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Seismic analysis of the stone masonry building in the Korčula archipelago

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Abstract

Seismic performance analysis of a stone masonry building dating back to the 15th century, a period in which seismic norms were non-existent, can give us invaluable information about the knowledge and expertise of builders from that time. By analyzing buildings made in previous centuries we find not only numerous aspects from which we can learn, but also many defects which we need to be aware of. Especially when those buildings have become a part of their respective cultural heritage, just like the building observed in this paper. The observed building is a monastery on the island of Badija in the Korčula archipelago in Croatia. Initially, the load-bearing structure was built with massive three-layer stone walls and floors with wooden beams. Throughout the centuries the structure has undergone numerous reconstructions, the last of which involved strengthening with concrete walls and composite floor slab. The numerical model was made and showed that the building does not have sufficient load capacity to seismic load. The linear analysis was made in accordance with Eurocode 8-3 and the load capacity of the structure was determined. Further analysis showed that the critical load-bearing structure was not up to present seismic norms, even concluding that major parts of the building would be in danger of failure. Results show that in case of an earthquake with a return period of 475 years the building would sustain critical damage and possibly failure. The critical parts of the structure and dominant failure mechanisms are determined. Based on the obtained results, retrofit measures of seismic strengthening are proposed.

Key words: stone masonry, seismic analysis, retrofit measures, usability classification

1 Introduction

1.1 Description of the building

The analyzed building is the main part of the Franciscan monastery complex located on the island of Badija in Croatia (Figure 1 and 3). The monastery complex showed on Figure 2, is protected as a cultural monument by the Croatian Ministry of Culture. Originally it was built during the end of the 15th century. It was damaged and rebuilt several times, with the last major renovation in 2012. [1]

Figure 1. Photograph of the building

Figure 2. The monastery complex ("visitkorcula.eu")

The monastery is divided into the western part, which was built earlier, and the newer eastern part (Figure 4). The western part of the monastery has an L-shaped layout with dimensions of 26.0 x 28.7 m, while the eastern part of the monastery has a rectangular layout with dimensions of 50.0 x 10.0 m. The central part of the monastery consists of the ground floor, 1st floor, 2nd floor and attic, while the west and east wings consist of the ground floor, 1st floor and attic. Figure 5 and Figure 6 show the building plan and cross-section from the monastery reconstruction project. The height of the building from the ground level to the rooftop is about 15 m, with the ground type made of A-solid rock.

Figure 3. The location of the building ("Google Maps")

Figure 4. Position of the cadastral parcel ("katastar.hr")

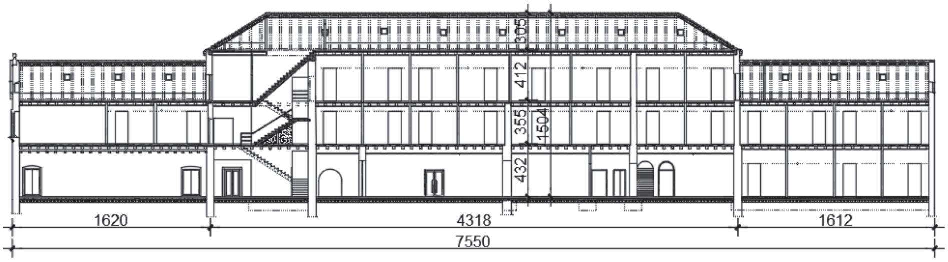

Figure 5. Characteristic building cross-section [1]

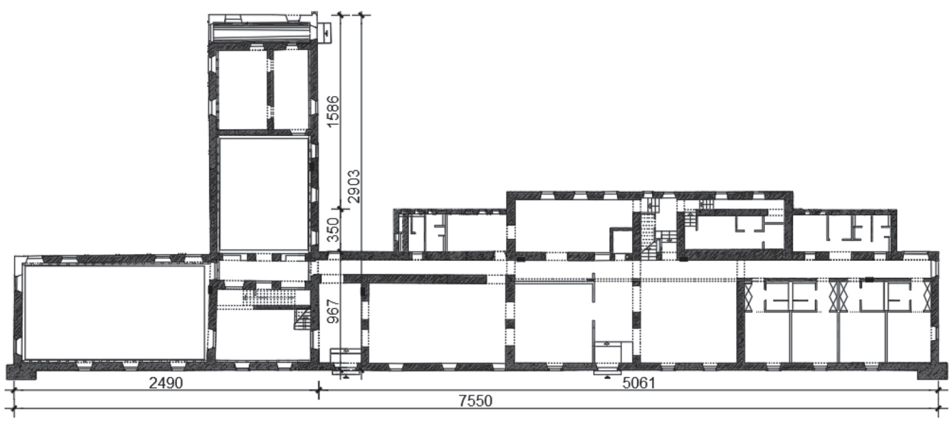

Figure 6. Plan view of the ground floor [1]

The load-bearing system consists of longitudinal and transversal massive three-leaf stone masonry walls, which are distributed along the outer line of the floor plan and in certain parts of the outer borders. The walls are three-leaf masonry walls made of dressed and semi-dressed rock, varying from 40 cm to 87 cm in thickness. The position of the stone walls for the ground, 1st and 2nd floor is presented in Figure 7. The area taken up by the stone walls for the ground floor is 11.5 % in direction X and 6.9 % in direction Y, for the first floor 8.1 % in direction X and 4.9 % in direction Y and for the second floor 8.6 % in direction X and 6.0 % in direction Y. The original floor system was made of wooden beams, except for the northern annex of the building and the terrace, where there was an existent concrete slab of 15 cm thickness. Wooden beams were placed at axial distances varying from 70 to 80 cm, with a size of 20×24 cm in the western-old part and 20×22 cm in the eastern-new part of the monastery.

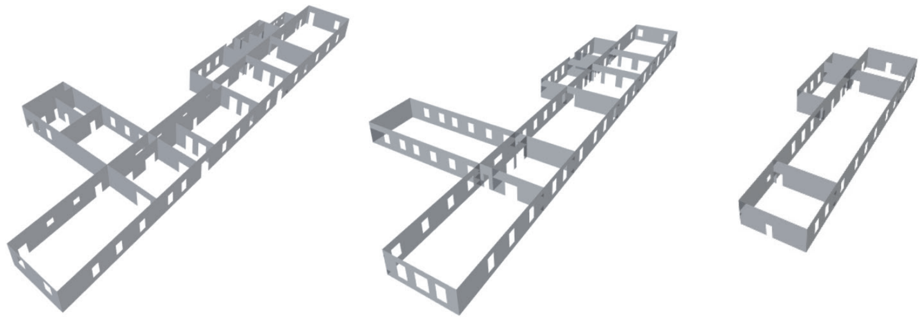

Figure 7. Stone load-bearing walls for the ground floor, first floor and second floor

In the last reconstruction of the year 2012, the building was strengthened using a composite floor slab, with varying thickness depending on which part of the monastery it was used for. For the eastern part of the monastery, a new concrete slab of 6 cm thickness was added on every floor, with the exception of the aforementioned northern annex and the terrace where the old slab was already placed. For the western part of the monastery, a concrete slab of 10 cm thickness was constructed to cover each floor. According to the reconstruction project, reinforced concrete walls were added to support the existing load-bearing system. For the eastern part, 20 cm thick reinforced walls were added for each floor, along with a reinforced concrete beam for the ground floor. The reinforced concrete core for the elevator was constructed in the northern annex. For the eastern part, the new staircase was strengthened with a reinforced concrete wall and a reinforced concrete beam for each floor [1].

The paper presents a seismic analysis of the resistance of the monastery, including the old-western and new-eastern parts of the monastery, the distribution of load-bearing elements by groups depending on the level of usability and the possibility of retrofit measures. The floor system in the numerical model of the building is made in accordance with the renovation project, including the new composite floor slab system, while the vertical load-bearing structure is observed in a state before the renovation, without the reinforced walls. Only massive three-leaf stone masonry walls will be included in the calculation, since the intention is to analyze their behaviour to the seismic load.

2 Numerical model and analysis method

2.1 Numerical model

The numerical model of the building was created in the ETABS software package (CSI 2011) [2], based on the available documentation from the reconstruction of the Franciscan monastery [1]. The material properties adopted in the analysis of stone masonry were in accordance with Italian standards (MIT 2009): the modulus of elasticity E_m = 1200 MPa, shear modulus G_m = 480 MPa, compression strength f_m = 2.0 MPa, initial shear strength f_{v0} = 0.16 MPa, tensile strength f_t = 0.06 MPa and density γ = 20 kN/m^3 [3]. The confidence factor was set to 1.0, even though no testing of the material has been done. Due to the simplification of the model, the walls were divided into several main thicknesses. The shear and flexural stiffness for walls was reduced to 50 % of the initial stiffness of the uncracked elements. The material of which the wooden beams were made is assumed to be holm oak, with a modulus of elasticity E = 14600 MPa and density γ = 500 kg/m^3. All concrete elements were made of MB30 (C25/30), with reinforcement bars made of B500B. [4] The complete numerical model is presented in Figure 8 and Figure 9.

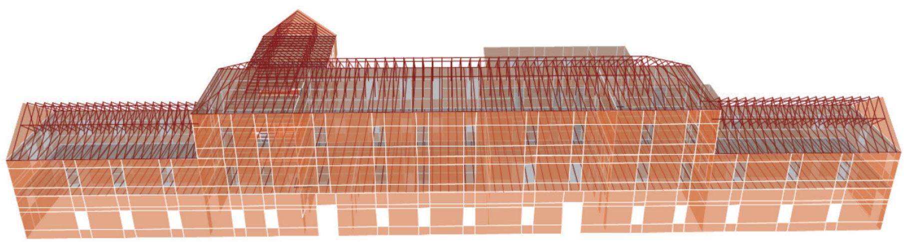

Figure 8. Three-dimensional display of load-bearing structural elements (south facade)

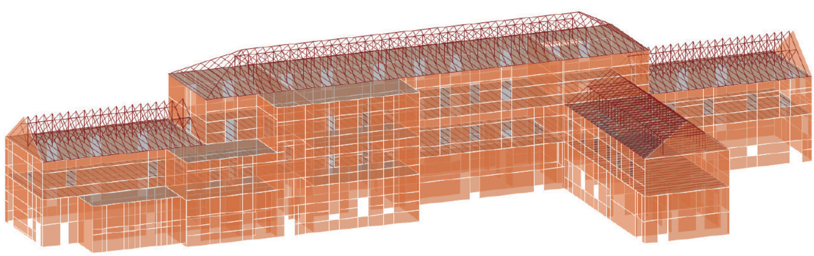

Figure 9. Three-dimensional display of load-bearing structural elements (north facade)

The seismic load was determined according to valid Croatian standards HRN EN-1998, with reference peak ground acceleration on the bedrock of 0.268g for the return period of 475 years and 0.141 g for the return period of 95 years.

2.2 Analysis method

The linear analysis with behaviour factor was made in accordance with HRN EN-1998-3 [5]. The following numerical models were observed:

- Numerical model with behaviour factor q = 1.5 (in further text *Model 1*)
- Numerical model with behaviour factor q = 2.0 (in further text *Model 2*).

The bearing capacity of each pier was calculated according to HRN EN 1998-3. Two types of element failure were observed: shear force failure (1) and bending failure (2), given in the equations below. V_F represents bearing capacity, while V_{Ed} represents acting shear force in seismic combination, in which Weight of the structure, Additional constant load and 30 % Live load were included.

Shear force bearing capacity:

$$V_{Fs} = f_{vd} \cdot t \cdot D' > V_{Ed} \tag{1}$$

Flexural bearing capacity:

$$V_{Fb} = \frac{D \cdot N}{2Ho} \cdot (1-1{,}15\, \nu_d) > V_{Ed}$$

where:
f_{vd} - shear strength
t, D, H - thickness, length and height of the element ("pier")
H_0 - the distance from the inflection point to the critical cross-section
N_{Ed} - axial force from the vertical load
f_d - compressive strength
ν_d - normalized axial load $\nu_d = \frac{N}{D \cdot t \cdot fd}$
D' - compressive length of the element $L_c = 3 \cdot \left[\frac{L}{2} - \frac{M_{Ed}}{N_{Ed}} \right] > 0.$

3 Analysis results

3.1 Dynamic properties

As a result of the elongated form of the building and the position of load-bearing walls, the first modal shape is a translation in direction Y, the second modal shape is a torsion and the third modal shape is a translation in direction X (Figures 10, 11 and 12). The first 5 Modal periods and Modal participating mass ratios of the building are presented in Table 1.

Table 1. Modal periods and Modal Participating Mass Ratios

TABLE: Modal Periods And Modal Participating Mass Ratios				
Case	**Mode**	**Period (sec)**	**Sum UX**	**Sum UY**
Modal	1	0.358	0.0000	0.5778
Modal	2	0.261	0.0006	0.6349
Modal	3	0.234	0.6692	0.6349
Modal	4	0.205	0.6725	0.7357
Modal	5	0.201	0.6731	0.7412

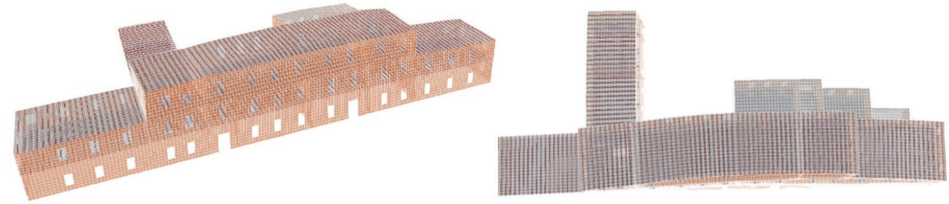

Figure 10. First mode shape with period T = 0,358s

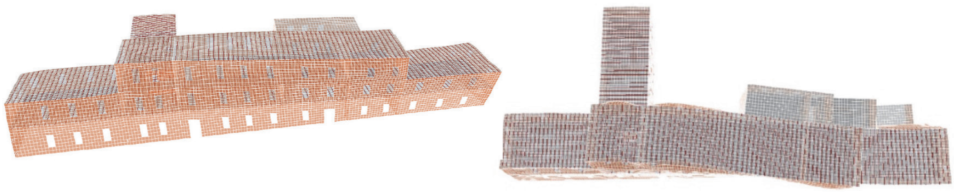

Figure 11. Second mode shape with period T = 0,261s

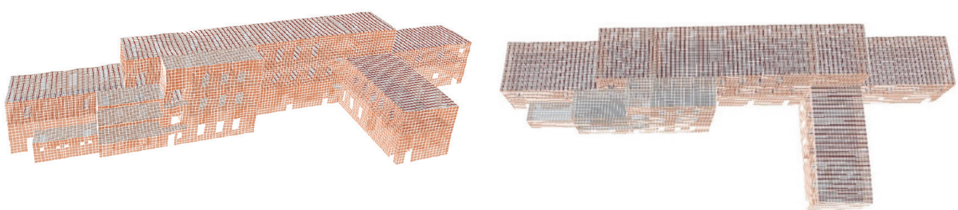

Figure 12. Third mode shape with period T = 0,234s

3.2 Distribution of the shear force

The weight of the building amounts to 49249 kN at the ground level, 26463 kN at the bottom of the first floor and 11427 kN at the bottom of the second floor. The calculated base shear coefficient amounts to 15.2 % for direction X and 13.6 % for direction Y for *Model 1*. For *Model 2*, base shear amounts 11.4 % for direction X and 10.2 % for direction Y. The results for additional floors of *Model 1*, as well as the results for *Model 2* are listed in Table 2. The reference shear force is determined for a return period of 95 years.

Table 2. Shear force and base shear for *Model 1* and *Model 2*

	Model 1 with behaviour factor q = 1.5			**Model 2 with behaviour factor q = 2**		
	Direction	Shear force [kN]	Base shear [%]	Direction	Shear force [kN]	Base shear [%]
GROUND FLOOR	X	7498.1	15.2	X	5624.6	11.4
	Y	6683.8	13.6	Y	5013.0	10.2
FIRST FLOOR	X	5983.7	22.6	X	4487.8	17.0
	Y	5318.3	20.1	Y	3988.7	15.1
SECOND FLOOR	X	3483.3	30.5	X	261.4	22.9
	Y	3196.9	28.0	Y	2397.7	21.0

3.3 Classification into groups

Two types of wall failure were observed: shear force failure and bending failure, the formulas and requirements for both are given in section 2.2. The walls in the numerical model were divided into 231 elements. The bearing capacity of each wall element ("pier") was calculated according to HRN EN-1998-3. Elements for which the compressive length requirement was not satisfied were excluded from the calculation, with the remaining 201 elements for *Model 1* and 207 elements for *Model 2*. The walls were categorized into 4 groups depending on the ratio of the acting force and capacity of the wall; Group 1 (*green colour*): less than 100 % ratio level, Group 2 (*yellow colour*): 100 % - 150 % ratio level, Group 3 (*orange colour*): 150 % - 200 % ratio level and Group 4 (*red colour*): more than 200 % ratio level. The categorization of groups by colour is shown in Figure 13 for *Model 1* and Figure 14 for *Model 2*.

The following results are presented for *Model 1* and *Model 2*, with a return period of 95 years for the control of the significant damage (HRN EN-1998-3). For *Model 1,* the calculations are satisfied by only 62 % of the walls. Most of the walls in the direction X were categorized in Group 1, while a large number of walls laid in the direction Y were categorized in Group 3 and Group 4. For *Model 2*, the number of the walls in Group 1 significantly increases, leading to 82 % of the walls being categorized as Group 1 (Table 3).

Table 3. Group categorization for Model 1 and Model 2

	Model 1 with behaviour factor q = 1.5		Model 2 with behaviour factor q = 2	
	Number of elements in the group	**Number of elements in the group [%]**	**Number of elements in the group**	**Number of elements in the group [%]**
Group 1	125	**62.19**	170	**82.13**
Group 2	21	**10.45**	20	**9.66**
Group 3	25	**12.44**	10	**4.83**
Group 4	30	**14.93**	7	**3.38**
Total	201	100	207	100

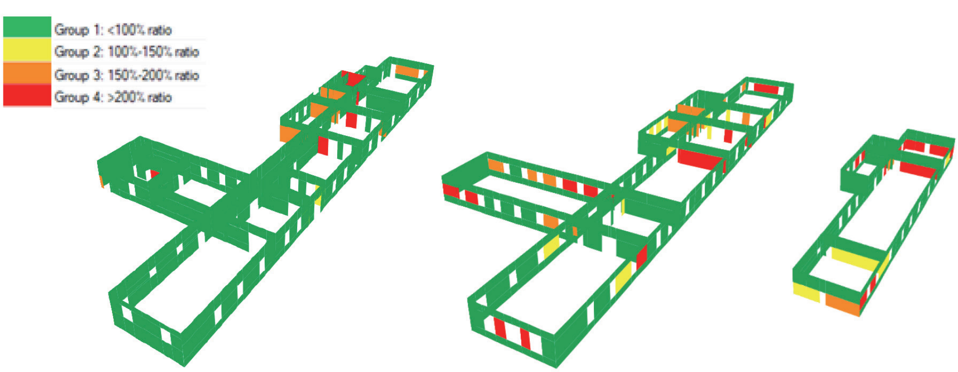

Figure 13. Distribution of elements by groups for *Model 1* (Ground floor, First floor and Second floor)

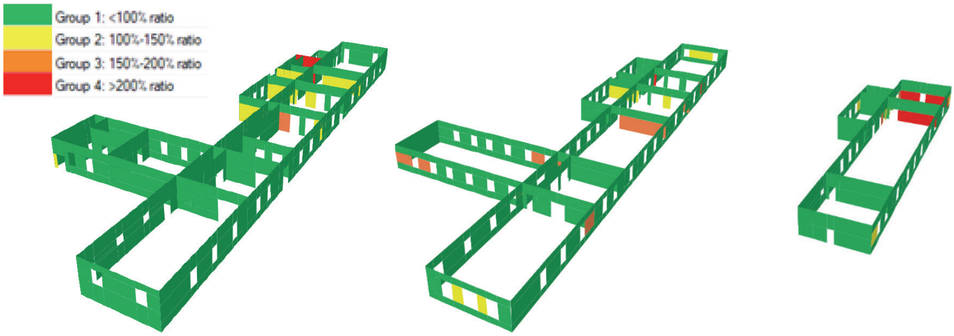

Figure 14. Distribution of elements by groups for *Model 2* (Ground floor, First floor and Second floor)

3.4 Retrofit measures

A recommended measure in improving the resistance to seismic load is strengthening the building with reinforced concrete walls. For *Model 1*, with a return period of 95 years, longitudinal and transversal reinforced concrete walls were added to the model (Figure 15). This resulted in increasing the stiffness of the building and leading to an 87 % increase of the wall elements which have not failed under seismic load. The results are shown in Table 4. for *Model 1*.
The critical three-leaf stone walls categorized in Group 3 and 4 are mostly short walls of great slenderness with no explicit pressure zone. They do not affect the global carrying capacity of the structure, even though they do not have enough capacity to withstand the bending collapse. Those walls can be locally strengthened by a very effective method-grout injection. This method significantly improves the load-bearing capacity of the stone walls [6].

Tablica 4. Categorization for Model 1 with reinforced concrete walls

Model 1 with reinforced concrete walls	Number of elements in the group	Number of elements in the group [%]
Group 1	167	**86.98**
Group 2	9	**4.69**
Group 3	8	**4.17**
Group 4	8	**4.17**
Total	192	100

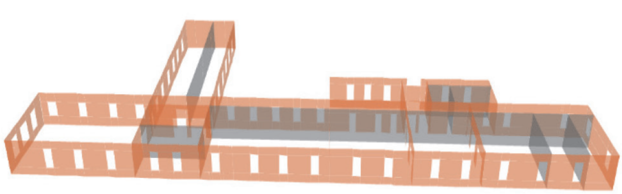

Figure 15. The first floor of the model with reinforced concrete walls

4 Conclusion

Based on the seismic analysis results, the conclusions of the building behaviour under seismic load may be presented. It should be taken into account that many of the material and structural properties were assumed, which enlarged the differences between the numerical model and the existing building. This contributes to the imprecisions of the numerical model which should be considered when interpreting the analysis results. The main problem with the concept of the building is in its elongated form, as well as the L shape of the western part, which resulted in an uneven load distribution. The distribution of wall elements into colour groups, based on the level of satisfaction ratio, gives an understandable and clear view of the critical parts of the building. For this building, most of the critical walls were placed in direction Y and the detached parts of the building. Based on the group categorization and base shear level, the critical floors were the 1st floor and 2nd floor.

Two models, *Model 1* with behaviour factor 1.5 and *Model 2* with behaviour factor 2.0, were observed and compared for a better understanding of the structure behaviour to seismic load. The massive three-layer stone walls, as the vertical load-bearing structure, were categorized into 4 groups depending on the ratio of the acting force and capacity of the wall; Group 1: less than 100 % ratio level, Group 2: 100 % - 150 % ratio level, Group 3: 150 % - 200 % ratio level and Group 4: more than 200 % ratio level. For *Model 1*, a significant amount of the stone walls was categorized into Group 3 and Group 4, generally placed in the problematic parts of the building. For *Model 2*, there is a significant increase in the load-bearing capacity of the elements, which was expected due to the higher behaviour factor assumed for this numerical model. Although the results for *Model 2* were improved, we have to take into account that the behaviour factor q = 2 should be assured and confirmed for the results of this model to be relevant.

In conclusion, two different models were made to show the differences in their behaviour and resistance to seismic load. The numerical model with the higher behaviour factor confirmed the expected higher load-bearing capacity of the structure than *Model 1* with behaviour factor q = 1.5. For *Model 1*, retrofit measures are suggested in this paper. By strengthening the building with reinforced concrete walls and grout injecting, there is a significant increase in the stiffness of the structure and the number of elements that have sufficient load-bearing capacity to seismic load. However, considering the significant seismic activity of the region in which the building is located, the best solution would be the reconstruction of the entire building form to increase the load-bearing capacity of the structure.

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