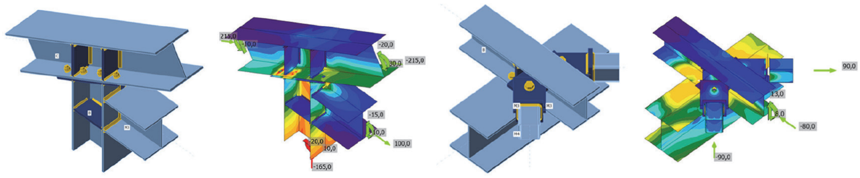


Figure 37. Design details of steel structure connections [12]

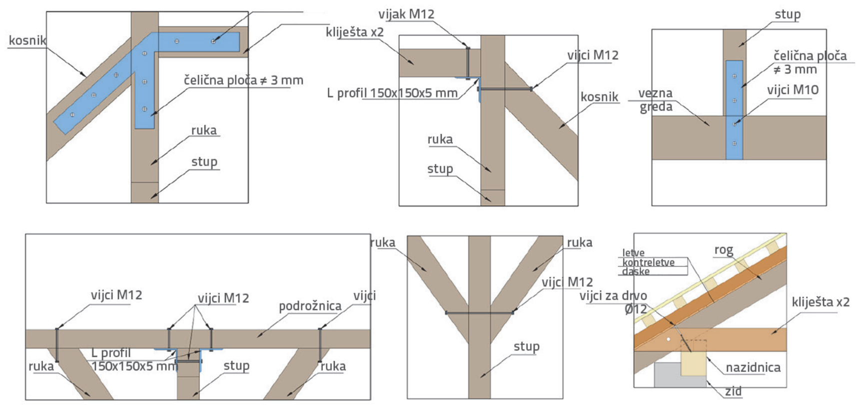


Figure 38. Details of reinforcement of existing wooden roof structure [12]

Figure 39. Images of the phase after removal of the cladding, when the structure is fully exposed [3]

the existing structure with additional measures to connect the nodes with the corresponding metal connections, as shown in Figure 38.

6. Design supervision and adaptation of design solutions during construction

Despite maximum effort in analyzing the existing condition, recordings of the existing condition, conducting investigative work, and detailed development of the main and detailed design after the start of the works, design supervision by the structural designer over the construction is necessary.

The initial construction period is the most intensive and requires the greatest commitment for the structural designer. The reason for this is the necessity of analyzing the existing condition of the load-bearing structure after removing all cladding. The structure's dimensions, layout, and any damage are all clearly obvious at this point.

The structural designer can then determine deviations from the existing condition for which he has prepared the reinforcement design. Even conceptual solutions should be corrected in cases of considerable variations (such as the current structure system), and budget evaluations should be repeated when implementation details are adjusted. Such a process requires intensive engagement of the structural designer in order to adapt the design solutions to the actual conditions on the construction site as quickly as possible.

If there are any local variations from the design documentation, which could happen for a number of reasons, such as:

- differences in the geometry of the existing structure
- differences in structural systems in individual local zones of the building
- inapplicability of technical solutions due to the Contractor's construction sequence
- modifications of technical solutions resulting from changes in other disciplines

It is necessary to make changes to the technical solution in such a way that the essential global strengthening concept (as defined by the structural analysis) is preserved. For such interventions, it is necessary to make local additional calculations and changes to the details of the connections with the construction sketches. Such local interventions occur during the execution of all structural strengthening works, and the role of the structural designer is precisely to adapt the details and solutions to the global concept. The speed of providing such solutions to the contractor is as important as the presentation of technical solutions that need to be applied as quickly as possible after the detection of problems (deviations).

Figure 40. Execution of works on the reinforcement of the foundation structure [3]

Figure 41. The current state of the existing structure of the large hall after the removal of the wooden roof [3]

7. Images of the construction works

Figure 40 shows the execution of works on the reinforcement of the foundation structure.

Figure 41 shows the condition of the existing structure after the removal of the wooden roof of the large hall and all surface layers of the structural elements.

Figure 42 shows the works on the reinforcement of the existing masonry.

Figure 43 shows the condition of the existing structure after the removal of the wooden roof of the large hall and all surface layers of the structural elements.

Figure 42. Execution of works on the reinforcement of the existing masonry [3]

Figure 43. Execution of works on the new steel roof structure of the hall [3]

Figure 44 shows the execution of works on the new floor structures and the reinforcement of the existing floor structures. Given that no renovation of the wooden roof structure is planned in the annex zone and above the library, Figure 45 shows additional measures for connecting joints using appropriate metal connectors.

Figure 44. Execution of works on new and reinforcement of existing ceiling structures [3]

Figure 45. Execution of works on the reinforcement of the existing wooden roof structure [3]

8. As-built condition

In September 2024, the newly renovated building of the "Hrvatski dom" Open University in Petrinja was officially opened. Today, the building of "Hrvatski dom" Open University accommodates the following units: "Krsto Hegedušić" Gallery, Counseling Center for Children, Adolescents and Families, Speech Therapy Office, Folklore Ensemble "Petrinjčica", Petrinja Wind Quartet as well as Literary and Recitation Section. The structure is significant to Petrinja's cultural life.

Figure 46. Renovated building and cultural content [13]

9. Conclusion

The renovation project of the building of "Hrvatski dom" Petrinja is an example of a successfully implemented comprehensive renovation encompassing technical, structural, and cultural aspects of the revitalization of the Petrinja following the earthquake.
The range of materials, structural systems, and methods that had to be used to guarantee the building's seismic resistance, stability, longevity, and safety reflects the project's complexity. The renovation was carried out respecting the principle of preserving the historical value of the building, while simultaneously improving its functionality and energy efficiency. Today, "Hrvatski dom" (Croatian Home) once again plays an important role in the cultural and social life of Petrinja. In addition to being a renovated structure, it also symbolizes tenacity, unity, and well-executed renovation—an illustration of how tradition and contemporary architectural knowledge may be blended to create a sustainable whole.
In this particular instance, it was both necessary and justified to remove the severely damaged portions of the structure (the large hall's roof) in order to replace them with reinforcements that would guarantee the earthquake resistance of the exact portion of the structure that had the initial flaws.
The possibility of leveling out small vertical deflections caused by the walls' loss of stability out of plane, which turned out to be the biggest initial weakness in the building's structure, makes the use of reinforcing existing masonry walls using the reinforced plaster method appropriate in this instance.

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