Structural Analysis of Historical Constructions, New Delhi 2006
P.B. Lourenço, P. Roca, C. Modena, S. Agrawal (Eds.)

Seismic Response of Heritage Stone-Masonry Buildings

Miha Tomaževič and Marjana Lutman
Slovenian National Building and Civil Engineering Institute, Dimičeva Ljubljana, Slovenia

ABSTRACT: The modelling of sAuthors of papers have to type these in a form suitable for direct reproduction by the publisher. In order to ensure uniform style throughout the volume, all the papers have to be prepared strictly according to the instructions set below. A laser printer should be used to print the text. The manuscript will be reduced to 85% by the publisher and will be printed in black only.eismic response of heritage stone-masonry buildings is discussed. Since the shear mechanism prevails in the behaviour of unreinforced historical buildings, simple multi-degree-of-freedom system with concentrated masses and storey stiffnesses provides good results if adequate input parameters and hysteretic rules are used in the calculations. It has been shown that up-to-date code requirements regarding the resistance demand are somewhat conservative and difficult to attain. On the basis of earthquake damage observations and subsequent analysis, a reduction of code required values has been proposed without risking the collapse but slightly increasing the expected level of damage to heritage buildings.

1 INTRODUCTION

Considering the response to earthquakes, the behaviour of traditionally built stone-masonry houses in historical urban and rural settlements was generally not adequate and resulted into heavy damage and collapse, . Most of the damage and, consequently, loss of life during recent earthquakes was the consequence of heavy damage and collapse of such buildings. Therefore,In order to improve the situation and preserve the heritage buildings, a substantial amount of experimental research to study the behaviour and verify develop strengthening measures for strengthening developed on the basis of earthquake damage observations, has been carried out in the past several decades. On the basisAs a result of investigations, numerical models have been also developed, which simulate the seismic behaviour and make possible the seismic resistance verification of heritage buildings in the process of redesign.

Traditional construction material of heritage urban and rural stone-masonry buildings is locally available lime-stone and slate. Stone masonryThe walls are made of rubble or river-bed stone, built in two outer layers of irregularly sized bigger stones, with an inner infill of smaller pieces of stone, in poor mud mortar with a little lime. In the city centers and towns, the walls are made of relatively compact mix of stone, brick and mortar, with no distinct separation between the individual layers of the walls. Regularly cut, or partly cut stone is rarely used. Connecting stones are also rare.

Typically, stone-masonry houses are 3 - 4 stories high in the cities and towns, whereas their height is limited to 2 stories in rural areas. Structural layout is usually adequate. The distribution of walls is uniform in both orthogonal directions, and, because of the thickness of load-bearing and cross-walls, as well as relatively small rooms, the wall/floor area ratio is very large, in many cases exceeding 10 %. Floor structures and lintels are traditionally wooden, without any wall-ties provided to connect the walls. Wooden floors are sometimes replaced by brick vaults above cellars, staircases and corridorsBrick vaults above cellars, staircases and corridors sometimes replace wooden floors. Roof structures are wooden. They are covered with ceramic tiles, sometimes laid in mortar. As a rule, the buildings are built without any foundation, and foundation walls are of poorer quality than the walls of the structure above the ground level.

Since the typical causes of inadequate seismic behaviour of heritage buildings are lack of integrity and low resistance capacity of stone-masonry walls, the tying of the walls with steel ties and injecting the walls with cementitious grouts have been the most widely used methods for the retrofitting in the last few decades. The efficiency of these methods has been verified by laboratory testing. Moreover, there have been also cases where the behaviour of retrofitted buildings has been verified by subsequent have been subjected to another strong earthquake, so that the efficiency has been verified even in the real situation.. In this contribution, some results of recent research in seismic behaviour of heritage buildings, carried out at Slovenian National Building and Civil Engineering Institute in Ljubljana, Slovenia, will be discussed.

2 BEHAVIOUR MECHANISM AND MODELS

Because of mechanical properties of masonry materials (low shear modulus G / modulus of elasticity E ratio), structural configuration and geometrical characteristics of masonry walls, shear mechanism determines the response of heritage buildings to earthquakes in the case of adequate structural integrity where the wall ties and/or floor structures prevent the separation of walls during vibration. Because of low tensile strength f_t / compressive strength f ratio, shear failure typically occurs in structural wallywalls, despite their geometry shape aspect (height / length) ratio (Figs. 1 and 2). Except for very slender elements, flexural failure mode is not observed.

Figure 1 : Typical shear cracks in short window piers

Figure 2 : Shear cracks occur alsofailure of in long walls

Shear failure takes place where the principal tensile stresses, developed in the wall under a combination of vertical and horizontal loads, exceed the tensile strength of masonry materials. Characteristic diagonal cracks develop in the wall just before the attainment of maximum resistance. For the case of the shear failure, the resistance of the walls $H_{s,w}$ is calculated by (Turnšek and Čačovič 1971):

$$H_{s,w} = A_w \frac{f_t}{b} \sqrt{\frac{\sigma_o}{f_t} + 1}, \tag{1}$$

where:
$H_{s,w}$ = shear resistance of the wall,
A_w = area of the horizontal cross section of the wall,

σ_o = compressive stress in the wall, f_t = tensile strength of the masonry,
b = coefficient of distribution of shear stresses.
By assuming the storey mechanism, effective lateral stiffness of the wall K_e is calculated by:

$$K_e = \frac{G\,A_w}{1.2\,h\left[1+\alpha'\dfrac{G}{E}\left(\dfrac{h}{l}\right)^2\right]}, \tag{2}$$

where α' = coefficient determining the position of the bending moment's inflection point along the height of the wall. α' = 0,83 in the case of a fixed-ended and α' = 3,33 in the case of a cantilever wall.

Although stone-masonry is considered as brittle structural material, stone-masonry walls possess substantial displacement and energy dissipation capacity. They are capable to carry gravity loads even after being damaged by a strong earthquake when responding in the non-linear range of vibration. Therefore, when modelling the seismic behaviour of heritage buildings, ductility capacity is attributed to masonry walls. The seismic loads are redistributed, and, consequently, the walls' lateral load bearing capacity is fully utilized.

In a general way, this is explained in Figure 3, where typical relationship between the lateral resistance of a critical storey and displacements, expressed in the non-dimensional form of storey rotation, i.e. storey drift d / storey height h ratio $\Phi = d/h$, is schematically presented, determined by characteristic limit states. Before the attainment of elastic limit (rotation $\Phi_e = d_e/h$) the structure is not damaged. The first cracks which change the stiffness but do not influence the usability of the building occur in the range between the elastic limit and maximum resistance limit state, i.e. where the resistance attains the maximum value (in the range between Φ_e and $\Phi_{Hmax} = d_{Hmax}/h$), whereas the structure is still safe until the design ultimate state is attained (rotation $\Phi_u = d_u/h$). Although the experimental investigations indicate that the structure enters the actual state of collapse much after the displacements exceed the design ultimate state, the displacement capacity beyond this point is not taken into consideration because of the heavy damage which occurs to structural walls. To follow the damage limitation requirement, it is assumed that rotation Φ_u, where the resistance degrades to 80 % of the maximum, defines the design ultimate state of collapse. In-situ and laboratory tests of existing and cement-grouted stone masonry walls yielded the following ranges of values of rotation at limit states: Φ_e = 0,2 - 0,3 %, Φ_{Hmax} = 0,4 - 0,5 %, and Φ_u =1,0 - 1,2 % (Klemenc et al. 2000, Tomaževič et al. 1989). The values are slightly higher in the case of original walls than in the case of cement grouted, more rigid walls.

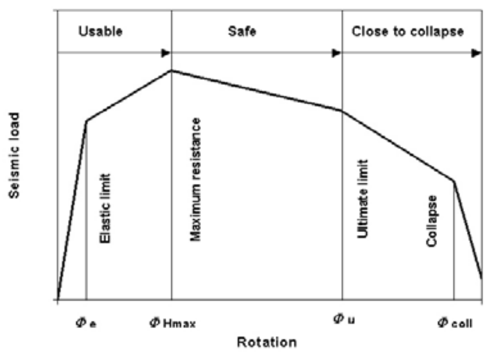

Figure 3 : Idealized story resistance envelope and definition of limit states

The resistance envelope of the critical story is obtained by superposition of resistance envelopes of individual walls which compose that story. It is assumed that individual walls resist the imposed displacement up to the attainment of their ductility capacity. Ultimately, it is also assumed that they carry the vertical loads although they fail for lateral loads. A push-over type method has been developed for the calculation of story resistance envelope. In the calculation, the displacements are imposed to the structure and the resisting forces of structural elements are calculated. The stiffness and resistance of individual walls in each step of calculation are determined considering the imposed storey displacements and idealised resistance envelopes of the walls. In such a way, the degrading branch of the resistance envelope can be also calculated. Computer program SREMB (Seismic REsistance of Masonry Buildings, in the first version known as POR) has been developed to make easier the calculation of the storey resistance envelope (Tomaževič et al. 1978, Tomaževič 1997).

It has been found in the previous studies, that in the case of the storey mechanism with predominant first natural mode of vibration, the non-linear seismic response of unreinforced heritage masonry structures can be adequately modelled with simple planar multi degree of freedom shear systems, with masses concentrated at floor levels, and storey stiffnesses (Tomaževič 1987). Storey resistance envelope is taken into account as a skeleton curve, whereas hysteretic rules are modelled by taking into consideration the measured relationships between lateral forces and displacements. They are modelled by a set of rules, which take into consideration strength and stiffness degradation and deterioration at repeated lateral load reversals. The parameters, which define the rules and shape of the hysteresis, have been developed on the basis of a substantial number of tests of masonry walls subjected to different static and dynamic load time histories (Tomaževič and Lutman 1996). An example of actual hysteretic lateral load-displacement relationships, obtained by testing a stone-masonry wall, is presented in Figure 4, whereas the hysteretic rules are schematically presented in Figure 5. Computer program DASS (Dynamic Analysis of Shear Systems) has been developed to calculate the non-linear response of simple planar shear systems with concentrated masses and storey stiffnesses.

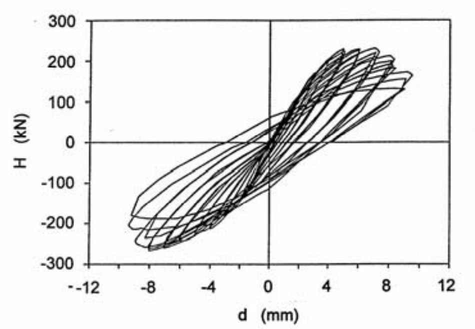

Figure 4 : Typical lateral load - displacement hysteresis loops, obtained by testing a stone-masonry wall

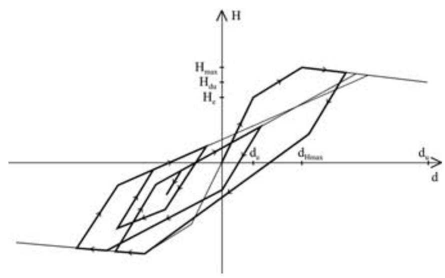

Figure 5 : Schematic presentation of hysteretic rules considered in the calculation of the non-linear seismic response (Tomaževič and Lutman 1996)

3 SEISMIC RESISTANCE: CODE DEMAND AND CAPACITY

In the verification of seismic resistance, the design resistance of the structure (capacity) in terms of ultimate design seismic resistance coefficient $SRC_{\mathrm{d,u}}$ (ultimate design resistance / weight of the building rato) is compared with the design seismic load (demand) given in terms of the ultimate design base shear coefficient $BSC_{\mathrm{d,u}}$ (ultimate design seismic load / weight of the building ratio). The basic condition:

$$SRC_{\mathrm{d,u}} \geq BSC_{\mathrm{d,u}}, \tag{3}$$

should be fulfilled.

As a rule, the same level of design seismic loads should be considered in the redesign of existing buildings as in the case of the new construction. According to Eurocode 8: Design of structures for earthquake resistance (EC 8, Eurocode 2004) the design seismic load in terms of the design ultimate base shear coefficient $BSC_{\mathrm{d,u}}$, is determined by:

$$BSC_{\mathrm{d,u}} = \frac{a_{\mathrm{g}}\, S\, \eta\, 2{,}5}{q}, \tag{4}$$

where a_g = the design ground acceleration at the site, S = the soil parameter, η = the damping correction factor, and q = the structural behavior factor (q = 1,5 for unreinforced masonry structures). The design values of $BSC_{d,u}$ requested by EC 8 for plain masonry buildings are high already for the case of the firm soil (Table 1). They become even higher for other types of soil.

However, there were cases observed where even stone-masonry houses resisted the earthquakes with little damage, although the values of $SRC_{d,u}$, calculated on the basis of experimentally obtained mechanical properties of masonry, were lower than $BSC_{d,u}$ values required for a given location or values resulting from acceleration response spectra calculated for the earthquake which affected the buildings. It has to be noted at this time that the values significantly higher than 0,3 are difficult to obtain with commonly used methods of strengthening. This justifies the idea that the design ground acceleration values may be reduced either in the case where the anticipated total costs of strengthening the entire building inventory of particular urban areas would sharply increase if a_g values would be raised towards the code required level, or if code required a_g values for redesign of a heritage building would lead to completely unacceptable architectural alterations.

On the basis of studies carried out in Ljubljana it has been proposed that for practical redesign of heritage buildings a reduction factor γ_n = 0,67 is considered in high seismic intensity zones (a_g > 0,20 g), and no reduction, i.e. γ_n = 1,00, is used in the low intensity zones (Tomaževič 2000). The resulting proposed reduced values of the ultimate design base shear coefficient $BSC_{d,ur}$ to be considered in the redesign of heritage buildings are given in Table 1.

Table 1: Code ($BSC_{d,u}$) and proposed reduced values of design ultimate base shear coefficient ($BSC_{d,ur}$) for heritage masonry structures (q = 1,5) on firm soil (S = 1,0) and different expected seismic intensities

Intensity	Low I	Low II	Moderate	High
a_g	0,05	0,10	0,20	0,30
$BSC_{d,u}$	0,08	0,17	0,33	0,50
γ_n	1,00	1,00	0,84	0,67
$BSC_{d,ur}$	0,08	0,17	0,25	0,33

Table 2 : Seismic resistance of existing and strengthened heritage buildings in terms of seismic resistance coefficient for both orthogonal directions of buildings ($SRC_u = H_u/W$)

Bldg. no.	No. of stories	Wall/floor area (%)		Existing			Strengthened		
		x-dir.	y-dir.	f_t (MPa)	SRC_{ux}	SRC_{uy}	f_t (MPa)	SRC_{ux}	SRC_{uy}
1	2	10,9	6,4	0,08	0,20	0,15	0,14	0,27	0,22
2	2	12,0	9,1	0,08	0,21	0,19	0,14	0,25	0,25
3	2	6,9	8,6	0,06	0,22	0,25	0,11	0,25	0,33
4	2	12,1	11,1	0,06	0,33	0,31	0,11	0,42	0,38
5	2	4,7	14,6	0,06	0,17	0,33	0,11	0,19	0,47
6	2	7,2	14,3	0,06	0,16	0,31	0,11	0,21	0,47
7	2	15,1	13,7	0,06	0,29	0,25	0,11	0,40	0,33
8	2	10,5	9,5	0,06	0,31	0,25	0,11	0,39	0,29
9	2	10,5	9,9	0,06	0,23	0,26	0,11	0,31	0,34
10	2	10,3	10,2	0,06	0,22	0,26	0,11	0,28	0,35
11	2	11,9	10,3	0,06	0,28	0,29	0,11	0,29	0,34
12	2	9,8	10,9	0,06	0,23	0,26	0,11	0,32	0,34
13	2	8,8	8,33	0,06	0,23	0,27	0,11	0,31	0,33
14	2	10,6	12,0	0,06	0,28	0,28	0,11	0,35	0,36
15	2	9,7	12,0	0,06	0,27	0,34	0,11	0,34	0,47
16	2	7,9	4,2	0,06	0,26	0,19	0,11	0,35	0,21

The proposal has been verified by analyzing the seismic resistance of a series of traditionally built stone-masonry houses in the region of Posočje (Soča River Valley), the western-most part of Slovenia. Using the values of mechanical properties of typical masonry in the area in the existing and strengthened, cement-grouted states, determined by in-situ tests (Klemenc et al. 2000, Tomaževič et al. 2000) as the input data, the seismic resistance of a number of typical buildings has been assessed. According to the seismic hazard map of Slovenia, design acceleration value a_g = 0,225 g should be considered in the area for assessing the demand, resulting in $BSC_{d,u}$ = 0,375. If this value is reduced as proposed, the seismic resistance of heritage buildings in the area in terms of seismic resistance coefficient should be at least $SRC_{d,ur}$ = 0,315 in the case that the buildings are located on the firm soil. As can be seen in Table 2, where the results of calculations are summarized, the buildings in the existing state are not resisting enough. If adequately strengthened, however, they will in most cases fulfill the requirements. As can be seen, some will be excessively damaged, but will survive the expected earthquake without collapse. It is to underline, however, that the assumption of uniform distribution of seismic loads onto structural walls, i.e. rigid floor diaphragm action, has been taken into account when evaluating the resistance of buildings.

4 SEISMIC RESPONSE: THE CASE OF BOVEC

It is a rare case that the same region suffered from three strong earthquakes in less than thirty years time period, as was . This is the case of Posočje (Soča River Valley), a mountainous region of Slovenia in the western part of Slovenia along the border with Italy, which in 1976 suffered from a series of earthquakes with epicenters in nearby Friuli, and then again in 1998 and 2004 from two local earthquakes with epicenters near the town of Bovec (Fig. 9). Although the earthquakes of 1998 and 2004 cannot be compared with Friuli earthquakes of 1976 by magnitudes, their maximum local intensities, estimated to VIII on the EMS seismic intensity scale were similar. There have been earthquake acceleration records obtained of the main shocks of September 1976 and July 2004 earthquakes, but no records exist of the earthquake of April 1998. Peak ground acceleration values of 0,53 g and 0,47 g have been measured in 1976 (in Breginj) and 2004 (in Bovec), respectively, though not on the same location. The N-S component of the acceleration record of July, 2004 earthquake and response spectra are presented in Figure Figures 6 and 7, respectively10 (Fajfar et al. 2004).

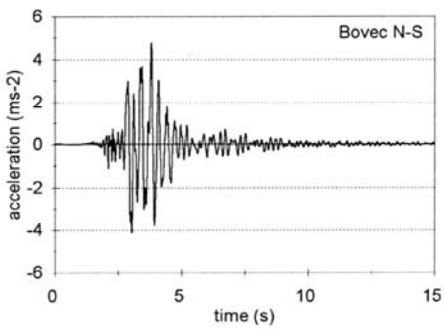

Figure 6 : N-S component of the Bovec ground acceleration record of July 2004 earthquake

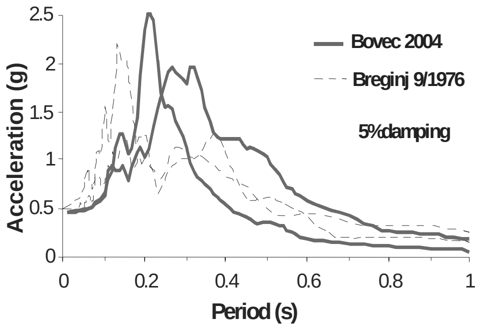

Figure 7 : Acceleration response spectra of earthquakes of 1976 (Breginj) and 2004 (Bovec; by courtesy of P.Fajfar)

Because of the mechanisms, sources and magnitudes of these earthquakes, not always the same part of the region suffered. Whereas the whole area of upper Posočje suffered in 1976, the village of Drežniške Ravne and environment suffered in 1998, and the village of Čezsoča near Bovec suffered in 2004.

Although not many buildings in the recently affected areas have been severely damaged in 1976 and thoroughly strengthened afterwards, their number is large enough to provide a good basis for the analysis and verification of the effectiveness of strengthening measures, recommended in 1976. Their number is also large enough to verify the recommendations for redesign of existing buildings specified in recent Eurocodes. Since the third earthquake in only thirty years caused many concerns and loss of confidence in aseismic strengthening measures applied to buildings after 1976 and 1998, only the analysis of this kind provides an acceptable answer should be given to these concernsmany questions.

As the first step of analysis, the mechanical properties of typical masonry of residential and public heritage buildings in the area in the existing and strengthened, cement-grouted state washave been determined by in-situ tests (Klemenc et al. 2000, Tomaževič et al. 2000). Using the obtained values as the input data, the seismic resistance of a number of typical buildings has been assessed. According to the seismic hazard map of Slovenia, design acceleration value a_g = 0,225 g should be considered in the area, resulting in $BSC_{d,u}$ = 0,375. If this value is reduced as proposed, the seismic resistance of heritage buildings in the area in terms of seismic resistance coefficient should be at least $SRC_{d,ur}$ = 0,315 in the case that the buildings are located on the firm soil. As can be seen in Table 2, where the results of calculations are summarized, the buildings in the existing state are not resisting enough. If adequately strengthened, however, they will in most cases fulfill the requirements. Some, however, will be excessively damaged, but will survive the expected earthquake without collapse.

Table 2 : Seismic resistance of existing and strengthened heritage buildings in terms of seismic resistance coefficient ($SRC_u = H_u/W$)

Bldg. no.	No. of stories	Wall/floor area (%)		Existing			Strengthened		
		x-dir.	y-dir.	f_t (MPa)	SRC_{ux}	SRC_{uy}	f_t (MPa)	SRC_{ux}	SRC_{uy}
1	2	10,9	6,4	0,08	0,20	0,15	0,14	0,27	0,22
2	2	12,0	9,1	0,08	0,21	0,19	0,14	0,25	0,25
3	2	6,9	8,6	0,06	0,22	0,25	0,11	0,25	0,33
4	2	12,1	11,1	0,06	0,33	0,31	0,11	0,42	0,38
5	2	4,7	14,6	0,06	0,17	0,33	0,11	0,19	0,47
6	2	7,2	14,3	0,06	0,16	0,31	0,11	0,21	0,47
7	2	15,1	13,7	0,06	0,29	0,25	0,11	0,40	0,33
8	2	10,5	9,5	0,06	0,31	0,25	0,11	0,39	0,29
9	2	10,5	9,9	0,06	0,23	0,26	0,11	0,31	0,34
10	2	10,3	10,2	0,06	0,22	0,26	0,11	0,28	0,35
11	2	11,9	10,3	0,06	0,28	0,29	0,11	0,29	0,34
12	2	9,8	10,9	0,06	0,23	0,26	0,11	0,32	0,34
13	2	8,8	8,33	0,06	0,23	0,27	0,11	0,31	0,33
14	2	10,6	12,0	0,06	0,28	0,28	0,11	0,35	0,36
15	2	9,7	12,0	0,06	0,27	0,34	0,11	0,34	0,47
16	2	7,9	4,2	0,06	0,26	0,19	0,11	0,35	0,21

Note: f_t = characteristic tensile strength of masonry.

TakingUsing advantage of ground acceleration records obtained in 2004, four typical stone masonry houses, strengthened after 1998 but damaged again in 2004, located near enough to the site where the records have been obtained, have been analyzed (Tomaževič et al. 2005). Although a simple, equivalent single-degree-of-freedom model has been used, the analysis provided a realistic simulation of what happened on July 12, 2004. Three buildings (the representative building A is only shown here) have been strengthened by tying the walls with steel ties and cement-grouting of stone-masonry walls, whereas in the case of the fourth one (building D), a new story built in confined clay hollow block masonry with new r.c. slabs has been placed on the existing, not strengthened stone masonry walls in the ground floor. Typical plans and sections of two of the analyzed buildings are shown in Figure 118. Although a simple, previously described simple multi-degree-of-freedom shear model has been used, the analysis provided a realistic simulation of what happened on July 12, 2004.

Taking into consideration the structural layout and seismic response mechanism of typical stone masonry buildings in the region, the buildings have been modeled as simple planar multi degree of freedom shear systems, with masses concentrated at floor levels and story stiffnesses. Hysteretic behavior has been modeled by a set of rules, which take into consideration strength and stiffness degradation and deterioration at repeated load reversals (Tomaževič and Lutman 1996). Typical results, such as displacement response time history of the top floor of the buildings and story shear-rotation hysteresis loops are presented in Fig. 12.

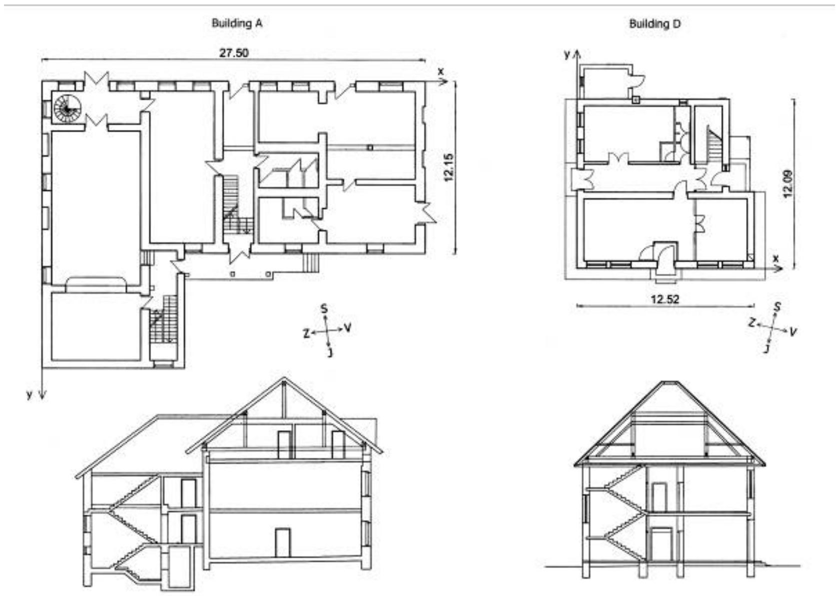

Figure 8 : Plan and section of typical analyzed buildings A and D (Tomaževič et al. 2005)

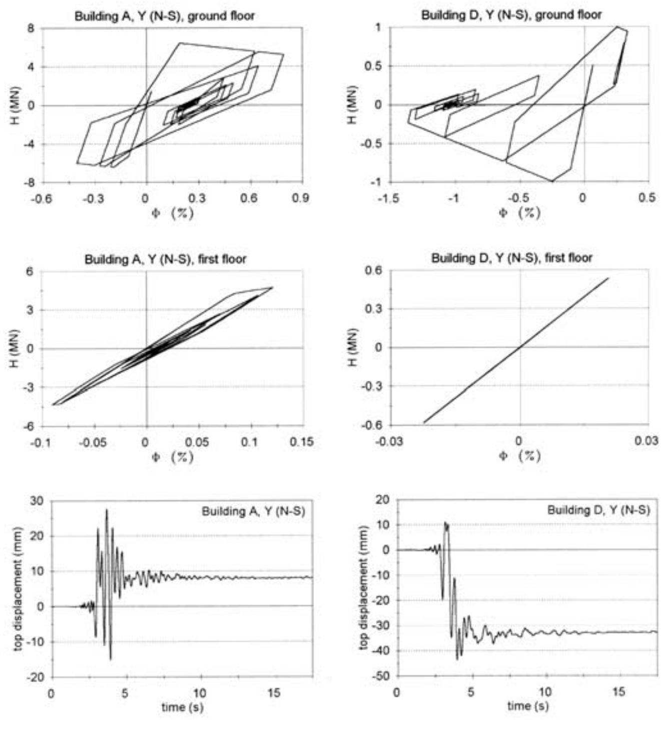

Figure 9 : Hysteretic response of the first and second floor and displacement response time history of the top of the building (Tomaževič et al. 2005)

Typical results, such as displacement response time histories of the top floor of the buildings and story shear - rotation hysteresis loops are presented in Figure 9. Non-linear character of the response of the analysed buildings can be clearly seen. The response analysis confirmed the conclusions based on the response spectra that the occurrence of damage to structural walls during an earthquake as strong as the earthquake of July 12, 2004, could have been expected, although the buildings had been previously strengthened. However, by comparing the calculated response values of maximum story rotation with typical values at characteristic limit states (Fig. 3), it can be seen that the amount of damage did not exceed the code expected level in the cases where the buildings have been adequately strengthened. The response analysis has also shown that the occurrence of cracks in both ground floor and first storey, as has been actually observed after the earthquake, has been confirmed also by calculation. Large displacement response is in good correlation with the excessive amount of damage in the case of inadequately strengthened building D. This all confirms the validity of this, though very simple, model.

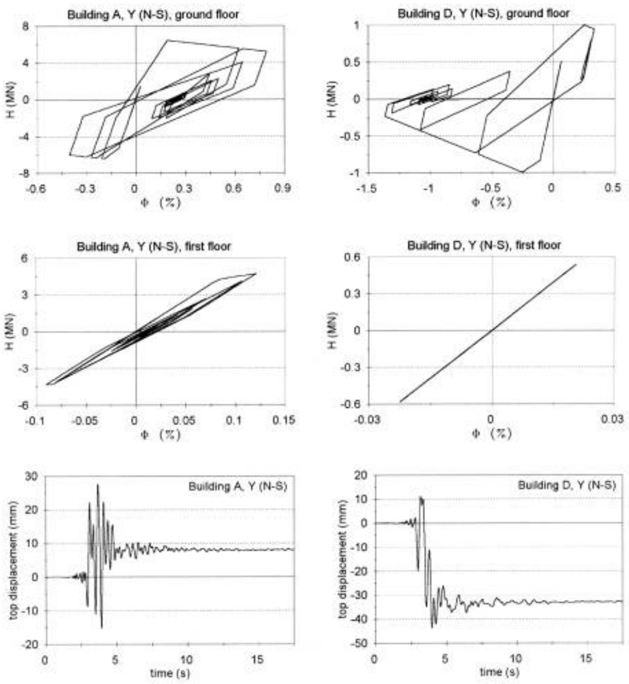

Figure 10 : Hysteretic response of the first and second floor and displacement response time history of the top of the building (Tomaževič et al. 2005)

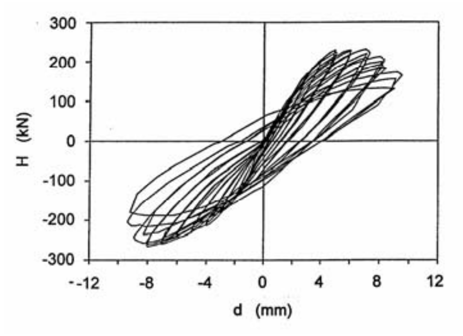

Figure 11 : Typical lateral load - displacement hysteresis loops, obtained by testing a stone-masonry wall

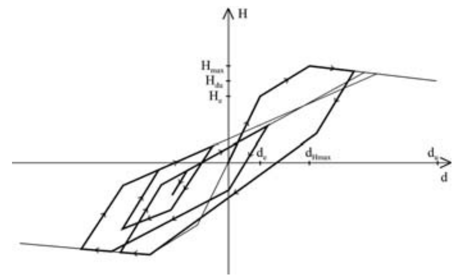

Figure 12 : Schematic presentation of hysteretic rules considered in the calculation of the non-linear seismic response (Tomaževič et al. 2005)

5 CONCLUSIONS

The correlation betweeen the damage observed on typical heritage buildings and calculated non-linear diplacement response to ground acceleration record of the same earthquake indicated that adequate results can be obtained by using simple non-linear MDOF shear system model with concentrated storey masses and non-linerar storey hysteretic rules as the input parameters.

By comparing the calculated response values of maximum story rotation with the experimental values obtained at characteristic limit states (Fig. 7), it can be seen that the dynamic response of the analyzed buildings to earthquake ground motion entered into the non-linear range of vibration. In other words, the occurrence of damage to structural walls during an earthquake as strong as was the earthquake of July 12, 2004, could have been expected, although the buildings had been previously strengthened by tying and cement grouting of stone masonry walls. The response analysis has also shown, that the occurrence of cracks in both, the ground floor and first story is to be expected in the particular case studied, as has been actually observed after the earthquake.

By definition, given in EC 8, design ground acceleration a_g is the maximum acceleration, which occurs on the bedrock during an earthquake with a return period of 475 years. As has been mentioned, in the region of Bovec design ground accelerations a_g = 0,225 g are expected according to the seismic hazard map of SloveniaAlthough the measured peak value of acceleration record (a_g = 0,47 g) was significantly higher than a_g = 0,225 g, which is the design ground acceleration (by definition the maximum acceleration, which occurs on the bedrock during an earthquake with a return period of 475 years) expected in the region of Bovec., During the earthquake of July 2004, significantly higher values of peak ground accelerations have been recorded (0,47 g). However, the expected design and maximum recorded values cannot be directly compared. Ground accelerations have been recorded on river gravel deposits of considerable depth. According to a preliminary backward response spectrum analysis, it has been estimated that spectral values on the firm soil would have been about two- to three-times lower than measured on the ground (Fajfar et. al., 2004). Very roughly, this would correspond to peak values of bed-rock accelerations between 0,16 g and 0,23 g, which means that the intensity of the earthquake was equal to or very close to the intensity of the design earthquake for the area of Bovec (a_g = 0,225 g). the response analysis indicated that the effect of earthquake was similar to the effect of the design earthquake. On the basis of this analysis it can be concluded that up-to-date code requirements regarding the resistance demand for heritage stone-masonry buildings are somewhat conservative and difficult to attain by means of economical strengthening measures. However, it can be also concluded that the requirements can be reduced by allowing slightly increased amount of damage to structural walls without risking the collapse of heritage buildings.

ACKNOWLEDGEMENTS

The research discussed in this paper has been part of the research program financed by the Ministry of High Education, Science and Technology, Agency for Research, and Ministry of Environment and Space, Agency for Environment, of the Republic of Slovenia.

REFERENCES

Eurocode 8: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings. 2004. *EN 1998-1*, CEN, Brussels.

Klemenc, I., Tomaževič, M., Lutman, M. 2000. The effect of earthquake of April 12, 1998 on buildings. Mechanical properties of stone-masonry and recommendations for design. *Report ZAG/PI-00/01*, Ljubljana (in Slovene).

Tomaževič, M. 2000. Seismic redesign of existing masonry buildings. *European Earthquake Engineering,* 14 (3): 59-66.

Tomaževič, M. 1999. *Earthquake resistant design of masonry buildings*. London: Imperial College Press.

Tomaževič, M. 1987.. Dynamic modelling of masonry buildings: storey mechanism model as a simple alternative. *Earthquake Engineering and Structural Dynamics,* 15 (6), John Wiley & Sons, 731-749.

Tomaževič, M., Lutman, M., Klemenc, I., Weiss, P. 2005. Behaviour of masonry buildings during the earthquake of Bovec of July 12, 2004. *European Earthquake Engineering*, 19 (1): 3-14.

Tomaževič, M., Klemenc, I., Lutman, M. 2000. Strengthening of existing stone-masonry houses: Lessons from the earthquake of Bovec of April 12, 1998, *European Earthquake Engineering*, 14 (1): 13-22.

Tomaževič, M., Lutman, M. 1996. Seismic behaviour of masonry walls: modeling of hysteretic rules. *Journal of Structural Engineering ASCE*, 122 (9): 1048-1054.

Tomaževič, M., Weiss, P., Velechovsky, T., Apih, V. 1989. Experimental study of grouting the stone-masonry walls. *Report ZRMK/PI-89/05*, Ljubljana (in Slovene).

Tomaževič, M., Turnšek, V., Terčelj, S. 1978. Computation of the shear resistance of masonry buildings. *Report ZRMK-IK*. Ljubljana (in Slovene).

Tomaževič, M. 1997. Seismic resistance verification of masonry buildings: following the new trends. In P. Fajfar and H. Krawinkler (eds.), *Seismic Design Methodologies for the Next Generation of Codes*: 323-334. Rotterdam: Balkema.

Turnšek, V., Čačovič, F. 1971. Some experimental results on the strength of brick masonry walls. *Proc. 2nd Int. Brick-masonry Conference*, Stoke-on-Trent: 149-15