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Nonlinear static analysis of the behavior of an existing masonry building under seismic action

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Abstract: In the territory of Herzegovina there is a large number of masonry buildings, the age of which exceeds 50 years. These are mostly smaller buildings, with one to two floors, while the load-bearing walls are mainly made of cut stone in lime mortar. Larger buildings with load-bearing walls made of bricks and concrete blocks in cement-lime mortar appeared a little later. The floor structures mainly consist of timber oak beams, supported by load-bearing walls, with boarding on the upper side and plaster, on reed netting, on the lower side. Such structures are exceptionally sensitive to seismic action, and almost certainly could not withstand the design seismic load without significant damage and collapse. A nonlinear static analysis of the structure of one such building was performed in this paper. A check of the existing condition was performed in the first analysis, and a check of the partially strengthened structure in the second. It is evident from the analysis that such strengthening significantly improves the bearing capacity, while the increase in deformability (ductility) is significantly smaller due to the very high stiffness of such structures.

Key words: masonry walls, seismic action, nonlinear static analysis, structural strengthening

Nelinearna statička analiza ponašanja postojeće zidane zgrade na potresno djelovanje

Sažetak: Na području Hercegovine nalazi se veliki broj zidanih objekata, čija starost premašuje 50 godina. Uglavnom se radi o manjim objektima, s jednom do dvije etaže, dok su nosivi zidovi uglavnom od obrađenog kamena u vapnenom mortu. Malo kasnije javljaju se i veći objekti s nosivim zidovima od opeke, i betonskih blokova u produžnom mortu. Stropne konstrukcije uglavnom se sastoje od drvenih hrastovih greda, oslonjenih na nosive zidove, s daščanom oplatom s gornje strane i mortom, po pletivu od trske, s donje strane. Takve konstrukcije su iznimno osjetljive na potresno djelovanje, te gotovo sigurno ne bi mogle izdržati projektno potresno opterećenje, bez značajnih oštećenja i rušenja. U ovom radu izvršena je nelinearna statička analiza konstrukcije jednog takvog objekta. U prvoj analizi izvršena je provjera postojećeg stanja, a u drugoj provjera djelomično ojačane konstrukcije. Iz analize je vidljivo da takvo ojačanje značajno poboljšava nosivost, dok je povećanje deformabilnosti (duktilnosti) znatno manje zbog jako velike krutosti ovakvih konstrukcija.

Ključne riječi: zidani zidovi, potresno djelovanje, nelinearna statička analiza, ojačanje konstrukcije

1. INTRODUCTION

In our area, there is a large number of buildings with masonry walls as the main vertical load-bearing elements. Older buildings were constructed mainly with walls made of cut stone in lime mortar, while construction of buildings with walls made of normal size bricks, in cement-lime mortar, began in the mid-1960s. Later, brick blocks, as well as concrete and lightweight concrete blocks of different dimensions, came into use. These buildings generally have a simple plan shape, and have a regularity in terms of height. These are usually buildings with two floors, and less often (later) masonry buildings with up to five floors are constructed.

Another important structural characteristic of such buildings are the timber floor structures, made of timber beams and boarding. The beams are mainly made of hardwood (oak), while the boarding is made of softwood (fir, spruce). Timber floors have low stiffness in their plane, as well as low bending stiffness, and their connection with load-bearing walls is weak, which directly leads to the poor behavior of such structures under seismic action.

It has been shown over time that such buildings are highly sensitive to seismic actions, and they should be strengthened, as required by modern regulations (EC 8-3 part [1]). The strengthening is related to increasing the integrity of the entire structure (connection of all elements into one whole - box concept), then to increasing the load-bearing capacity of the connections between the walls and the floor structure and/or the roof structure. Increasing the stiffness of the floor structure in its plane is also a frequent need, and so is increasing the bearing capacity and deformability of the walls in their plane and perpendicular to it. Sometimes it is necessary to stiffen the roof structure itself and improve its connection with the gable walls.

Before any intervention on the building structure, it is necessary to perform an analysis of its existing state (nonlinear static analysis), and based on these results, to perform strengthening of the structure and verification of the obtained results. In order to be able to do this, it is necessary to be familiar with the mechanical properties of the incorporated materials, as well as the mechanical properties of walls and floor structures [2, 9, 15].

The methods of analyzing the improvement of the seismic response of ordinary masonry structures are constantly being improved. In the previous two or three decades, the displacement-based analysis concept led to the increased application of nonlinear static methods (pushover) [5, 7, 10]. These methods result in a comparison of displacement capacity and required capacity, from which important conclusions can be drawn about the behavior of the masonry structure and its condition under the action of seismic loading.

2. BRIEF DESCRIPTION OF THE BUILDING

In this paper, a nonlinear analysis of the structure of an existing building, constructed in the 1950s, was performed, and the effect of strengthening the structure by adding prestressed steel bars at the level of the floor structures, and strengthening individual walls in the same way, was analyzed. The building has dimensions of 14.6x10.5m, and three floors, one of which is a basement. The ceiling above the basement is vaulted with an arch made of bricks, of a standard size (25x12x6.5 cm). Due to the great stiffness of the basement floor and the fact that it is almost completely embedded in the ground, it will not be taken into account when analyzing the structure for earthquake action. Also, the subsequently added part above the entrance to the ground floor was not taken into account due to its very low stiffness in relation to the original building. The roof is a classic gable roof, with a timber roof structure made of softwood. The roof is covered with tiles. The floor structure above the ground floor and first floor consists of timber oak beams, at an approximate axis distance of 65 cm, and single-layer boarding. The beams are supported by the longitudinal load-bearing walls and it can be assumed that they are not sufficiently connected to the load-bearing walls. The load-bearing walls have a thickness of 50 cm, and they are constructed of bricks of normal size. The vertical communication between the ground floor and the first floor consists of a wooden spiral staircase, framed by brick walls, 38 cm thick (Figure 2).

Figure 1. Photographs of the building

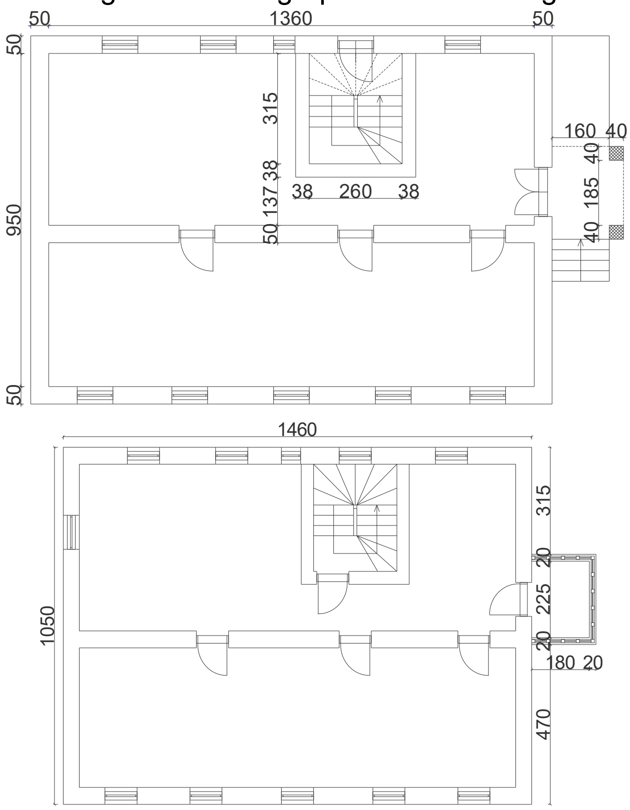

Figure 2. Ground floor and first floor plans

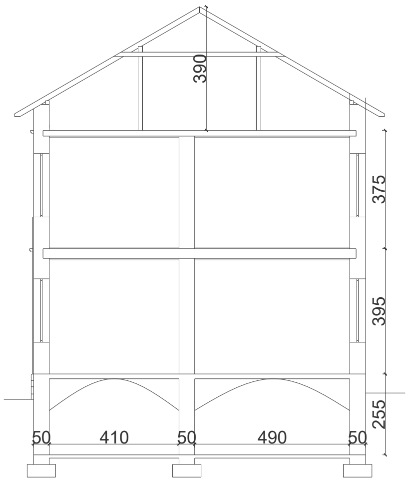

Figure 3. Cross section

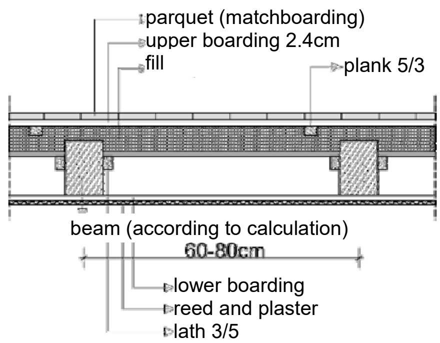

Figure 4. Typical section of the timber floor [16]

3. SEISMIC ACTION AND MECHANICAL PROPERTIES OF MATERIALS

The maximum ground acceleration of the building location, for the return period of $T_{NCR} = 475$ years, is $a_{gR} = 0.25g$, with the probability of exceedance of 10% in 50 years [11]. This value is taken for the control of limit state SD (significant damage), while limit state DL (damage limitation) and limit state NC (near collapse) are not controlled in this text [1]. Figure 5 a) shows the elastic acceleration spectrum, for soil type B and damping of 5%, while Figure 5 b) shows the elastic spectrum in AD format. The importance factor of the building is $\gamma_I = 1.0$, so that the design ground acceleration is equal to $a_{gR} = 2.45\ m/s^2$.

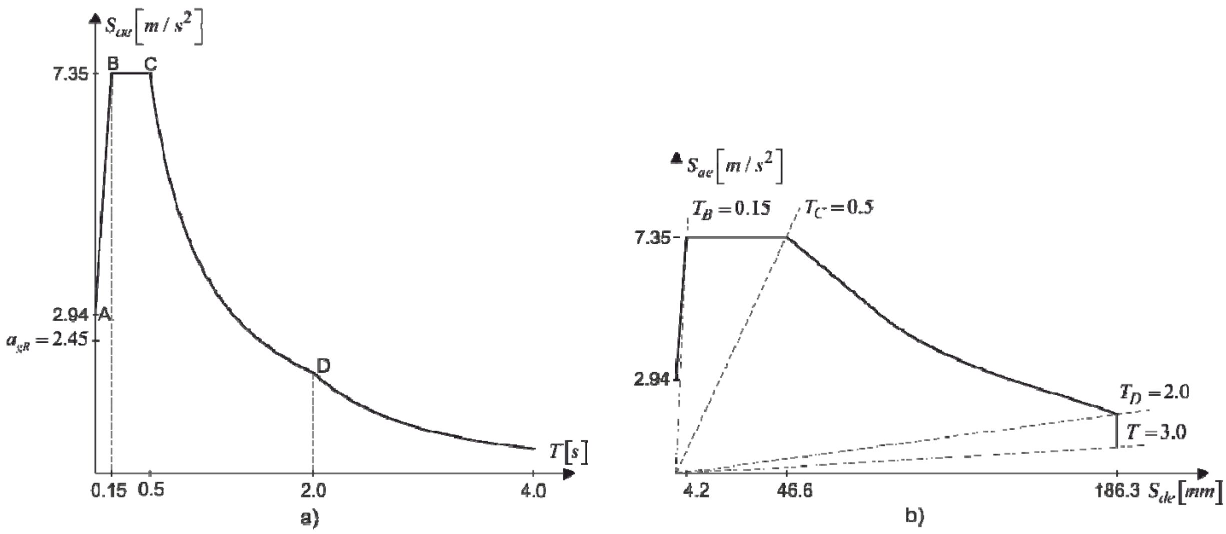

Figure 5. a) elastic spectrum for soil type B and damping of 5% [1], b) AD format

The masonry control category B, and the wall element production category II were adopted. From this it follows that the safety factor for walls is $\gamma_m = 2.5$. For existing masonry structures, the average compressive strength of the walls must be divided by the confidence factor, which depends on the knowledge level. For this structure, the knowledge level KL1 is assumed, and the confidence factor is equal to $CF_{KL1} = 1.35$ [1].

The mechanical properties of the walls and floor timber beams are adopted according to Table 1. As the mechanical properties of the materials were not tested, approximate values were taken from the available literature [3, 5, 8, 9]. Determining the approximately accurate values of the mechanical properties of the existing walls is a complex task, which is the reason for the large range of values, for example, of the modulus of elasticity of the walls, in which it varies. For example, according to Tomaževič [8], the modulus of elasticity of the walls is in the range $200 f_k \leq E \leq 2000 f_k$.

Table 1. Mechanical properties of materials

Material	**Modulus of elasticity E** [N/mm^2]	**Shear modulus G** [N/mm^2]	**Compressive strength f_{c0k}** [N/mm^2]	**Tensile strength f_{t0k}** [N/mm^2]	**Shear strength f_{vk}** [N/mm^2]
Hardwood D30	10000.0	600.0	23.0	18.0	3.0
Walls	1440.0	360.0	3.0	0.0	0.25

4. MODELING OF MASONRY STRUCTURES

The seismic behavior of modern masonry structures is characterized by the resistance of individual walls in their plane, and with stiff floor structures that connect the masonry walls well, these structures develop spatial resistance to seismic action. This fact mostly does not apply to older structures, with timber floor structure, which does not provide a good connection of the masonry walls in the horizontal plane.

Typical failure mechanisms of masonry walls are shown in Figure 6. There are several factors that influence the failure mechanism of masonry walls, such as the wall geometry (the wall length to height ratio, and the size and position of openings in the wall), the quality of the material (wall element and mortar), boundary conditions and the load on the wall. Flexural failure (a) occurs due to the lifting and lowering of the ends of the wall, where the pressed edge of the wall is crushed. Sliding failure (b) occurs due to poor mortar quality and low level of vertical compressive stress. The mechanism of diagonal cracks (c) is also a consequence of shear forces. In this case, the crack can occur along horizontal and vertical joints (stepwise), which is a consequence of the poor quality of the mortar. A diagonal crack can also extend through the wall element, which is a consequence of the insufficient tensile strength of the wall elements (bricks, blocks).

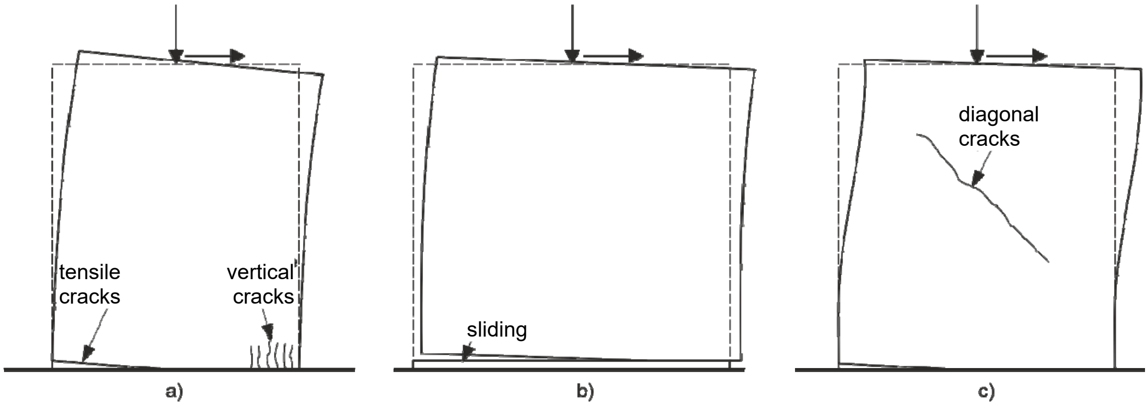

Figure 6. Typical modes of masonry panel failure [6]: a) uplift, b) shear sliding, c) diagonal cracking

Different approaches to modeling masonry walls are properly described in the literature [12]. The finite element method is the most widespread method for structural analysis, and the accuracy of its results largely depends on the adopted material model. As walls are a complex heterogeneous material, the practical application of FEM requires a large calculation volume and complex material behavior models. For this reason, for the modeling of masonry structures in practice, models based on their simulation with beams and panel-type elements (macro elements) are increasingly used.

Nonlinear behavior of non-reinforced masonry walls is mainly related to their behavior in the plane (plane stress state). Models based on macro-elements are led by this fact. Modeling of masonry walls with macro-elements is mainly reduced to two approaches. The first approach treats the walls as frames, consisting of rigid nodes and deformable beam elements. Such an approach was implemented in the 3Muri software package [4], which is used in this analysis. The second approach uses planar macro-elements.

Many experimental studies on the behavior of masonry walls in their plane, as well as out of plane, under seismic actions have been carried out in recent decades. Some of them are described in the references [2 and 3].

4.1 Equivalent frames

Equivalent frames are a simple approach to the nonlinear analysis of masonry structures. The walls are observed as idealized frames, where the deformable elements, piers and spandrels, are connected by rigid connections (Figure 7). The nonlinear behavior of the walls includes a cyclic relationship of stresses and deformations, where the tensile strength of the walls is excluded. The piers are the main load-bearing vertical elements, which take lateral and vertical loads, while the spandrels are called secondary horizontal elements, which connect the adjacent deformable piers. They affect the boundary conditions of the piers, and therefore have a significant influence on the behavior of the wall loaded with seismic forces.

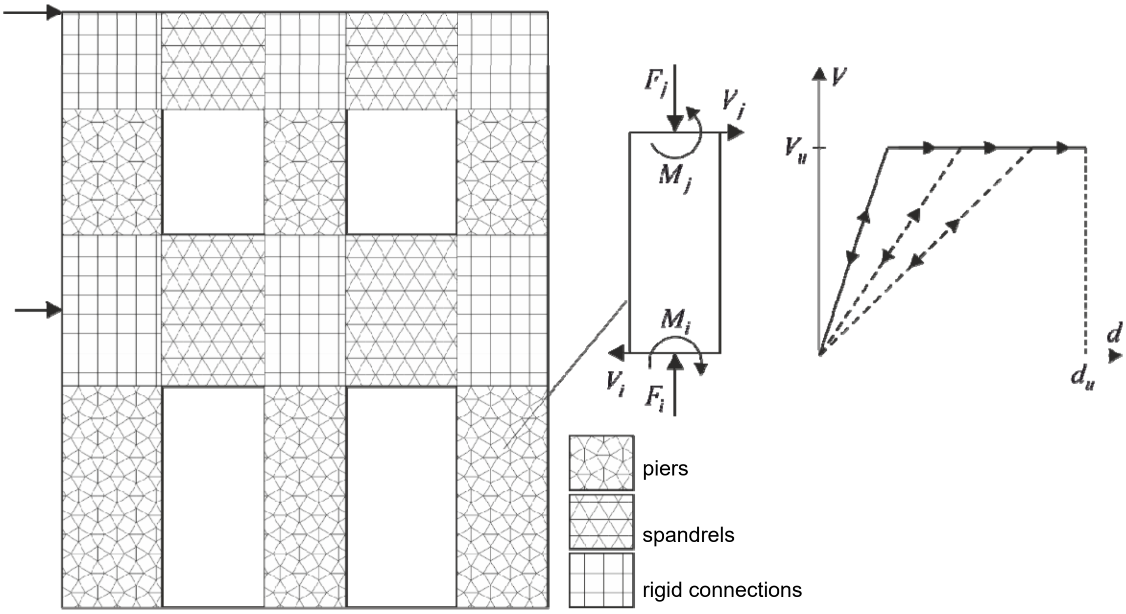

Figure 7. Idealization of the wall with an equivalent frame [6]

A nonlinear beam element is used to model piers and spandrels. The initial stiffness of the element is defined by the elastic properties, or properties of the fractured material. The relationship between shear force and displacement (moment and angle of rotation) is bilinear (Figure 7). The element enables the redistribution of internal forces and the detection of damage at limit states. Stiffness degradation in the plastic range occurs according to the defined relationship between force and displacement. Ductility control is performed on the basis of determining the maximum displacement d_u that occurs with a certain failure mechanism. According to EC 8, for the limit state DL (damage limitation) this displacement is:

$$d_u = \begin{cases} 0.004 & shear \\ 0.008\frac{H_0}{l_w} & pressure - bending \end{cases}, \tag{1}$$

where l_w is the wall length, and H_0 is the distance from the section where the bending capacity is reached to the point of inflection.

The limit bending moment is given by the expression:

$$M_u = \frac{l_w^2 b_w \sigma_0}{2}\left(1 - \frac{\sigma_0}{0.85 f_m}\right) = \frac{N l_w}{2}\left(1 - \frac{N}{N_u}\right), \tag{2}$$

where b_w is the wall thickness, f_m the average compressive strength of the walls, N the longitudinal force, and $\sigma_0 = N / l_w b_w$. For existing masonry structures, the average compressive strength of the walls f_m must be divided by the confidence factor, which depends on the knowledge level. Since this is a building constructed in the 1950s, the first knowledge level KL1 is assumed, and the confidence factor is equal to $CF_{KL1} = 1.35$ [1].

Shear failure is defined by the Turnšek-Čačović criterion [13], which is more appropriate for existing masonry structures than the Mohr-Coulomb criterion:

$$V_u = l_w b_w \frac{1.5\tau_0}{b}\sqrt{1+\frac{\sigma_0}{1.5\tau_0}} = l_w b_w \frac{1.5\tau_0}{b}\sqrt{1+\frac{N}{1.5\tau_0 l_w b_w}}. \tag{3}$$

The coefficient b depends on the wall height to width ratio, $b = h_w / l_w$, with the limitation $1 \le b \le 1.5$.

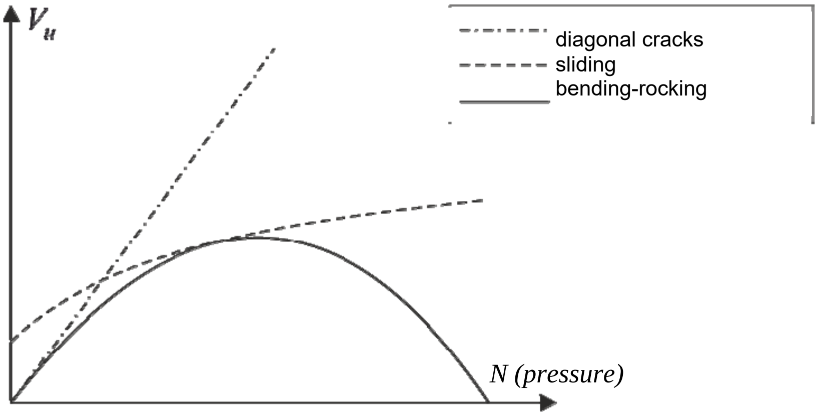

Figure 8. Comparison of criteria [4]

5. ANALYSIS OF THE BUILDING STRUCTURE

Two analyses of the structure of this masonry building were conducted. The first analysis is related to the original structure, while the second one is concerned with the strengthened structure. The strengthening was carried out with prestressed steel bars (4Φ16) in the direction of all load-bearing walls, at the levels of the floor structures. The bars are placed in the grooves, two bars on the outside and inside of the wall each, and anchored over anchor plates d=20mm, which are located at the ends of the walls. The grooves are subsequently filled with mortar. The bars in the 50 cm thick walls are prestressed with a total force of 20 kN, while the bars in the 38 cm thick walls are prestressed with a total force of 10 kN. S335 quality steel was taken. Diagonal strengthening of walls P2, P6 on the ground floor was also carried out with prestressed steel bars S335, diameter 32 mm. The prestressing force is 10.0 kN. Likewise, wall P2 (entrance to the building) was strengthened in the same way, on both floors. The diagonal bars, with a diameter of 32 mm, are prestressed with a force of 20.0 kN.

The total gravity load of floor structures is taken according to the following expression:

$$q = 1.0g + 0.3p = 1.0 \cdot 2.5 + 0.3 \cdot 2.0 = 3.1\, kN / m^2,$$

while the roof load is taken in the amount:

$$q = 1.0g + 0.3s = 1.0 \cdot 1.2 + 0.3 \cdot 1.0 = 1.5\, kN / m^2.$$

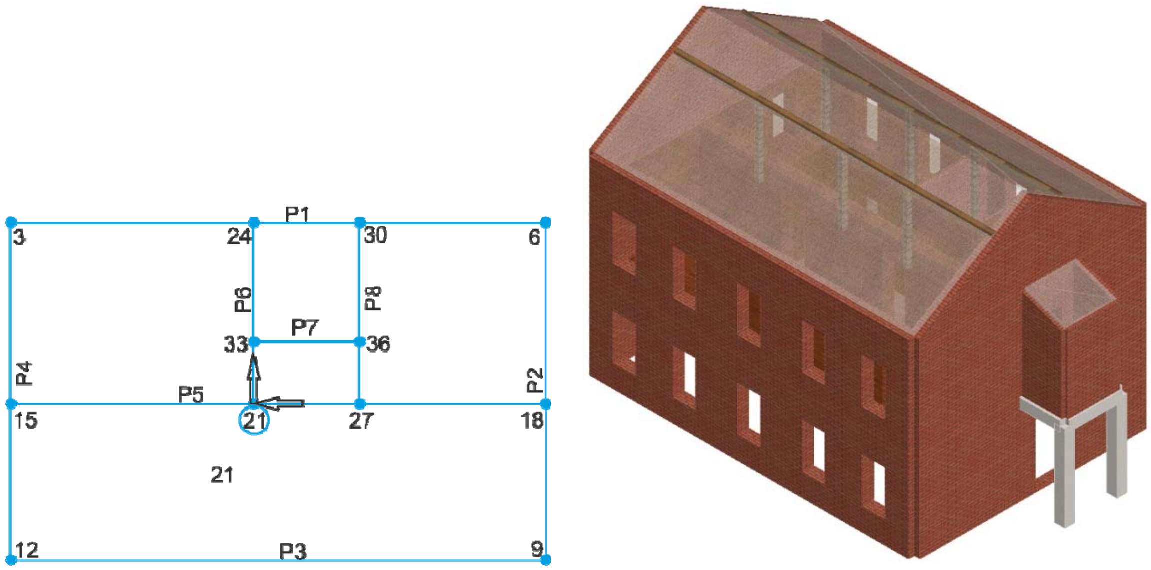

Figure 9. Position of walls and nodes, and 3D model of the building

In both cases, eight pushover analyses were performed, as shown in Table 2.

Table 2. Global pushover analyses of the structure

Analysis	Direction	Distribution of lateral forces by height	Design ground acceleration a_{gR} [m/s^2]	Level	Control node
1	+x	uniform	2.45 m/s^2	2	21
2	+x	static		2	
3	-x	uniform		2	
4	-x	static		2	
5	+y	uniform		2	
6	+y	static		2	
7	-y	uniform		2	
8	-y	static		2	

5.1 Existing condition of the structure

Table 3 shows the analysis results of the existing structure, without strengthening. It is evident that the structure is not satisfactory, with the most significant analyses being 4 and 6, in the –X and +Y directions, respectively (rows colored yellow). Capacity (pushover) curves are shown in Figures 11 and 13, for the global X and Y directions. It is obvious from the disposition of the building that the deformability of the structure in the Y direction is significantly greater, and therefore the damage to the walls lying in that direction is greater than the damage to the walls lying in the X direction. In addition, the floor beams are supported by the walls lying in the X direction, so their connection is greater than the connection of the walls lying in the Y direction.

Table 3. Results of the global pushover analysis of the structure (actual state)

Analysis	Direction	Distribution of lateral forces by height	Ecc. [mm]	dt SD [mm]	dm SD [mm]	SD Ver.
1	+X	Uniform	0	43.54	32.31	No
2	+X	Static	0	43.36	55.40	Yes
3	-X	Uniform	0	20.15	13.66	No
4	-X	Static	0	22.48	13.40	No
5	+Y	Uniform	0	15.52	10.19	No
6	+Y	Static	0	17.16	10.83	No
7	-Y	Uniform	0	13.67	11.14	No
8	-Y	Static	0	15.32	12.09	No

Analysis	Direction	Distribution of lateral forces by height	Ecc. [mm]	α SD
1	+X	Uniform	0	0.754
2	+X	Static	0	1.267
3	-X	Uniform	0	0.727
4	-X	Static	0	0.646
5	+Y	Uniform	0	0.719
6	+Y	Static	0	0.689
7	-Y	Uniform	0	0.865
8	-Y	Static	0	0.839

The compliance factor α denotes the maximum value of the load, taken into account in a particular limit state, that the structure can take, i.e.:

$$\alpha_{SD} = \frac{PGA_{CSD}}{PGA_{DSD}} = \frac{capacity}{requirement}, \tag{4}$$

where PGA_{CSD} is the acceleration capacity corresponding to limit state SD, and PGA_{DSD} is the peak ground acceleration corresponding to the limit state SD.

A check of wall collapse is performed through relative floor displacements:

$$\delta = \frac{u_{i+1} - u_i}{h_{kata}} + \frac{\varphi_i + \varphi_{i+1}}{2} \le \delta_0, \tag{5}$$

where the limit value of the relative floor displacement δ_0 depends on the type of wall failure, and ranges from 0.4% to 0.8%. The values of the relative floor displacements of the existing structure, for the X direction, are given in table 4, while in table 5 those values are given for the Y direction.

For the X direction, analysis 4 is the most significant, while for the Y direction, analysis 6 is the most significant. Figure 10 shows the degree of damage to wall P5, for the maximum displacements reached in the X direction, while Figure 12 shows the degree of damage to wall P2.

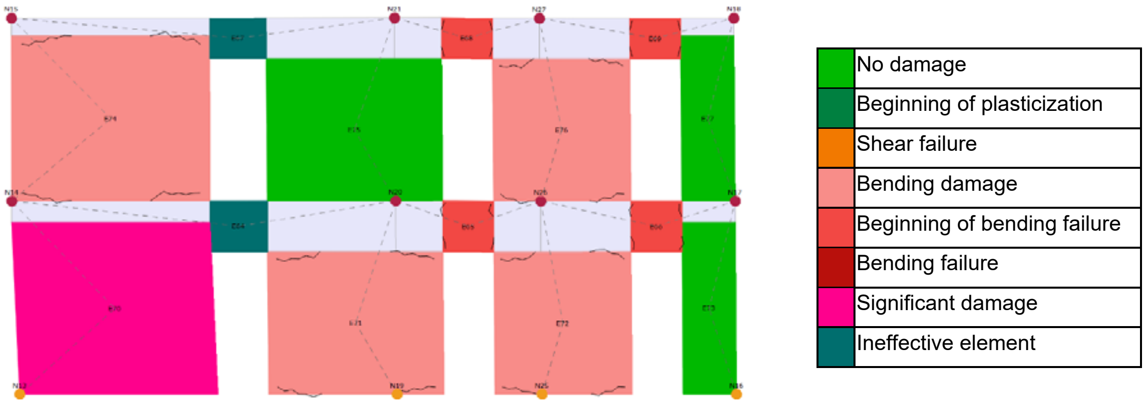

Figure 10. Failure mechanism of wall P5 when the displacement $d_m = 13.4\ mm$ is reached (3)

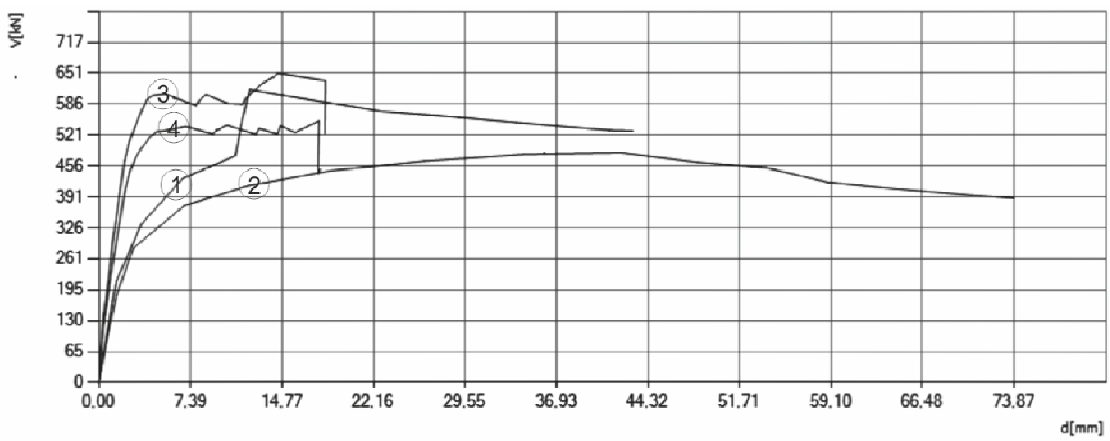

Figure 11. Pushover (capacity) curves for the X direction (actual state)

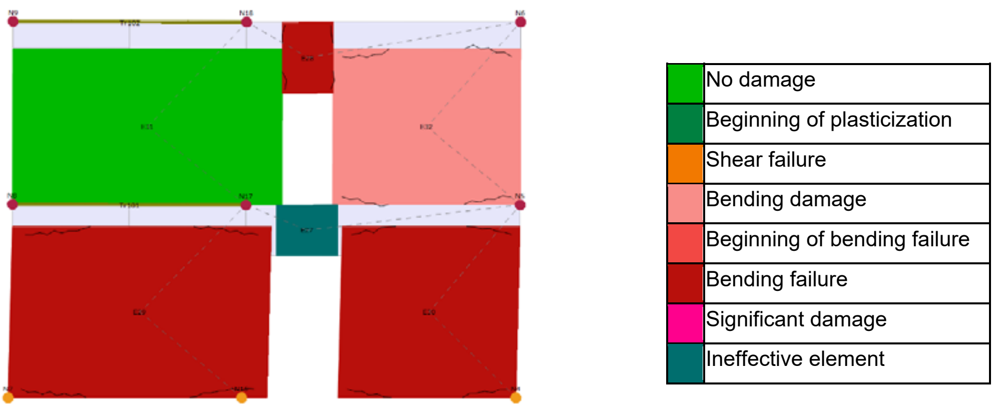

Figure 12. Failure mechanism of wall P2 when the displacement $d_m = 10.8\ mm$ is reached (6)

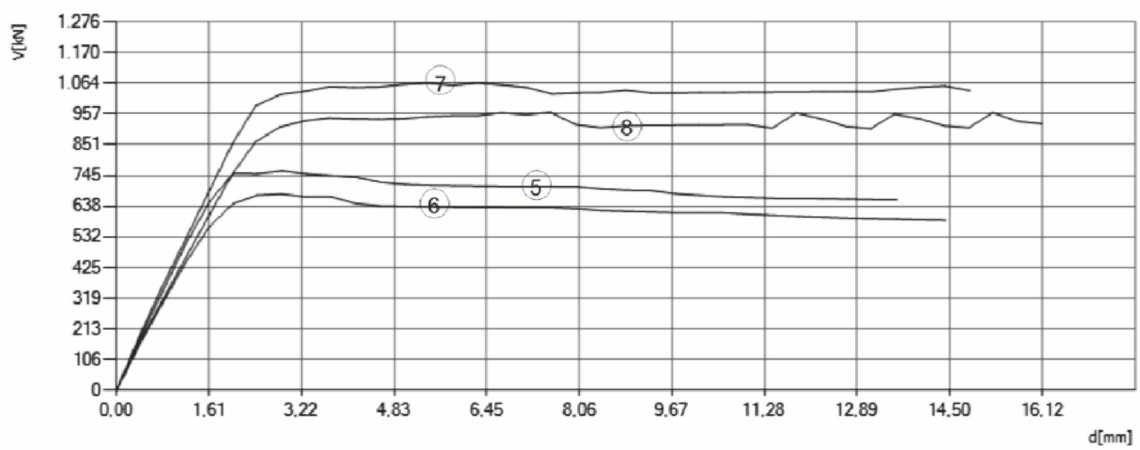

Figure 13. Pushover (capacity) curves for the Y direction (actual state)

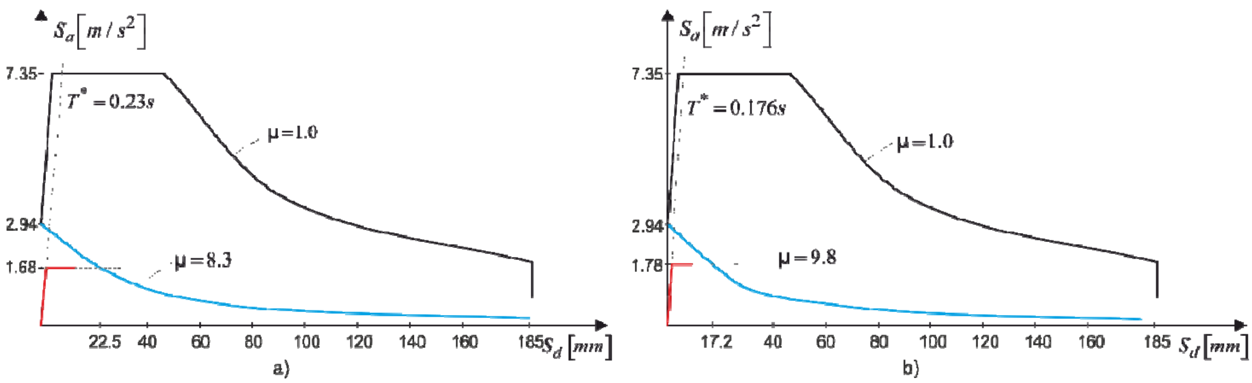

Figure 14. Results of the analysis of the unstrengthened str., a) analysis 4, b) analysis 6

Table 4. Relative floor displ. X direction (4)

Wall	Node at the bottom	Node on top	Relative displacement [mm]	Floor
1	1	2	16.95	1
1	2	3	19.36	2
3	10	11	12.37	1
3	11	12	21.70	2
5	13	14	34.33	1
5	14	15	0.45	2
7	31	32	0.89	1
7	32	33	16.45	2

Table 5. Relative floor displ. Y dir. (6)

Wall	Node at the bottom	Node on top	Relative displacement [mm]	Floor
2	7	9	22.78	1
2	8	9	3.27	2
4	10	11	0.28	1
4	11	12	0.28	2
6	19	20	12.07	1
6	20	21	1.93	2
8	25	26	16.60	1
8	26	27	2.54	2

From table 4 it can be seen that the relative displacement of the first floor of wall P5 is 0.9%, of the second floor of wall P7 it is 0.47%, of wall P3, in the second floor 0.57%, while the relative floor displacement of wall P1 in the second floor is 0.51%. Therefore, wall P5, which extends in the X direction, does not meet the criteria given by expression (5) or (1).

This should be observed in the light of the fact that the existing structure have not met the requirement defined by the spectrum (Figure 14). The same applies to the Y direction.

Table 6 shows the analysis results of the strengthened structure. It can be seen that the structure meets the design seismic load, where the most significant analyses are 3 and 6, in the +X and +Y directions, respectively (rows colored yellow). Capacity (pushover) curves are shown in Figures 15 and 16, for the global directions X and Y, while Figure 17 shows the damage levels of walls P2 and P5. It is evident from this figure that both walls are dominantly deformed by bending, and the level of damage is low. In the strengthened wall P2, the prestressed tension elements take over the principal tensile stresses, and to a certain extent also the shear stresses, which led to the acceptable behavior of this wall under the action of the design seismic load.

Table 6. Results of the global pushover analysis of the structure (strengthened structure)

Analysis	**Direction**	**Distribution of lateral forces by height**	**Ecc. [mm]**	**dt SD [mm]**	**dm SD [mm]**	**SD Ver.**
1	+X	Uniform	0	7.46	13.69	Yes
2	+X	Static	0	10.74	17.60	Yes
3	-X	Uniform	0	6.23	9.39	Yes
4	-X	Static	0	8.93	16.46	Yes
5	+Y	Uniform	0	11.31	13.09	Yes
6	+Y	Static	0	14.41	15.21	Yes
7	-Y	Uniform	0	9.23	15.51	Yes
8	-Y	Static	0	11.68	18.09	Yes

Analysis	**Direction**	**Distribution of lateral forces by height**	**Ecc. [mm]**	**α SD**
1	+X	Uniform	0	1.371
2	+X	Static	0	1.386
3	-X	Uniform	0	1.227
4	-X	Static	0	1.489
5	+Y	Uniform	0	1.074
6	+Y	Static	0	1.030
7	-Y	Uniform	0	1.339
8	-Y	Static	0	1.286

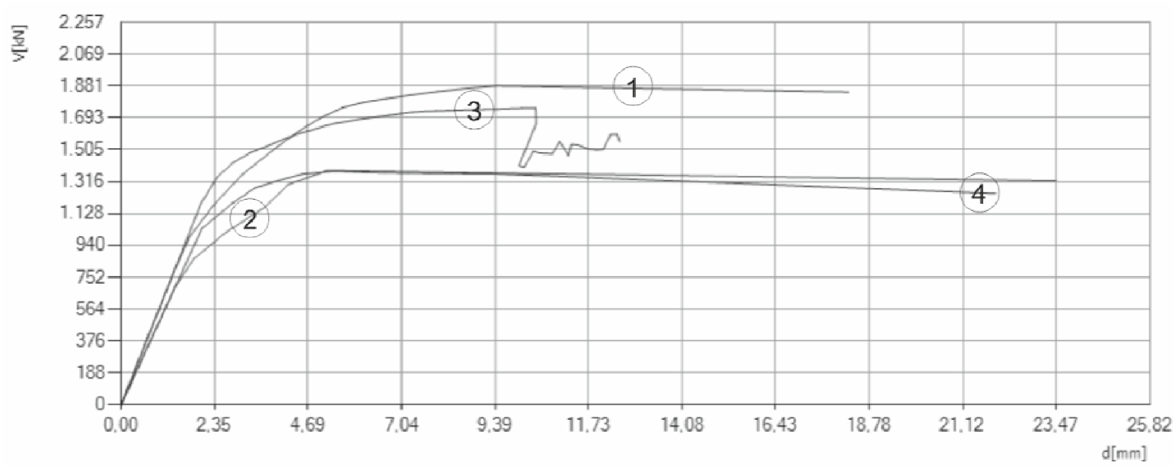

Figure 15. Pushover (capacity) curves for the X direction (strengthened structure)

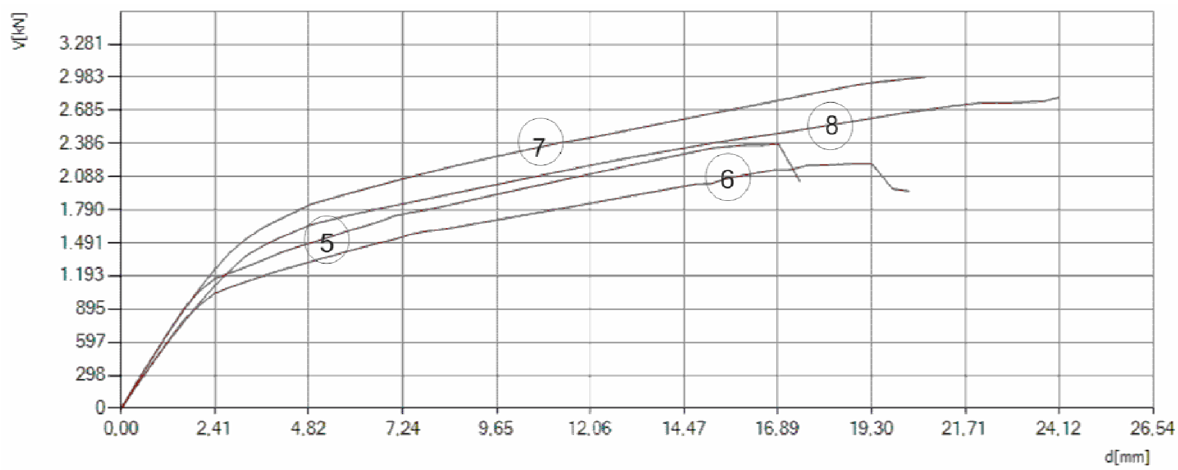

Figure 16. Pushover (capacity) curves for the Y direction (strengthened structure)

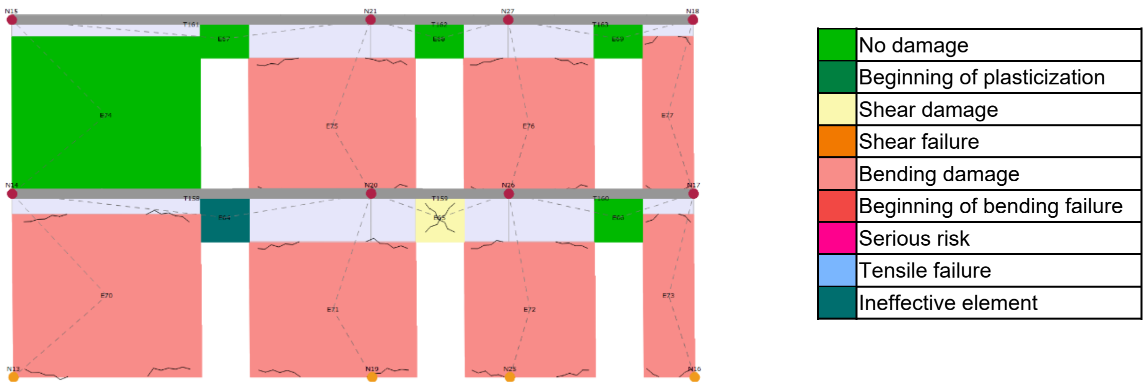

Figure 17. Damage levels to wall P5 (6) at target displacement (strengthened structure)

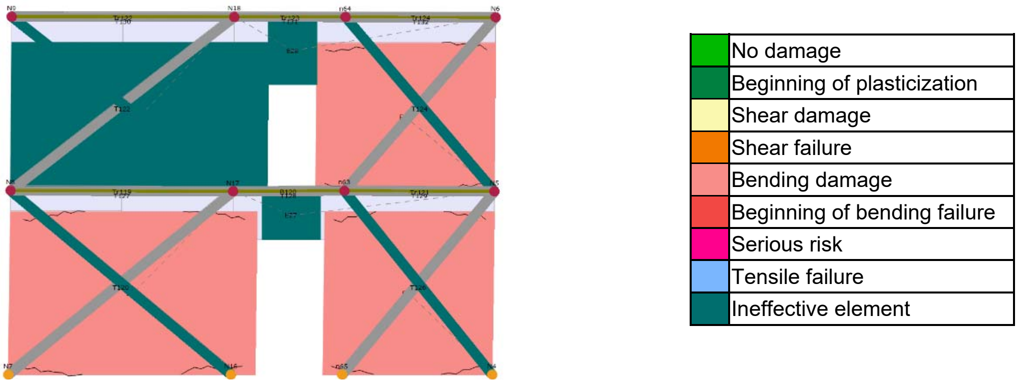

Figure 18. Damage levels to wall P2 (3) at target displacement (strengthened structure)

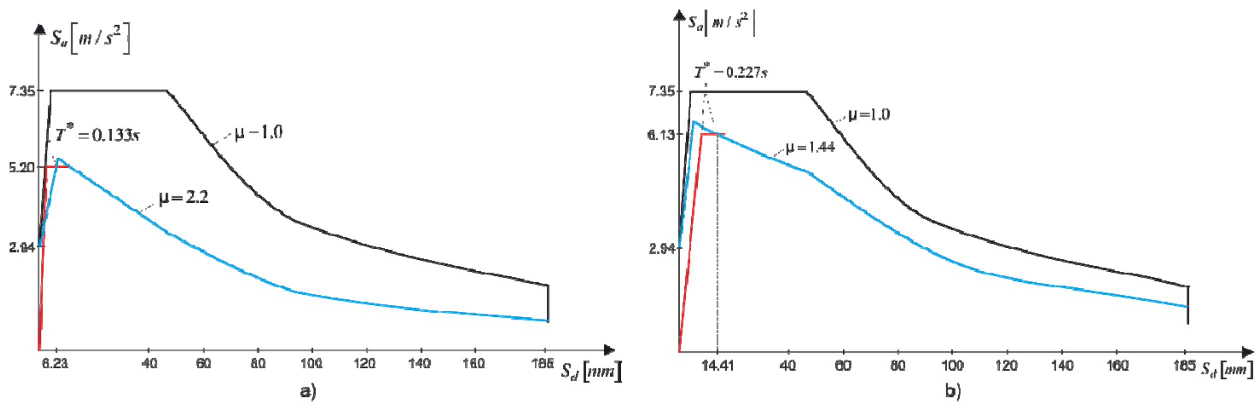

Figure 19. Results of the analysis of the strengthened structure, a) analysis 3, b) analysis 6

Table 7. Relative floor displ. X direction (3)

Wall	Node at the bottom	Node on top	Relative displacement [mm]	Floor
1	1	2	3.18	1
1	2	3	1.51	2
3	10	11	0.71	1
3	11	12	0.55	2
5	13	14	15.18	1
5	14	15	4.96	2
7	31	32	1.38	1
7	32	33	18.92	2

Table 8. Relative floor displ. Y dir. (6)

Wall	Node at the bottom	Node on top	Relative displacement [mm]	Floor
2	7	9	25.3	1
2	8	9	5.74	2
4	10	11	4.58	1
4	11	12	2.77	2
6	19	20	15.3	1
6	20	21	4.68	2
8	25	26	19.44	1
8	26	27	5.34	2

After strengthening the structure by adding prestressed steel elements (4Φ16~1Φ32) at the height of the floor structure, as well as by strengthening the walls P2, P6 and P8, an improvement in its load-bearing capacity is visible. A comparison of the capacity curves

(Figures 11 and 15, and 12 and 16) shows a significant increase in the bearing capacity of the structure in both directions, while the deformability capacity is much less pronounced.

6. CONCLUSIONS

Based on the performed analyses of the building structure, the effect of strengthening of the most vulnerable parts (walls) of the structure can be seen. By pushover analysis of the structure in two orthogonal directions, a fairly clear insight into its condition is obtained, with regard to the formation and expansion of plasticization (damage) zones, assuming that possible torsional eigenvectors are not significantly expressed. On the basis of these findings, strengthening of the walls at the levels of floor structures, as well as of individual walls in their plane, was carried out with prestressed steel bars. From the comparison of the two analyses, the improvement in the response of the structure to the design seismic load is observable.

Ordinary masonry structures are very stiff, and it is difficult to achieve an increase in their deformability, or ductility, by strengthening. On the other hand, strengthening can significantly increase the bearing capacity of the structure, which is evident from the bearing capacity curves of the existing and strengthened structure.

Strengthening of masonry structures can be carried out in several ways (reinforced layer of shotcrete, prestressing in the vertical direction, FRP strips, strengthening of floor structures, strengthening of gable walls and the roof structure, and other methods). In the process, one should always take into account the practical application of a particular method of strengthening structures of masonry walls, because these are interventions that are not simple, and usually require significant financial resources.

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