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STRUCTURAL ANALYSIS OF CONDITIONS AND POSSIBLE UPGRADING OF THE SEISMIC PERFORMANCES OF MASONRY BUILDINGS WITH UNREINFORCED WALLS

D. Kostić[1], R. Folić[2], J. Savić[3], V. Milošević[4]

***Keywords**: existing masonry buildings, seismic analysis, strengthening*

***Research area**: masonry structures*

ABSTRACT

The number of storeys is limited in technical regulations for masonry brick buildings with unreinforced bearing walls. Those buildings that were built prior to the contemporary technical regulations do not have adequate seismic performance and, depending on their importance, demand appropriate intervention for their improvement. Two characteristic buildings (A and B) with different degrees of regularity are analyzed in this paper. In both of them stress and deformation state is determined by using "Fedra" specialized software package for masonry. Recommendations given in EN 1998 are discussed. The effects of different interventions on the structure, for improving their seismic resistance, of the considered building are analyzed and appropriate recommendations are formulated.

1. Introduction

Masonry buildings make up the biggest part of the engineering fund, where many of them were built before the current technical regulations and lack the adequate seismic

[1] D. Kostić, Prof. Dr., Faculty of Civil Engineering and Architecture, 14 Aleksandra Medvedeva St., Niš, Serbia, e-mail: dragan.kostic@gaf.ni.ac.rs

[2] R. Folić, Prof., Professor Emeritus University of Novi Sad, Faculty of Technical Sciences, Department CE and geodesy, 6 Trg D. Obradovića, Novi Sad, Serbia, e-mail: folic@uns.ac.rs

[3] J. Savić, MSc., 14 Aleksandra Medvedeva St., Niš, Serbia, e-mail: jelena.savic@gaf.ni.ac.rs

[4] V. Milošević, MSc., 14 Aleksandra Medvedeva St., Niš, Serbia, e-mail: vuk.milosevic@gaf.ni.ac.rs

performances. Earthquakes that have occurred in recent years have caused serious damage, even collapse of masonry buildings. As a result, much attention has been devoted to seismic protection, particularly in countries with strong seismic actions. Seismic protection of buildings is regulated by technical regulations (seismic design). Masonry structures are more susceptible to major damage than reinforced concrete and steel structures, primarily due to the structure's large mass and brittle walls. Difficulties in assessing the actual capacity of masonry buildings against seismic actions are related to the unreliability of the properties of the materials used and the quality of the works. Based on analyses of causes of earthquake damage it was concluded that the building's configuration and regularity also plays a significant role [1]. More favourable in earthquake conditions is the behaviour of buildings with compressed symmetrical bases, with adequately interconnected walls, and with floor structures.

In the case of low resistance of masonry buildings, ensuring the adequate performances (seismic resistance) requires strengthening the structure. The seismic resistance of masonry buildings significantly depends on the quality of walls, and whether they are non-reinforced, but confined using vertical and horizontal tie beams (confined walls), or reinforced walls. It is also an influential factor in limiting the number of floors in ordinary masonry buildings containing the above wall structures (Table 1). In addition to the number of floors, some conditions for the formation of the building's plan were also prescribed. The base of the building must be approximately rectangular, and the ratio between the length of its shorter and longer side must be higher or equal to 0.25. Deviations from the rectangular shape in the form of turns or depressions are up to 15% of the length sides parallel to the direction of delivery.

Table 1. Allowed number of storeys above ground and minimum horizontal area of shear walls for "*simple masonry buildings*", after *EN 1998*

Acceleration at site $a_g \cdot S$		$\leq 0{,}07 k \cdot g$	$\leq 0{,}10\ k \cdot g$	$\leq 0{,}15\ k \cdot g$	$\leq 0{,}20\ k \cdot g$
Masonry type	Number of floor (n)**	Minimal horizontal shear walls cross-sections in both direction in percentages of total floor area per storey (p_{Amin})			
Unreinforced masonry	1	2.0%	2,0%	3,5%	.n.a.
	2	2.0%	2,5%	5,0%	n.a.
	3	3.0%	5,0%	n.a.	n.a.
	4	5.0%	n.a.*	n.a	n.a.
Confined masonry	2	2.0%	2,5%	3,0%	3,5%
	3	2.0%	3,0%	4,0%	n.a.
	4	4.0%	5,0%	n.a.	n.a.
	5	6 .0%	n.a.	n.a.	n.a.
Reinforced masonry	2	2.0%	2,0%	2,0%	3,5%
	3	2.0%	2,0%	3,0%	5,0%
	4	3.0%	4,0%	5,0%	n.a.
	5	4.0%	5,0%	n.a.	n.a.

** Roof space (attic) not included in the number of storeys; *n.a. means "not acceptable".

The shear walls of a building should fulfil some conditions [1]: the building should be stiffened by shear walls, placed almost symmetrically in plane in two orthogonal directions, a minimum of two parallel walls should be placed in two orthogonal directions, the length of each wall being greater than 30% of the length of the building in the direction of the wall under consideration; at least for the walls in one direction, the distance between these walls should be greater than 75% of the length of the building in the other direction; shear walls should be continuous from the top to the bottom of the building. For unreinforced masonry buildings, walls in one direction should be connected with walls in the orthogonal direction at a maximum spacing of 7 m. In both orthogonal horizontal directions the difference in mass and in the horizontal shear wall cross-sectional area between adjacent storeys should be limited to a maximum value both of 20%.

For all structural members the ultimate limit state verification must be verified according to EC 6 and EC 8 ($E_d \le R_d$; resistance must be greater than actions). For buildings satisfying rules for "simple masonry buildings" it is not necessary to derive an explicit safety proof.

In EN 1998 – Part3 the following methods of analysis are recommended: multi-modal response spectrum analysis, nonlinear static analysis, nonlinear time-history analysis and *q*-factor approach. Criteria for structural intervention (retrofitting) and their design procedures are described in [2]. With introducing RC belts in masonry construction or possible transformation of existing non-structural elements into structural elements, the vulnerability of the structure as a whole can be eliminated.

This paper analyzes two types of existing masonry buildings in the city of Nis, providing a comparative analysis of their seismic response. The structures of these buildings are of different configurations and levels of regularity. The stress state was determined using the Phaedra software package [2], and analyzing the other parameters affecting the behaviour of the structure in earthquake conditions. The state of the structure has also been assessed upon the introduction of appropriate methods of strengthening.

2. Analysis and assessment of the state of the existing structures

The present paper analyzes two types of existing masonry buildings. The observed structures are typical by nature; they were constructed before the current technical regulations became effective, and are represented in the observed residential block in Nis. Similar buildings are present in over 70% of the housing fund. The analysis was conducted using the aforementioned Phaedra software package [2], which is based on a simple analytical procedure for assessing the behaviour of the structure in the process of its rehabilitation or changing its purpose. The way in which changes to the structure as part of the architectural change of purpose affect the seismic behaviour is also considered here.

Computer program **FEDRA**, intended for the design and analysis of masonry structures, is based on the finite element method [2]. The program is designed to be applied to buildings for which the major part of the loads is taken from the masonry. Also, all horizontal seismic forces are taken from the masonry, and it is assumed that RC columns, if any, do not take any seismic loads. The stiffness of the columns is negligible compared to that of the masonry walls. The floors should have enough horizontal stiffness to act as stiff diaphragms in the horizontal direction.

The design for the masonry is done for the ultimate limit state based on Eurocode 6, chapter 6. All the checks for loading cases 1.35g + 1.50q, and 1.00g + 0.30q + earthquake, are done for compression and shear. In addition, verification of slenderness ratio requirements and checks for strength at stress concentrations are performed according to Eurocode 6.

The seismic design is based on equivalent static loads at the level of each floor according to Eurocode 8. The distribution of the seismic force along the structure height is a reverse triangular distribution. Unreinforced, reinforced and confined masonry are designed according to the provisions given in Eurocode 6.

A specialized design module "gunites" offers option for design of strengthening walls applied to existing buildings by gunites-concrete jackets. Then, Fedra computes properties of an equivalent wall with increased shear strength.

Building „A“

Building A, of which the typical floor plan is shown in Figure 1, is a masonry building of 39.33 m wide and 14.63 m long rectangular plan. The number of floors is Basement + Ground floor + 3. The building was built in 1960 with walls of solid brick (MO10) in compo mortar (basement and ground floor) and lime mortar (I, II and III floors). In relation to the acceptance of vertical loads, the building's structural system consists of longitudinal 25 and 38 cm thick load-bearing walls without vertical tie beams. The building also contains 25 and 38 cm thick tier walls running in transverse direction. The floor structure is a 30 cm thick semi-prefabricated, ribbed structure ("Avramenko"). The floor height is 2.90 m. The roof is flat, executed as impassable rooftop terrace.

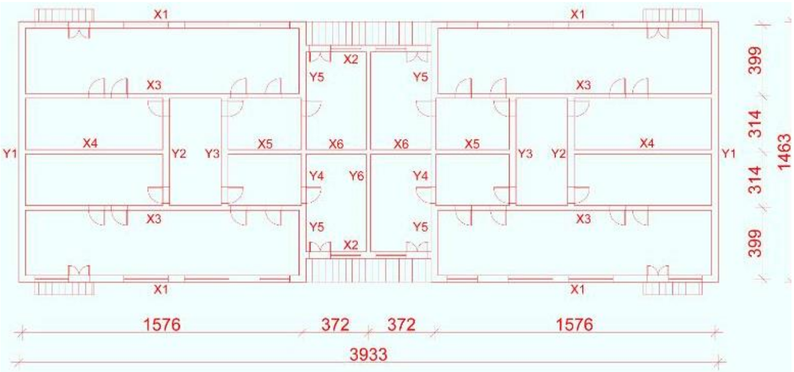


Figure 1. Plan of a typical floor, building A

Building "B"

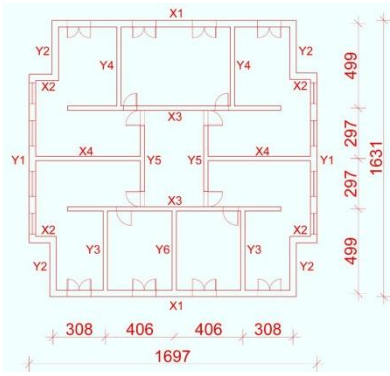


Table 2. Area of shear walls

	Percentage of walls in *X* directions	Percentage of walls in *Y* directions
Building „A“	12.2%	6.59%
Building „B“	9.1%	8.8%

Structures of building B is near regular, in opposite to building A in which deficit of walls in Y direction are registered.

Figure 2. Plan of the typical floor, building B

Building B (Figure 2) is a masonry building, roughly square in plan with the following dimensions: length 16.97 m width 16.31 m. The number of floors is Ground floor

+ 3. The building was built in 1960 with walls of solid brick (MO10) in compo mortar (ground floor) and lime mortar (I, II and III floor). In relation to the acceptance of vertical loads, the building's structural system consists of longitudinal 25 and 38 cm thick load-bearing walls without vertical tie beams, and 25 and 38 cm thick tier walls running in transverse direction. The floor structure is a 30 cm thick semi-prefabricated, ribbed structure ("Avramenko"). The floor height is 2.82 m. The roof is flat, executed as impassable rooftop terrace.

3. Structural Analysis

In order for the building to be resistant to seismic loads, the entire building must have adequate resistance, ductility and ability of dissipating the energy. Engineering measures for improving seismic resistance of the existing masonry buildings are based on the following:

- Walls must be adequately connected, floor structures should be anchored to the walls, and their stiffness should meet the requirements for distributing seismic loads on the walls.
- Structural walls must be evenly distributed in the two orthogonal directions of the building. There should be enough walls and their strength should be sufficient to enable successful resistance to the expected seismic loads.
- The properties of foundations must enable the increased limit loads occurring due to structural reinforcements to be transferred to the ground.

If we start from the basic analysis of the surface of structural walls and the building's number floors, then, according to EN1998-1, we can conclude that despite the good representation of load-bearing walls (as seen in Table 1), buildings built with ordinary masonry walls are not allowed in seismic areas with calculated ground acceleration of a_g = 0.2 g.

Analytical models and calculations that were performed in the Fedra software package have been developed in four separate models. The first model presents the building's current state; the second model shows the building strengthened with vertical tie beams, in the third model the building was strengthened with reinforced masonry walls, while in the fourth model the structure is strengthened with reinforced concrete jackets. The models were first calculated for vertical load that consists of permanent and useful load. In this way, the correctness of the model was tested, as well as the behaviour of the structure under a vertical load.

The structure's stress state was tested by taking into account the mechanical properties of the built in masonry elements. The compressive strength f_k for solid bricks is 10 N/mm^2, for compo mortar f_m = 2.5 N/mm^2, and for lime mortar f_m = 1.0 N/mm^2. The other input parameters, as shown in Figure 3, were: coefficient of acceleration a_g = 0.25 g, category of structure II, continuous load useful load on slabs 2.5 kN/m^2, on stairs and balconies 4.0 kN/m^2, brand of concrete, reinforcement, ground bearing capacity, etc. Since the existing structure in the process of rehabilitation, except for structural repair, experience a certain form of architectural reconstruction, the stress state was tested with bigger useful load characteristic of repurposing the building (e.g. converting to solitary hotel) and rehabilitating it in public space.

Based on the analysis of results obtained for building A, it was found that in the case of unreinforced masonry all walls on all floors except the third floor (as seen in Table 2) exceed the calculated value of capacity in the case of shear V_{Rd}. Values of capacity of the wall under vertical load N_{Rd}, were not exceeded, which means that the wall has a sufficient capacity for vertical load.

If we want to strengthen the existing state of the building by introducing vertical tie beams at places of confluence and confrontation load-bearing walls, we would get a situation almost identical to the previous, where the calculated values of shear load V_{Ed} exceed the permissible values of shear capacity V_{Rd}. In this case, the capacity of the wall under vertical load is also not exceeded. By installing reinforcement in the existing wall's lime mortar in horizontal and vertical direction as the third strengthening alternative, the results would indicate an improvement in shear capacity of the wall. However, it still does not strengthen the structure to the extent that all the walls are resistant to shear load.

The fourth alternative of strengthening a masonry structures is installing wire mesh reinforcement in the concrete layer (Figure 3) [5]. This strengthening can be performed either on one or both sides of the wall. In our case, the wire mesh consists of Ø10 rods with 10 cm spacing and is anchored in the existing wall. These meshes are then covered with a 10 cm a layer of concrete C25/30. This strengthening was executed on one side of the wall. When installing these meshes the following should be taken into account in particular: they should be with sufficient overlap, they should be installed at an appropriate distance from the wall, and that the anchoring elements should be made of stainless steel.

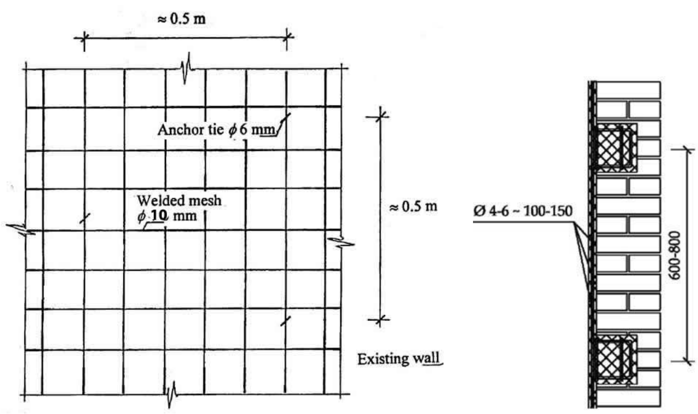


Figure 3. Applying RC jackets to brick masonry wall

As indicated by the results obtained in this analysis, this is a well-strengthened structure, given that the calculated values of shear load V_{Ed} are lower than the permissible values of capacity under shear load V_{Rd}. It should be noted that the analysis of the structure has been carried out with various spacing of reinforcement in the mesh (8 cm – 25cm) while not changing the Ø10 diameter of the reinforcement. However, the most realistic and the best solution have been adopted in case of Ø10 rods with 10 cm spacing. If the reinforced concrete jackets are applied only on the ground floor, then the problem is in excess shear load in walls that could be resolved in the ground floor, while the situation on upper floors remains unchanged.

The analysis results for building "B" being the same as the previous one, we can conclude that in the case of masonry walls usually unreinforced masonry – all the walls on all floors except the third floor, as seen in Table 3, exceed the budget value of capacity at shear V_{Rd}. The values of the capacity of the wall with a vertical load N_{Rd}, are not exceeded, which means that the wall has sufficient load-bearing capacity for vertical load. If the current condition is improved by the introduction of vertical tie columns at places suticanja and confrontation load-bearing walls, we would get almost identical to the previous situation, where the budget values shear loads V_{Ed}, exceed the permissible values of shear capacity V_{Rd}. In this case, the load on the vertical wall of the load is not exceeded.

Table 2. Stress state in walls of buildings "A"

Type of str.		Basement	Ground floor	I story	II story	III story
Unreinforced masonry	Shear	All walls not satisfied	All walls not satisfied	All walls not satisfied	All walls not satisfied	X4, X5, Y1, Y5, Y6, not satisfied
	$V_{Ed} > V_{Rd}$	9 – 435%	33 – 385%	25 – 260%	10 – 175%	3 – 50%
Confined Masonry	Shear	All walls not satisfied except X3	All walls not satisfied	All walls not satisfied except X2	All walls not satisfied except X2	Not satisfied in Y1 & Y6
	$V_{Ed} > V_{Rd}$	13 – 140%	1 – 175%	35 – 160%	12 – 115%	10 – 15%
Reinforcd Masonry	Shear	All walls not satisfied except X3, X5, X6	All walls not satisfied except X2	All walls not satisfied except X2	All walls not satisfied except X1, X2, X3,Y4	✓
	$V_{Ed} > V_{Rd}$	3 – 105%	10 – 125%	8 – 115%	5 – 70%	✓
Unreinforced walls with gunites – RC jackets	Shear	Not satisfied Y5	Not satisfied. Y1, Y5	Not satisfied Y5	✓	✓
	$V_{Ed} > V_{Rd}$	10%	< 10%	10%	✓	✓

V_{Ed} – Design shear force from the analysis for the seismic design situation;
V_{Rd} – Design value of capacity under shear load.
Notice: Compression stresses in all walls are satisfactory for both buildings (A and B).

As a third variant of reinforcement of masonry constructions, we have adopted reinforced masonry wall in lime mortar and thereby get the results that indicate the improvement of capacity of the wall shear. However, as shown in Table 3, it still does not strengthen the structure to the extent that all the walls to be relatively resistant to the shear load may.

The fourth variant strengthening masonry structures is the use of wire mesh reinforcement in the concrete layer, Figure 4. These ribs are placed on one side of the wall. Mesh, in this case, is made up of rods Ø10 to 10 cm and anchored in the existing wall. Upon completion of the setting up of these networks, a layer of concrete C25/30 is carried out in a thickness of 10 cm.

In addition to stress state, values of global deformations were also analyzed for both buildings (A and B) for both X and Y directions as shown in Figures 4 through 7. What can be seen in these Figures is that the behaviour of confined masonry, reinforced masonry and unreinforced masonry is clearly more favourable than that of unreinforced masonry. It can also be seen that the behaviour of building B with regular structure is clearly more favourable than that of building A with irregular structure.

Table 3. Stress state in walls of buildings “B”

Type of struct.		Ground floor	I story	II story	III story
Unreinforced Masonry	Shear	All walls not satisfied (NS)	All walls NS	All walls NS	Only X4, Y2, Y3, Y4, Y6 NS
	$V_{Ed} > V_{Rd}$	60 – 220%	100 – 180%	10 – 110%	8 – 12%
Confined Masonry	Shear	Only X2 satisf.	Only X2 satisfied	Only X2 satisfied	✓
	$V_{Ed} > V_{Rd}$	40 – 90%	40 – 90%	15 – 50%	✓
Reinforced Masonry	Shear	Only X2 satisf.	Only X2 satisf	Only X1, X2, X3, Y1, Y5, satisfied	✓
	$V_{Ed} > V_{Rd}$	17 – 43%	14 – 40%	≤ 20%	✓
Unreinforced walls with gunites – RC jackets	Shear	Y3, Y4, Y6, Not satisfied	✓	✓	✓
	$V_{Ed} > V_{Rd}$	10 – 20%	✓	✓	✓

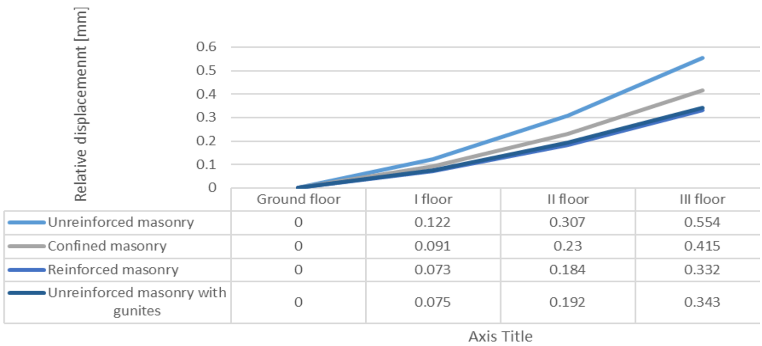


Figure 4. Global displacement of the building model A obtained in X direction

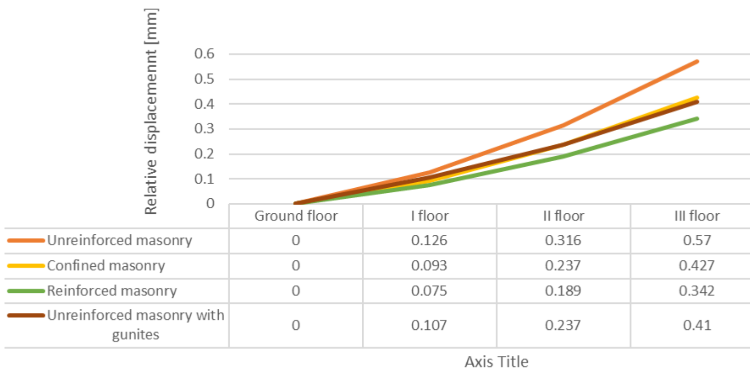
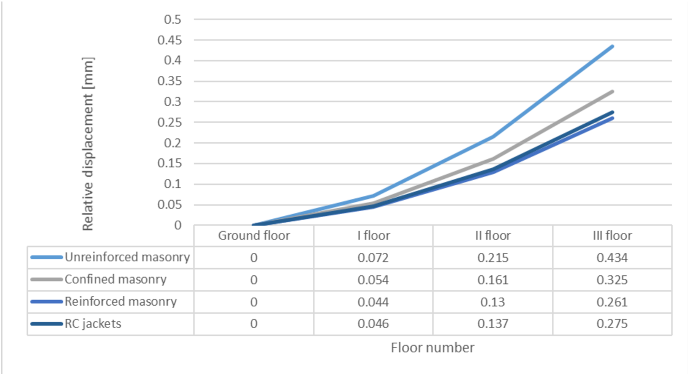
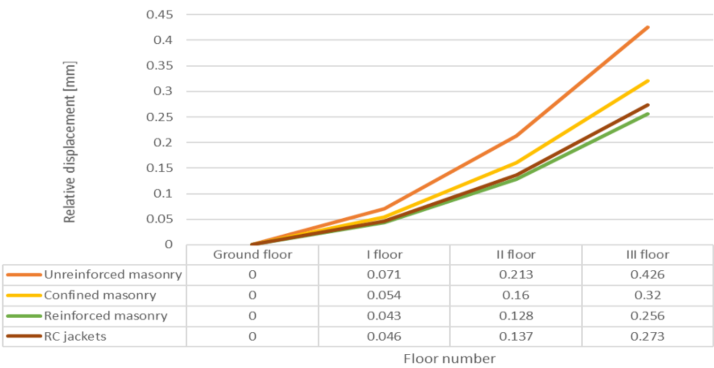


Figure 5. Global displacement of the building model A obtained in Y direction



	Ground floor	I floor	II floor	III floor
Unreinforced masonry	0	0.072	0.215	0.434
Confined masonry	0	0.054	0.161	0.325
Reinforced masonry	0	0.044	0.13	0.261
RC jackets	0	0.046	0.137	0.275

Figure 6. Global displacement of the building model B obtained in X direction



	Ground floor	I floor	II floor	III floor
Unreinforced masonry	0	0.071	0.213	0.426
Confined masonry	0	0.054	0.16	0.32
Reinforced masonry	0	0.043	0.128	0.256
RC jackets	0	0.046	0.137	0.273

Figure 7. Global displacement of the building model B obtained in Y direction

3. Concluding Remarks

Based on the structural analysis of the buildings aided by the Fedra software package, the present paper offers recommendations for seismic strengthening and re-design of old masonry buildings. Seismic resistance can be significantly increased by strengthening the walls. Although the arrangement of load-bearing walls in the existing masonry structures and the quality of materials are limiting factors in the effort to achieve the highest possible level of seismic resistance by implementing measures of strengthening, the conclusion is that the seismic behaviour of buildings strengthened using the above methods can be expected to be acceptable even in the event of severe earthquakes.

Given the fact that during the constructive analysis the adequate seismic resistance has only been obtained with the application of reinforced concrete strengthening, we can conclude that in the conditions of increased useful load the adequate earthquake resistance cannot be easily achieved. This means that changing the purpose of masonry buildings, which implies increasing the load, in these conditions seismic design may be feasible only with a comprehensive analysis of state of the structure and method of strengthening.

The aspects indicated by the European regulations listed in the calculation of damaged buildings are mostly related to the use of linear static analyses and dimensioning reinforced concrete elements. The procedures defining the non-linear analyses of structures are minimized. In comparison to the European regulations, the American regulations FEMA

273 and other [4] consider the assessments of conditions and rehabilitation of structures, which were exposed to earthquake action in detail. However, they fail to take into consideration other accidental actions and relatedness of numerical models before and after sustaining damage. For the purpose of clearer defining and more accurate calculation of such complex problem, the scenario of related non-linear analyses is developed, which should be performed in order to make adequate decisions on the condition of the structure.

Buildings complying with the provisions mentioned above, for which materials of corresponding properties are applied and which fulfil specific requirements regarding number of storeys, geometry of the building and shear walls, and also conditions concerning distribution of stiffness and mass along the height of the building, are classified in EC 8 as "simple masonry buildings". For such structures explicit safety verification is not mandatory.

APENDIX

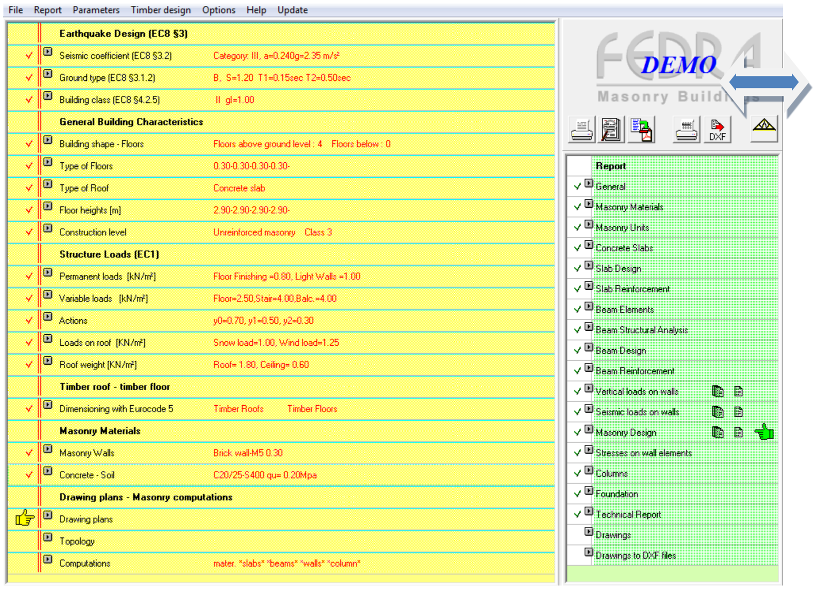


Figure 8. Package Fedra – input parameters for analysis

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