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Why did residential block in Croatian national revival street in Sisak damaged in earthquake had to be demolished

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Professional paper

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Residential block located in Croatian national revival street in Sisak is the largest building demolished after being damaged by Petrinja earthquakes in 2020. After the earthquake building was declared unsuitable for housing because of damage. All residents had to move out and since then the building is not in use. Classification of damage to building was determined according to The European Macroseismic Scale (EMS-98) and Technical Regulations for Structures (NN 7/22, article 24). In this paper is explained why is it neccessary to determine the level of actual earthquake resistance for buildings of this type, period of construction and initial seismic flaws. Earthquake resistance level of existing building can be determined only by seismic design calculations. Except for standard methods of calculations, such as linear-elastic calculation using response spectrum, it is necessary to calculate structures using nonlinear methods, such as pushover analysis. Reconstruction level and concept are defined by buildings damage level and determined earthquake resistance of the building. Course of procedures, such as evaluation of damage, study of state of existing structure and finally building design, which led to decision to demolish the building will be shown in this paper.

Key words:

Petrinja earthquake, damage assessment, structural reconstruction, unreinforced masonry, reinforced concrete structures

Stručni rad

Dragan Kovač, Juraj Herenda

Zašto smo morali srušiti stambeni blok oštećen u potresu u Ulici Hrvatskog narodnog preporoda u Sisku

Stambeni blok u Ulici Hrvatskog narodnog preporoda u Sisku najveća je u potresu oštećena zgrada koja je u cijelosti uklonjena nakon Petrinjskog potresa 2020. Nakon potresa zgrada je zbog utvrđenih oštećenja ocijenjena kao neuporabljiva te je iseljena i više se ne koristi. Stupanj oštećenja zgrade utvrđen je u skladu sa smjernicom EMS-98 odnosno u skladu s Tehničkim propisom za građevinske konstrukcije (NN 7/22, članak 24.). U radu pojašnjeno je zbog čega je za zgrade takve tipologije, iz tog perioda gradnje i s takvim brojem izvornih nedostataka u odnosu na seizmičku otpornost neophodno utvrditi stupanj zatečene potresne otpornosti. Stupanj potresne otpornosti postojeće zgrade može se odrediti jedino proračunom konstrukcije na potresno djelovanje. Osim uobičajenih metoda proračuna konstrukcije linearno elastičnim proračunom (spektralna analiza) poželjno je provesti nelinearni proračun metodom postupnoga guranja (pushover analiza). Stupanj oštećenja zgrade i njezina zatečena otpornost određuju razinu i koncept obnove. U radu prikazani su ocjenjivanje stanja oštećenja konstrukcije, izrada elaborata ocjene postojećeg stanja konstrukcije te analize koje su dovele do donošenja konačne odluke o uklanjanju zgrade.

Ključne riječi:

Petrinjski potres, pregled oštećenja, konstrukcijska obnova, nearmirano ziđe, armiranobetonska konstrukcija

1. Introduction

Residential - commercial block located in Croatian national revival street 2-10 in Sisak was built in the early 1960s. Block is divided in 5 structurally independent dilatations. All dilatations have regular floor plan shape, measuring 22.25 x 15.50 m in the floor plan and they are 7 storeys high (basement, ground floor and 5 storeys above ground floor level). 3D visualisation of building block model is shown on Figure 1.

Total gross area of residential block is 10.667 m^2. Load bearing structure of the building consists of unreinforced masonry walls made of solid clay brick. Some load-bearing walls are discontinued in ground level and they are supported by reinforced concrete girders and columns instead. Building block was heavily damaged during earthquake wich occured on 29. December 2020. After the initial assessment, which was made by professional engineers to determine usability of the building, it was declared that the building is unsuitable for housing because of damage it sustained. Residents were required to move out and building was not in use since then. Building block front is shown on Figure 2.

Damage level assessment process, and consequently process of determing required reconstruction level, was complex and strenous. Survey was carried out in order to determine deformations of existing structure. Its results, combined with multiple calculation analyses which were conducted, were later used to help determine the state of exsisting structure. Calculation methods that were used are linear-elastic spectral analysis and non-linear pushover method. By conducting structural survey, detailed damage assessment, in-site research and finally linear and non-linear analysis, it was determined that existing structure has significantly lower seismic resistance than it looks regarding visible damage it endured. In-site research discovered far more initial seismic deficiencies than expected. It also revealed major differences beetween archive documentation wich was used to build the structure and actually built-in materials and elements. Since the building was designed and built in late 1950s, before first seismic standards were adopted in 1964., it is obvious that building was neither designed, nor capable of withstanding lateral seismic actions. Structures built in this era (beetween 1945. and 1964.) can be really dangerous during earthquakes. Why was it neccessary to demolish all buildings of this residential block and all steps of design which led to this decision will be shown in this paper.

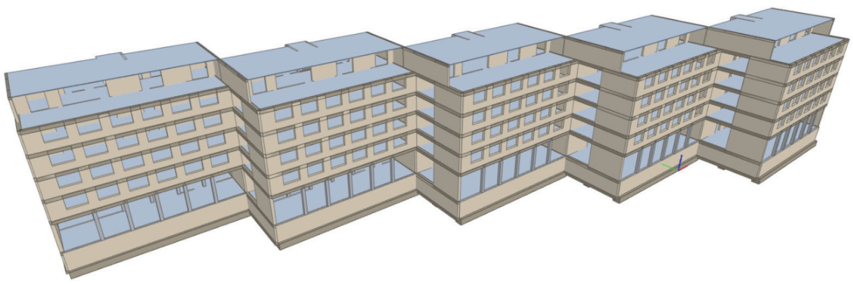


Figure 1. 3D visualisation of building block model

Figure 2. Building block front facade

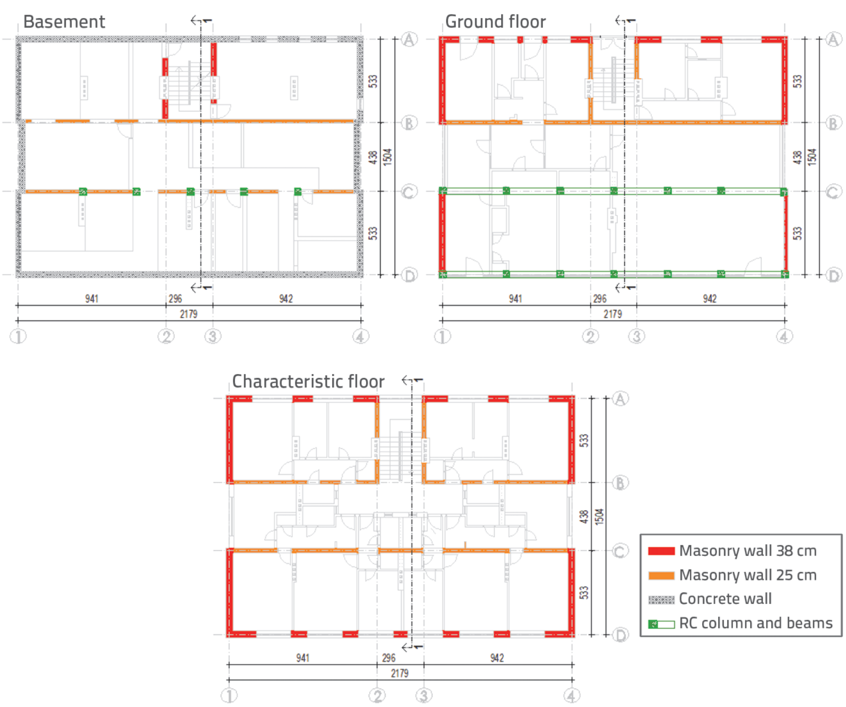


Figure 3. Vertical load bearing structure elements

2. Description of existing structure

Residential - commercial block is divided in 5 structurally independent dilatations. Load bearing structure of every dilatation consists of unreinforced masonry walls made of solid clay brick. Outer walls are

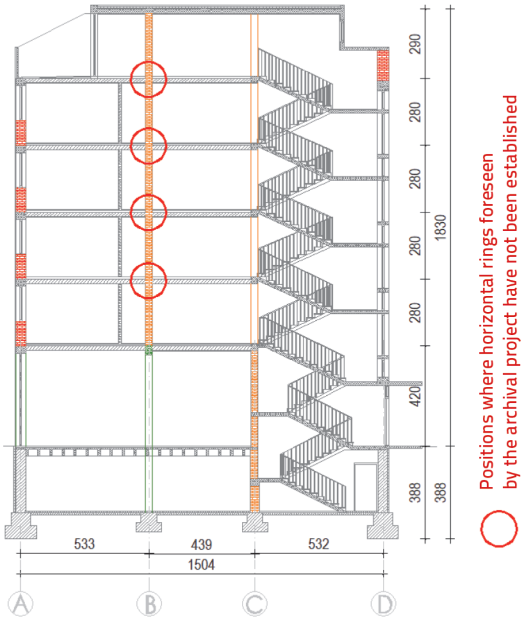


Figure 4. Cross section of exsisting building

Figure 5. In-situ testing of ground floor column

38 cm thick, interna walls are 25 cm thick, while non-bearing walls are 6.5 cm thick. Outer walls of basement level are made of concrete. Some load-bearing walls are discontinued in ground level and they are supported by reinforced concrete girders and columns instead. Load bearing vertical elements are shown on Figure 3.

It is visible on the first glance that thickness of load bearing elements is inadequate for building which is 7 floors high. Ceiling structures are made of premade "monta" system which transfers loads in transversal direction of building. According to original project, system consist of reinforced concrete girders with masonry elements inbetween, covered by reinforced concrete slab above. Cross section of the building is shown on Figure 4, while in-situ testing is visible on Figure 5.

Available archive documentation of existing building consists of original static calculations, formwork plans and reinforcement drawings. Part of documentation is shown on Figure 6. Since available archive documentation was extensive, it was planned to conduct only minor number of in-situ tests. Goal of conducting those tests was to confirm that existing state of building matches with available archive documents. However, those control in-situ tests have shown significant differences compared to archive documentation. Bond beams above internal walls were not determined and instead of reinforced concrete slab, unreinforced cement screed was visible above "monta" floor. According to original project, strip footings should be constructed in two central longitudinal axis, but pad footings were spotted after in-situ testings. Since such important structure elements were not built according to archive documentation, desing team was suspiocious towards build quality. Original archive drawings are shown on Figure 6. Footing drawing is shown on Figure 6.a, while characteristic

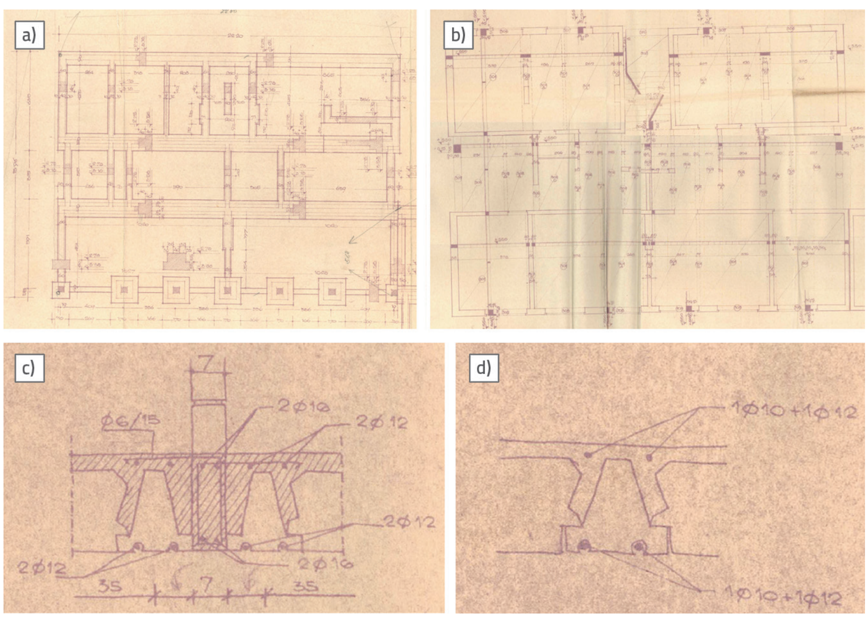


Figure 6. Archive drawings: a) Footing drawing; b) Characteristic floor formwork drawing; c) Reinforcement detail of beam beneath non-bearing wall; d) Reinforcement detail of premade floor beams

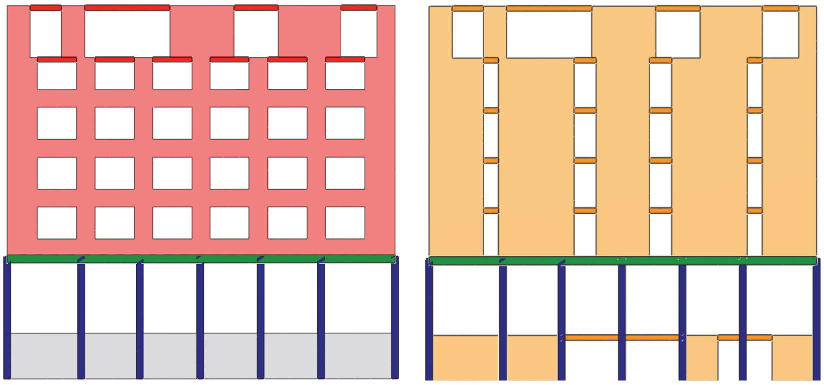


Figure 7. View on walls which are discontinued on ground floor level

floor formwork drawing is shown on Figure 6.b. Reinforcement detail of beam beneath non-bearing walls is shown on Figure 6.c and reinforcement detail of premade floor beams is visible on Figure 6.d.

Because of notable differences between in-situ tests and archive drawings, additional, significant number of in-situ tests had to be conducted. There were total of 70 in-situ tests, of which 60 were on floor slabs and load bearing structures and 10 excavations of foundations. Significant number of initial seismic deficiencies was determined on existing structure, such as:

- Lack of bond beams and tie columns
- More floors than allowed for unreinforced masonry buildings
- Inadequate area of load bearing walls compared to floor area (less than 3 % in longitudinal direction)
- Too thin walls compared to building height
- Too slender ground floor walls
- Reinforced concrete slab is not cast above "monta'' ceiling
- Continuity of load bearing walls does not exsits through all levels, beacuse some walls are supported by beams and columns on ground floor level
- Relatively heigh soft storey ground floor level (4.20 m)

It is not possible to gain a complete picture of all seismic flaws of the building by only examining archive documentation and inspecting the building visually, yet it is neccessary to perform static and seismic calculations of structure. This is how part of the initial seismic flaws of this building was discovered. By conducting modal analysis it was calculated that the first oscillation period of the structure is translation in transversal direction, while second oscillation period is torsional, which is in correlation to damages structure sustained. Other consequences of non existing continuity of walls on ground floor level are shear force values in beams and longitudinal forces in columns greater than allowed for frequent combination of actions. Also, calculated soil reaction is 362 MPa, which is significantly more than soil bearing capacity of 294 MPa calculated in additional geotechnical research.

3. Damage assessment level of structure according to EMS-98

Epicenters of earthquakes which struck Petrinja on 28 and 29 of December 2020. were located southwest of city center. Because of relatively big distance between earthquake epicentre and city center, there were mostly no heavily damaged structures in Sisak. Basic visual inspection of structure was performed immediately after the earthquake. All dilatations were classified as unusable because

Figure 8. Cracks in load bearing walls: a) Longitudinal wall damage POS 1; b) Transversal wall damage POS 2; c) Damage in corner of the wall POS 3; d) Transversal wall damage POS 4; e) Longitudinal wall damage POS 5; f) Transversal wall damage POS 6

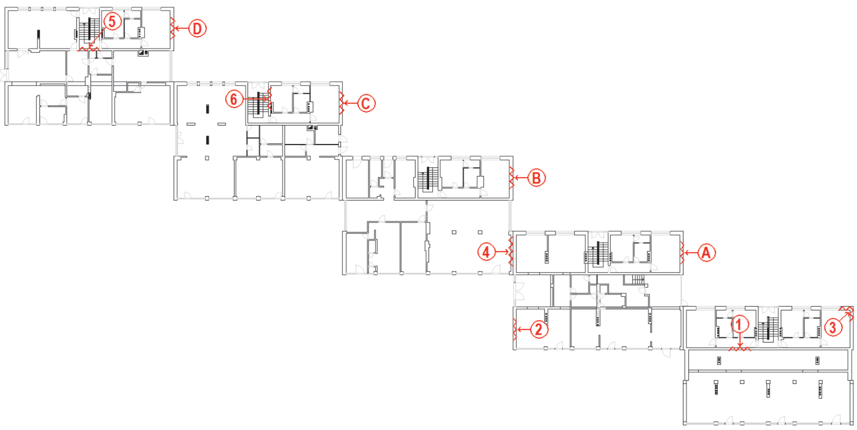


Figure 9. Ground floor plan – positions (POS) of damage shown (POS A to POS D)

of sustained damage and all residents were promptly evacuated. Damage of structural elements was classified as substantial, while damage of non-structural elements was classified as very heavy.

Part of substantialy damaged bearing walls is shown on Figure 8, while on Figure 9 ground floor plan with marked positions of those damaged walls is shown. Typical masonry wall failures, such as diagonal tensional failure and sliding shear failure are visible on those photographs.

Substantial damage was spotted on transversal load bearing outer walls. Such large dislocations of bearing walls are sign of excessive structure deformation. Deformations measured on places where walls have cracked are over 10 cm (between upper and lower part of wall section), which is visible on Figure 10. On Figure 9 location of those damages is shown.

It can be concluded taht those substantially damaged walls have no bearing capacity left over. Because of substantial deformations spotted and potential separation of floors and walls in their joints, survey was made immediately after the earthquake. Vertical load bearing structural elements and their deformations were measured, and results are shown in Professional opinion on state of load bearing structure [9]. Horizontal displacements of up to 24 cm are shown in survey, which is visible on Figure 11. According to mentioned Professional opinion, small part of measured displacements are imperfections during the construction phase, which are not recorded separately, but fact that there are measured displacements of up to 24 cm undoubtedly proves that building lost verticality and that rotations and separations of walls occurred. Sole survey used to measure vertical displacements is not enough to prove that structure suffered excessive deformations as a result of earthquake. Deformations measured could be results of imperfections during the construction phase. It is neccessary to observe survey in correlation with damage building suffered and occurrance of separation of floors and walls (loss of stability out of plane). According to EMS European Macroseismic Scale 1998 [4] classification of damage to masonry buidling is split in 5 categories, as shown in table 1 below. According to Technical Regulations for Structures [5], reconstruction Level is defined by classified level of damage and purpose of building. For example, private houses are reconstructed on Level 3, regardless of classified level of damage.

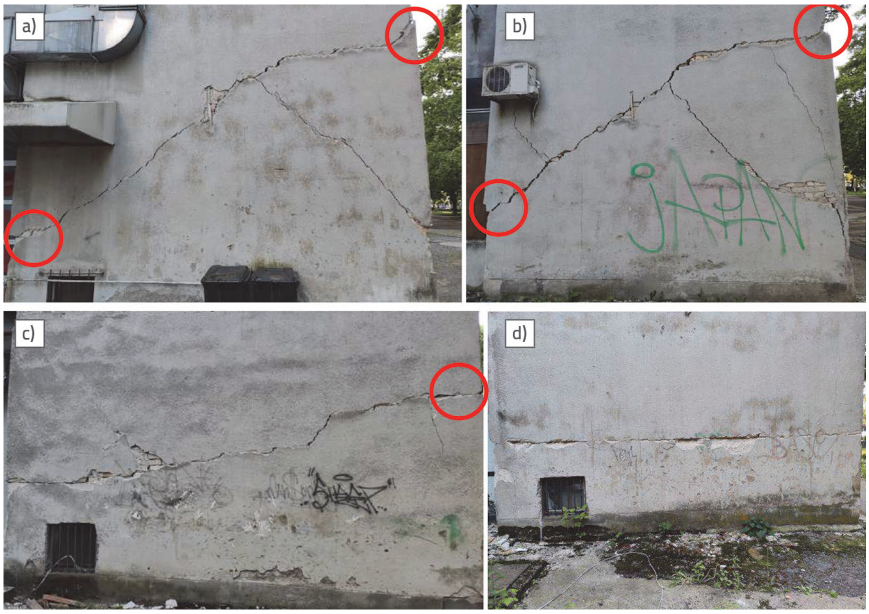


Figure 10. Cracks and dislocations of outer walls: a) Transversal wall damage POS A; b) Transversal wall damage POS B; c) Transversal wall damage POS C; d) Transversal wall damage POS D

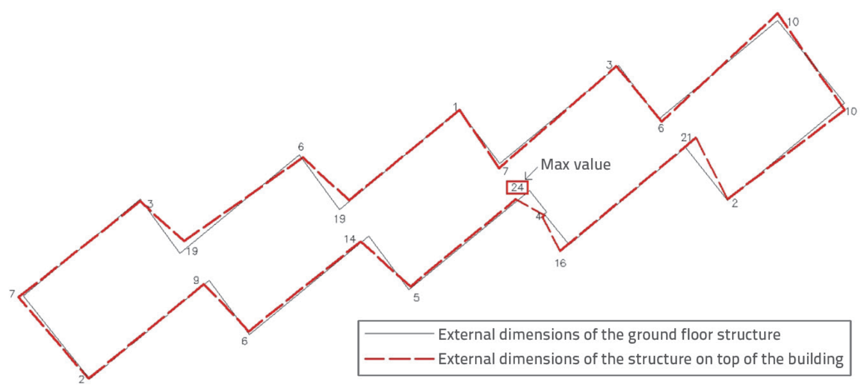


Figure 11. Vertical displacements of building contours

Table 1. Classification of damage according to EMS-98 and scheme of reconstruction levels

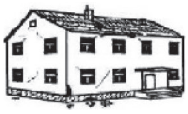
Classification of damage to masonry buildings		Reconstruction level according to damage level
	Level 1: Negligible to slight damage ▪ no structural damage ▪ slight non-structural damage	LEVEL 1 all buildings
	Level 2: Moderate damage ▪ slight structural damage ▪ moderate non-structural damage	LEVEL 2 IZO at least 0.5
	Level 3: Substantial to heavy damage ▪ moderate structural damage ▪ heavy non-structural damage	DECISION MADE BY ENGINEER LEVEL 3 IZO at least 0.75
	Level 4: Very heavy damage ▪ heavy structural damage ▪ very heavy non-structural damage	DECISION MADE BY ENGINEER
	Level 5: Destruction ▪ very heavy structural damage	IZO -Significant Damage Index LEVEL 4 IZO at least 1.00

Table 2. Guidelines for the classification of damage to masonry buildings [6]

 **Substantial to heavy damage** is a damage that significantly alters the capacity of the structure, but not close to the limit of partial collapse of the main structure. The fall of non-structural elements is possible.	**Level 3: Substantial to heavy damage** • **moderate** structural damage, • **heavy** non-structural damage, • **wide and numerous** cracks in **most** walls, • Roof tile falling off, • Chimney breaks in the roof plane • Collapse of individual non-structural elements (partition walls, gables).	This level of damage includes damage greater than level 2 and may also include level 1b damage, but not necessarily. Non-structural damage cannot be relevant for assessing the degree of damage if there is no structural damage of level 3. • These damages are accompanied by major non-structural damages. • Foundations (permanent deformations in the ground that caused significant damage to the walls and which require strengthening of the foundations). • Significant gap between floors and/or stairs and walls and between vertical walls up to approximately 1 cm with visible dislocations. • Vertical cracks at the corners of the walls up to 5 mm. • Deflection of the load-bearing wall out of plane in such a way that the failure mechanism is visible and that significant cracks have opened at the joints of the wall with ceilings and vertical walls over a longer area of the floor plan (often only the parapet in the attic is activated, which does not lead to damage level 3). Note: Geodetic survey is not evidence of residual displacement, but it is necessary to determine the displacement from the cracks. • Possible minor partial collapses in the side beams of the floors. • Cracks of a few mm in the vaults **Unreinforced masonry with wooden beams** • In more than 60 % of load-bearing walls, large and long cracks (less than 0.5 cm on the masonry - not the plaster). • From 30 % to 60 % of load-bearing walls have large cracks (up to 1 cm) where the wall is about to collapse. In the same walls, there may be a deflection (dislocation) of the wall out of plane (up to a maximum of 1.0 cm). • Up to 20 % of load-bearing walls (minimum 2 load-bearing walls - not the lintels) have very large cracks (greater than 1 cm). In the same walls, there may be a deflection (dislocation) of the wall out of plane (up to a maximum of 1 cm). • Locally possible wide cracks on one load-bearing wall with a dislocation out of plane, but not endangering the global stability of the building (gable wall or wall in a plane in which a significant redistribution of internal forces is possible). **Note:** The percentage includes all load-bearing components (walls, lintels, vaults, staircases). It is necessary to calculate (estimate) the percentage for each floor and then cumulatively for the entire building. The final percentage refers to the entire building. **Unreinforced masonry with rigid ceilings** Particular attention should be given to the critical floor (not the attic or the highest floor, usually the critical ground floor or the first floor above ground) and the distribution of cracks in each direction of the structure. • In more than 60 % of load-bearing walls, there are large and long cracks (about 0.5 cm on the masonry - not the plaster). • From 30 % to 60 % of load-bearing walls have very large cracks where the wall is about to collapse (about 1 cm). In the same walls, there may be a deflection (dislocation) of the wall out of plane (up to a maximum of 1.0 cm). • Up to 30 % of load-bearing walls (minimum 2 load-bearing walls - not lintels) have very large cracks (greater than 1 cm). In the same walls, there may be a deflection (dislocation) of the wall out of plane (up to a maximum of 1 cm). • Locally possible wide cracks on one load-bearing wall with a dislocation out of plane, but not endangering the global stability of the building (gable wall or wall in a plane in which a significant redistribution of internal forces is possible) **Note:** For the criteria that the building has rigid diaphragms, it is required that a rigid diaphragm be installed across the entire floor plan (not in the case of partial reconstruction). If the building has different ceilings on different floors, both criteria should be combined. For buildings with rigid ceilings, the stricter criteria should be selected.

It is somewhat harder to select appropriate reconstruction Level for residential buildings, such as this one is. Residential buildings classified as damage Level 2 can only be reconstructed to the Level 2. Residential buildings classified as level of damage 3 can be reconstructed according to Level 2 or 3, while residential buildings classified as level of damage 4 can be reconstructed according to Level 3 or 4. Visual representation of explained process is shown in table 1. Decision on which reconstruction Level should building be reconstructed is made by structural engineer.

In practice, classification of damage and reconstruction Level, was made in Assessment of existing state of structure, which preceded creation of Study of existing state of structure and complete project documentation. Classification of damage and required reconstruction Level were defined solely based on visible damage of the building and to some extent subjectively based on exprience of engineer. In accordance with the above, according to the visible damage, the degree of damage was determined as level 3 – substantial to heavy damage.

On buildings that are substantialy to heavily damaged, a detailed inspection of possible separation of ceilings from walls or the loss of verticality of walls should always be conducted Such damage (significant deformations) indicate the problem of loss of stability of walls outside their plane. In accordance with the guidelines on the application of the EMS-98 table according to Uroš et al. [6], in the case of determining an unacceptable degree of deformation, it is possible to determine a higher degree of damage. The deformation determined by survey cannot be the only reason for classifying a higher degree of damage, but in combination with other determined damage and calculation results, it is certainly a good guideline for classifying the degree of damage of the structure as level 4 – very heavy damage. Table 2 provides an example of guidelines for damage level 3.

The degree of damage and the required level of reconstruction are determined through the Study of the existing state of the structure, because it determines the level of existing resistance in relation to the full resistance according to applicable regulations. In order to determine the actual level of resistance to seismic actions, it is necessary to carry out a sesimic analysis of the existing structure. To correctly determine the reconstruction Level it is neccessary to calculate the degree of the existing resistance of the structure. The resistance of the existing structure actually determines the concept of structural reinforcement. It is to be expected that if the existing resistance level is relatively close to that required to achieve Level 2, this can be achieved by reinforcing the existing structural elements. However, if the existing resistance is very low, as in the example of this building, it is not possible to reinforce the existing structural elements to the level required to achieve the structural resistance required for Level 3. For such a multiple increase in the resistance level of the existing building, it is necessary to integrate a new structure in the existing structure to achieve such a level of seismic action.

4. Calculation of the seismic resistance of an existing building

One of the biggest shortcomings of this block of buildings is the extremely small area of the walls in relation to the floor plan area of the building. The area of the load-bearing walls of the ground floor for each direction is about 10.5 m2, or about 3 % of the gross floor area. From this data, it is already possible to get an impression of the expected level of resistance, which will be presented in the rest of the paper.

The calculation of the structure of the existing state was carried out as a linear-elastic calculation using spectral analysis. Considering that all 5 structural dilatations are almost equal, the calculation of the existing state was carried out for only one. The ground acceleration for a 475-year return period is 0.15 g. The masonry is built of solid brick in lime-cement mortar, so the appropriate characteristics for this type of masonry were used (modulus of elasticity, shear modulus, compressive strength and other parameters). The mechanical characteristics were determined based on the in-situ tests work and the recommended values according to the Italian guideline NTC 2018 [8] for solid brick masonry in lime mortar, and are given below.

- Masonry :Modulus of elasticity 1500 MPa
- Shear modulus 500 MPa
- Characteristic compressive strength 2.40 MPa
- Characteristic shear strength 0.09 MPa
- Concrete strength class MB 30 (corresponds to modern strength class C25/30)
- Reinforcement: smooth GA 240/360

Due to cracking, the bending stiffness of all elements is reduced by 50 %. The importance factor was taken in the amount of 1.0 because it is a building of ordinary importance. The behavior factor is 1.50 in accordance with HRN EN 1998-1 [7] for an unreinforced masonry system. Since unreinforced masonry is not ductile and there is no possibility of seismic energy dissipation, a higher behavior factor should not be used in the calculation. The soil category taken into account is C. The value of the soil reaction modulus according to the geotechnical study is 6300 kN/m3, and the modal analysis was performed on a model where the supports are assumed to be infinitely rigid. Figure 12 shows the construction elements of the calculation model.

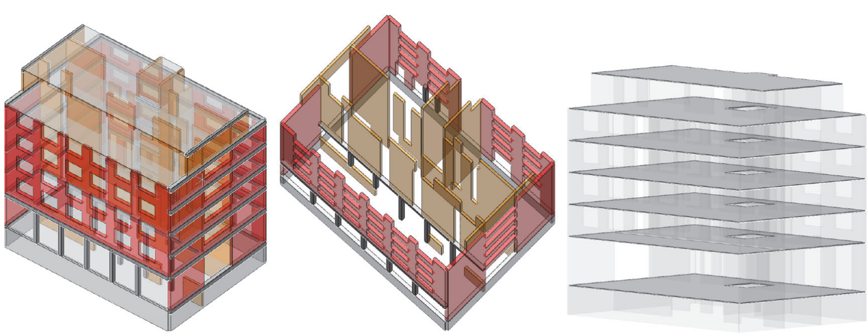


Figure 12. Calculation model of the structure

The modal analysis conducted determined that the first three periods of vibration of the structure are T1 = 1.79 s, T2 = 1.47 s and T3 = 1.28 s, respectively. The first form of vibration is translational in the transverse direction, the second is torsional, and the third is translational in the longitudinal direction. The identified damage to the structure is in correlation with the results obtained from the calculations. The severe damage to the facade load-bearing walls determined corresponds to the first, translational form of vibration in the transverse direction of the building, i.e. in the direction of these walls, while the separations of the building edges determined by survey correspond to the second, torsional form of vibration. Due to the large vibration periods, which are on the descending branch of the response spectrum, the seismic forces acting on the structural elements are significantly reduced, as shown in Figure 13.

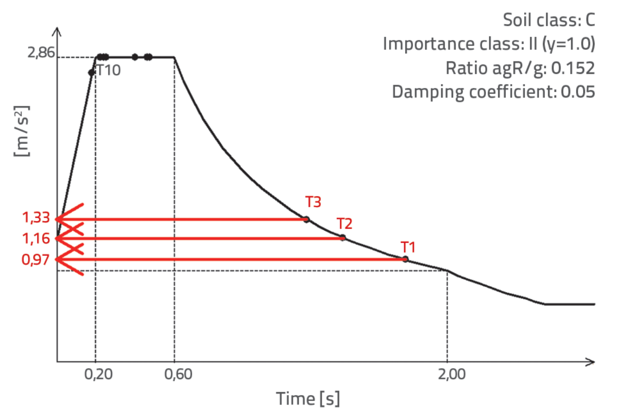


Figure 13. Design spectrum

Through linear-elastic calculation for ground acceleration with a 475-year return period, i.e., for 0.15g, the internal forces in the walls were obtained, and masonry design was performed. The total weight of one expansion joint is 34,000 kN. Using linear-elastic calculation with the response spectrum method, the transverse force in the ground floor was obtained as 4,400 kN in the longitudinal direction and 3,100 kN in the transverse direction. Therefore, the transverse force in the longitudinal direction of the building is 13 % of the mass, and in the transverse direction 9 %, which is significantly higher than the forces for which buildings built in that period were calculated and designed. Since the elements are brittle, or do not have sufficient ductility, and also insufficient load-bearing capacity, they fail at relatively small values of internal forces in the elements. In other words, the limit state of significant damage is reached for a significantly lower ground acceleration. For this reason, it is not possible to verify the limit state of significant damage, or determine the IZO factor, using a linear elastic calculation for the full value of ground acceleration. As a rule, for such structures, the determination of the existing resistance should be demonstrated by performing several iterative linear calculations, reducing the ground acceleration until each structural element satisfies it. In accordance with point 9.4 (6) of the HRN EN 1998-1 standard, it is also possible to take into account the possibility of redistributing forces to other masonry structural elements up to 25 %. The ratio of the achieved design seismic resistance of the structure (action expressed as peak ground acceleration type A) to the structural requirements for the limit state of significant damage (peak ground acceleration type A for a return period of 475 years) represents the significant damage index (IZO). By performing the iterative procedure using a linear elastic calculation (spectral analysis), the value of IZO = 0.15 was determined for the building in question.

Linear elastic analysis is conservative because the action at which the entire structure fails is taken as the action at which the first primary seismic element fails. In the linear analysis, the redistribution of force to other structural elements, nor the nonlinear contribution of individual elements in the plastic range, i.e. their ductility, are not taken into account. In the nonlinear analysis, the redistribution of forces and the nonlinear contribution are taken into account.

For the analysis of this building, a nonlinear analysis will be performed using the Pushover method. Material nonlinearity is taken into account at the element level via the corresponding load-bearing capacity curves. A bilinear material behavior curve (strain-strain) is adopted. The mechanical characteristics of the materials used in the model for nonlinear calculation are not the same, as nonlinear calculations use mean values, while linear calculations use characteristic values. A representation of the structural model is given in Figure 14.and the values used are as follows:

- Masonry: Modulus of elasticity 1500 MPa
- Shear modulus 500 MPa
- Characteristic compressive strength 3.40 MPa
- Characteristic shear strength 0.16 MPa
- Concrete strength class MB 30 (corresponds to modern strength class C25/30)
- Reinforcement: smooth GA 240/360

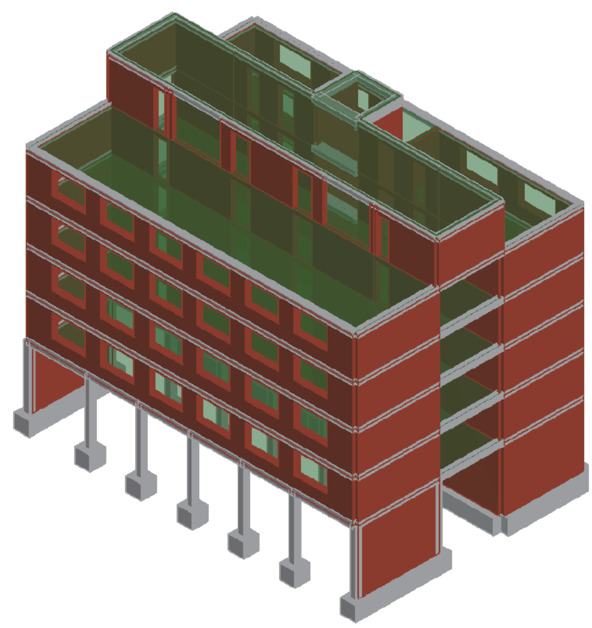


Figure 14. Calculation model of the structure

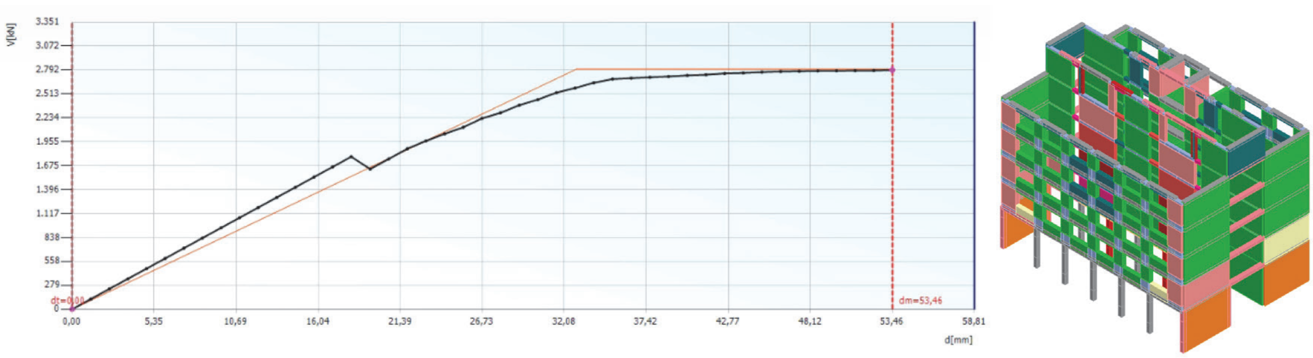


Figure 15. Relevant design curve showing damage in the last step of the analysis (Base shear V = 2 792 kN, Displacement d_m = 53,46 mm)

It is necessary to control the limit state of significant damage (SD) and the limit state of limited damage (LD). The ground acceleration for the limit state of significant damage (SD) corresponds to the value of the ground acceleration for the seismic action of the return period of 475 years, and for the location in question is ag = 0.152 g. The ground acceleration for the limit state of limited damage (LD) corresponds to the return period of 95 years, and is ag = 0.072 g.

Through nonlinear analysis using the pushover method, the existing resistance of the structure was determined to be 35 %, and the significant damage index (IZO) is 0.35. The significant damage index of the structure is the ratio of the calculated seismic resistance and the resistance for the limit state of significant damage, i.e., seismic action with a return period of 475 years.

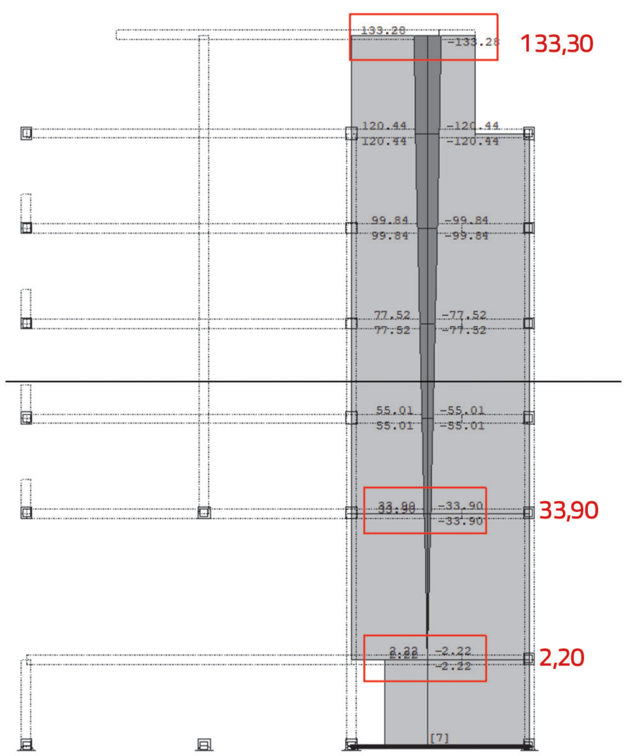


Figure 16. Inter-storey drift

It is important to note that the obtained resistance of the model in question refers to an undamaged idealized structure. The actual resistance of the structure damaged by the earthquake is significantly lower.

The calculation showed lower load-bearing capacity of the building in the transverse direction, which is consistent with the observed damage to the structure, with more significant damage and deformations visible in the walls in the transverse direction.

The nonlinear analysis provided a higher resistance compared to the linear-elastic calculation. The significant damage index based on the linear calculation is 0.15.

According to the Technical regulation for structural design, for reconstructions at Levels 3 and 4, a check of the limit state of limited damage is required for seismic action corresponding to a return period of 95 years, i.e., it is necessary to check the usability limit state criteria. The stiffness of the elements in the calculation model used to check the serviceability limit state is not reduced, but is given in full, and the behavior factor for the calculation is 1.0.

The obtained total elastic displacement of the top of the building is 13.3 cm, while the allowed total displacement is 9.2 cm. The largest inter-storey displacement was obtained for the ground floor, and is 3.1 cm, and the largest allowed inter-storey displacement is 2.2 cm. Figure 16 shows the relevant elastic displacements of the structure for seismic action on the selected frame.

5. Removal of the building

As stated in the previous chapter, during the preparation of the Study of the existing condition of the building structure, the possibility of retaining the existing structure and strengthening it to Level 3 was considered. With such a level of reconstruction, it is necessary to achieve a significant structural damage index (IZO) of at least 0.75, i.e. it is necessary to achieve at least 75 % resistance compared to the full required resistance according to the HRN EN 1998-1 standard. The existing resistance of the existing structure is extremely low. In order to significantly increase the level of structural resistance, it is necessary to integrate new elements to accept seismic action. The most effective way to significantly increase the resistance of the structure to seismic forces is to

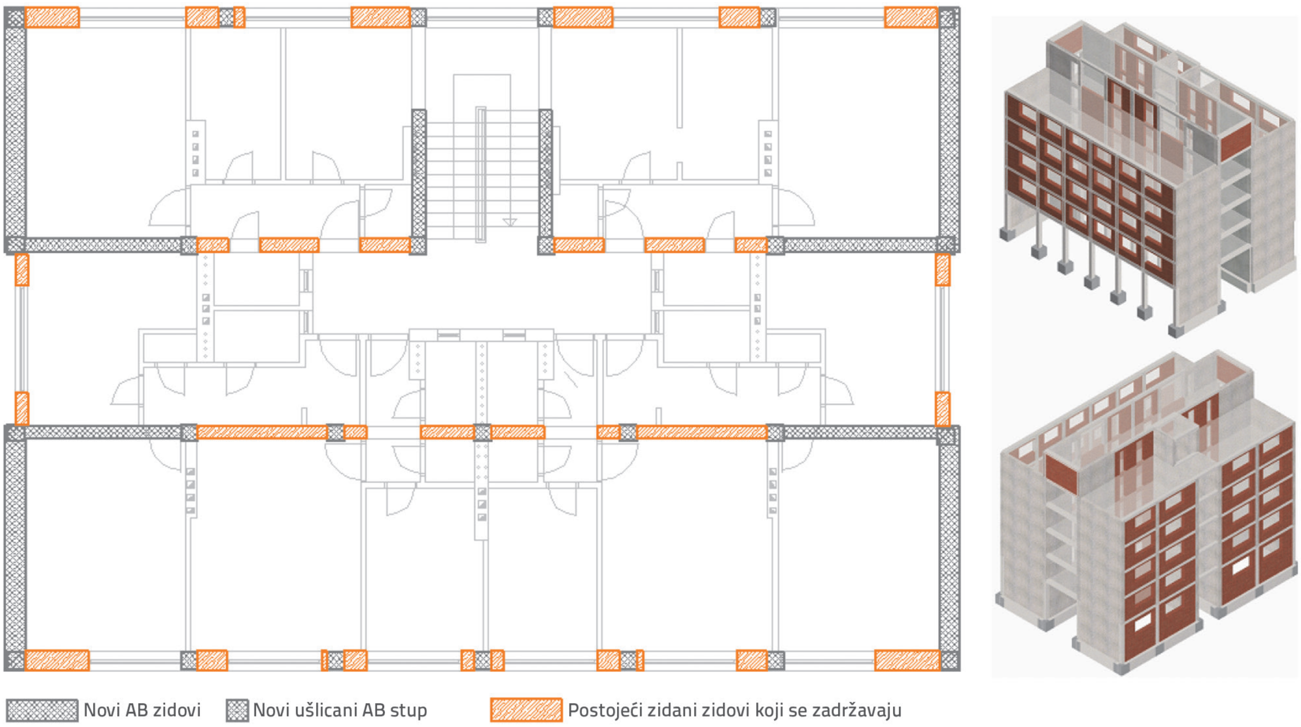


Figure 17. Proposal for a technical solution to strengthen the existing structure to Level 3

construct a rigid reinforced concrete structure in both directions of the building.

For this building, a solution for strengthening the structure was considered, involving the removal of part of the masonry walls and replacing them with reinforced concrete walls. In certain positions, columns and vertical tie beams would be inserted. The aforementioned solution is shown in Figure 17. With such interventions, which belong to Level 3 interventions according to TPGK, a linear elastic calculation was performed (using spectral analysis) and it was possible to achieve an IZO greater than 0.75.

However, the removal of masonry walls and their gradual replacement with reinforced concrete is an extremely complex intervention. As determined by in-situ tests, there are no horizontal tie beams above the interior walls, nor are there compression slabs on the prefabricated ceilings. Additionally, survey established that the building has suffered significant permanent deformations. The execution of the works would require complex spatial supports for the remaining structure, and such complex interventions would pose a high risk of losing the stability of the structural elements during construction.

According to article 7 of the Technical regulation for building structures, engineer is responsible for ensuring load-bearing capacity and stability during the construction phase, and not only for the final designed state. Ensuring load-bearing capacity and stability during construction for this building and the concept of the intervention, which would involve removing the existing load-bearing walls and replacing them with new reinforced concrete walls, would be extremely complex, time-consuming, and hazardous to the workers' lives. Given the condition of the existing building and the circumstances presented, the designer was unable to ensure and guarantee the load-bearing capacity and stability of the structure during the construction phase, or to comply with the requirement from the technical regulation. This was the final technical reason for the designer's decision that the existing structure must necessarily be removed.

In addition to the above, a comparative financial analysis of the reconstruction of the building by significant reconstruction compared to the construction of a replacement building was conducted. The analysis found that restoring the building through significant reconstruction would be more expensive and take longer than removing the existing structure and building a new reinforced concrete structure. The calculation of the construction price by significant reconstruction determined that the construction price would be more than 2,000 euros/m2, and the construction of a new building no more than 1,500 euros/m2. In addition to the above, the contracted construction price in public tenders for the renovation of similar buildings was checked. The new building would ultimately meet other basic requirements for the building to a much greater extent, while ensuring mechanical resistance and stability according to all applicable regulations, than would be the case with the renovation of the existing one. A mitigating circumstance was also the fact that the building is not a cultural heritage site nor is it located in a cultural and historical entity.

6. Load bearing structure of the replacement building

The new replacement building is being constructed as a relatively simple and typical reinforced concrete structure. The original design flaws have been corrected to the greatest extent

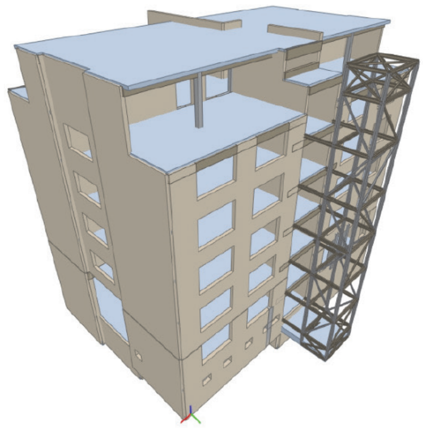


Figure 18. Presentation of the newly designed dilatation 6

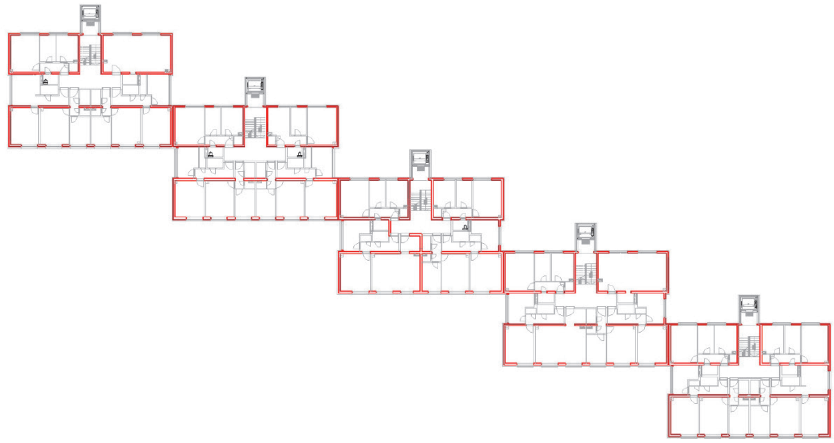


Figure 19. Floor plan of the newly designed structure

possible. Since the layout of the space in the newly designed building needs to be maintained as it is in its current state, the grid of the load-bearing structure and the external dimensions of the building have been retained.

The new structure is entirely made of reinforced concrete. The load-bearing walls are 20 and 25 cm thick, depending on the position, which further increases the usable area due to the smaller thickness of the walls compared to the original state. To prevent exceeding the maximum allowable shear force on the ground floor beams, their heights have been minimally increased at critical points. Additionally, due to exceeding the allowable compressive stress and fire protection requirements, the dimensions of the ground floor columns have been increased, and higher-strength concrete has been used compared to the rest of the structure. The ceiling slabs are 20 cm thick, load-bearing in two directions, so the load distribution on the vertical elements is uniform in both directions. In this way, the occurrence of torsion in the second vibration tone was avoided. The foundation was built over a 70 cm thick foundation slab, which significantly reduced the stresses on the ground.

The quality of the building was increased in terms of accessibility, because an elevator was added. Since it is not possible to place it inside the building due to lack of space, it is built outside. It is built as a steel frame structure, connected to the building at the level of the staircase landing. Figures 18 and 19 show the structures of the replacement building.

7. Conclusion

The apartment block on Croatian national revival street in Sisak was built before the first seismic regulations were adopted in 1964. Its relatively high number of floors, thin walls, lack of vertical tie beams, wall discontinuities on the ground floor, and other original deficiencies regarding earthquake resistance are clear indicators of the risks posed by buildings from that construction period. Conducting investigative work through an examination of existing documentation and field research is an essential step in assessing the condition of the structure. After an earthquake, buildings need to be inspected, and all damage must be identified. However, the observed damage alone is not a sufficient indicator of the actual structural condition of the building. To determine the actual seismic resistance of the structure, a structural analysis of the existing state must be performed. Only after completing this analysis can the appropriate level of reconstruction be determined. According to the Technical regulation for building structures, designers are responsible for ensuring the mechanical resistance and stability of a building not only in its final state but also during construction. Additionally, a comprehensive financial comparative analysis between a complex reconstruction of the existing building and the construction of a new replacement building must be conducted. A proper reconstruction project can only be implemented after all these steps are correctly carried out.

For the building in question, due to its extremely low level of existing seismic resistance, achieving the required resistance level (Level 3) specified by the technical regulations would necessitate extensive structural interventions. This would involve integrating a new reinforced concrete structure into the existing one. Given the layout of the load-bearing walls in the current state, determined by the dense arrangement of small apartments, the only feasible solution would be to replace them with reinforced concrete walls in the same positions. Such interventions would require complex temporary supports during construction.

The main reasons for opting for the construction of a new building were the designer's obligation to ensure the load-bearing capacity and stability of the structure during construction, as well as the financial comparative analysis of the complex reconstruction versus the construction of a new building. Due to the building's extreme damage and deformation, the designer could not meet the requirement outlined in Article 7 of the Technical regulation for building structures, which mandates ensuring load-bearing capacity and stability during construction, as well as guaranteeing the safety and health of workers.

The financial comparative analysis also revealed that the cost of a complex reconstruction would exceed €2,000/m², whereas, according to the designer's calculations, the cost of constructing a new building would not exceed €1,500/m².

REFERENCES

[1] Kovač, D.: Elaborat ocjene postojećeg stanja građevinske konstrukcije, 2023.

[2] Kovač, D.: Projekt obnove konstrukcije zgrade, 2023.

[3] Kovač, D.: Projektna i fotodokumentacija projektnog ureda Capital Ing d.o.o. Zagreb

[4] Grűnthal, G.: European Macroseismic Scale, Cahiers du Centre Europeen de Géodynamique et de Séismologie, Vol. 15, 1998.

[5] Tehnički propis za građevinske konstrukcije (NN 17/17, 75/20 i 7/22)

[6] Uroš, M., Atalić, J., Crnogorac, M., Kovač, D., Pojatina, J., Tarnik, K., Todorić, M., Trogrlić, B.: Procjena oštećenja zidanih zgrada nakon potresa, 9. Sabor Hrvatskih graditelja, 2024.

[7] HRN EN 1998-1 : Projektiranje potresne otpornosti , 1. dio – opća pravila, potresna djelovanja i pravila za zgrade, 2011

[8] Circolare 2 febbraio 2009, n. 617 C.S.LL.PP.

[9] Frančišković, Ž.: Stručno mišljenje o stanju nosive konstrukcije građevine nakon potresa, 29.12.2020., 2021.

[10] Radnić, J., Grgić, N., Buzov, A., Banović, I., Smilović Zulim, M., Baloević, G., Sunara, M.: Potres Petrinja magnitude Mw 6.4: glavni parametri potresa, utjecaj na građevine i preporuke za njihovu obnovu, GRAĐEVINAR, 73 (2021) 11, pp. 1109-1128, doi: https://doi.org/10.14256/JCE.3243.2021

[11] Uroš, M., Šavor Novak, M., Atalić, J., Sigmund, Z., Baniček, M., Demšić, M., Hak, S.: Procjena oštećenja građevina nakon potresa – postupak provođenja pregleda zgrada, GRAĐEVINAR, 72 (2020) 12, pp. 869-893, doi: https://doi.org/10.14256/JCE.2969.2020

[12] Perić, L., Matorić, I.: Pouzdanost tradicionalnih zidanih građevina s drvenim stropovima na potresno djelovanje, GRAĐEVINAR, 72 (2020) 10, pp. 869-893, doi: https://doi.org/10.14256/JCE.2965.202