1975

IN SITU TESTS FOR THE ASSESSMENT OF SEISMIC RESISTANCE OF OLD STONE-MASONRY HOUSES

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SUMMARY

On April 12, 1998, a local M = 5.5 earthquake with estimated epicentral intensity VII - VIII by European Macroseismic Scale (EMS) severely damaged more than 300 buildings in the area of Upper Posočje (Soča River Valley) in western Slovenia. By carrying out the in situ lateral resistance tests of existing and cement-grouted stone-masonry walls, typical for the traditional construction in the region, mechanical characteristics of masonry have been obtained. On the basis of experimentally obtained data, the seismic resistance of a series of existing buildings has been assessed and the effectiveness of strengthening the walls by cement-grouting verified. By correlating the observed degree of damage to buildings and the calculated values of seismic resistance, it has been found that the values of effective ground accelerations during the earthquake did not exceed 0.15 g. It has also been found that, if cement-grouted, adequate seismic behaviour of stone-masonry can be ensured in the zones, where the design ground accelerations of up to 0.2 g are expected (EMS intensity VIII).

INTRODUCTION

The earthquake of April 12, 1998, which struck the area of Upper Posočje (Soča River Valley) in western Slovenia was not very strong by magnitude and intensity. The epicentral intensity of the M = 5.5 main shock was estimated to be of grade VII - VIII by the new European Macroseismic Scale (EMS - European, 1993). Nevertheless, it caused considerable damage to more than 300 buildings. Among them, about 1/3 have been damaged seriously, in most cases beyond repair, whereas 2/3 suffered repairable damage.

The region of Posočje has already suffered damage from a series of earthquakes with epicentres in neighbouring Friuli, Italy, in 1976. At that time, more than 6100 buildings have been damaged: 1700 had to be demolished and rebuilt, and 4400 have been repaired and/or strengthened by tying the walls with steel ties and cement-grouting the walls.

It is a rare case that the buildings, strengthened after an earthquake, have been subjected to another strong earthquake within a short time interval of 22 years. However, the consequences of earthquakes of 1976 were not so severe in the area which suffered in 1998 and not many buildings in the recently affected area have been severely damaged in 1976 and thoroughly strengthened afterwards. Nevertheless, the earthquake of 1998 provided a good opportunity to analyse the causes of poor behaviour of buildings and identify some mistakes made during the reconstruction of the area in 1976.

In order to analyse the seismic resistance of typical buildings in the region and to verify the improvement obtained by strengthening the walls, the mechanical characteristics of existing and strengthened stone-masonry, typical for the region, have been determined by in situ tests. The experimentally obtained results have been used for the evaluation of seismic resistance of a series of typical buildings in both existing and strengthened states. The results of tests and subsequent analysis will be reported in this paper.

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TYPOLOGY, DAMAGE AND SELECTION OF BUILDINGS FOR IN SITU TESTS

Traditional construction material in the region is locally available lime-stone. Stone-masonry walls are made of rubble stone, built in two outer layers of bigger stones, with an inner infill of smaller pieces of stone, in poor mud mortar with a little lime. Connecting stones are rare and only sometimes stones are cut or partly cut at the corners. With the exception of some traditional houses in remote villages, which are categorised as important architectural cultural assets of the region, most buildings in towns have been rebuilt after the destruction during the World War I. It has been found that the quality of stone-masonry of larger public buildings in towns is much better than the quality of, usually smaller, residential houses. It has been also found that the quality of stone-masonry in some villages is better than the quality of masonry of houses in towns, destroyed during World War I and rebuilt afterwards.

Typically, stone-masonry houses are 2 - 3 storeys high (Figure 1). Floor structures and lintels are traditionally wooden, without any wall-ties provided to connect the walls. Wooden floors and lintels have been replaced with reinforced-concrete slabs in the cases where houses have been remodelled to meet the demands of the growing tourism in the region in the last decades. However, wall-ties have been installed only in the case of a small number of buildings after the earthquake of 1976. 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 lesser quality than the walls of the structure above the ground level.

Figure 1. Typical village in the earthquake-damaged area

Structural layout, i.e. the distribution of structural wall in plan of typical houses, 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 % (see Table 2), which is much greater than required by Eurocode 8 for simple buildings (5 - 6 %).

A large number of voids and lack of connecting stones were the main reason for delamination and disintegration of stone-masonry walls, whereas lack of connection between walls and lack of anchoring of floors into the walls caused separation of the walls at vertical joints and intersection zones, as well as collapse of the walls, orthogonal to the main direction of the seismic motion.

Typical damage to stone-masonry houses can be classified into following categories: (a) horizontal cracks along the joints between walls and floors; cracks along closed wall and door openings, (b) vertical cracks at the corners and wall intersections; separation of walls, collapse of gables, (c) cracks in the structural walls, falling out of masonry at lintels, closed openings and in corner zones, and (d) heavy damage to walls, such as wide cracks, disintegration and partial collapse.

In order to assess the seismic resistance of typical stone-masonry houses in the region, as well as to verify the resistance of the strengthened houses, the values of mechanical characteristics, such as tensile (shear) strength, should be known. Since these values strongly depend on the type of stone-masonry and vary from region to region, actual data had to be obtained. Because of specific properties of masonry materials and the way of construction, it is not easy to reproduce the existing stone-masonry in the laboratory, although thorough mechanical and chemical analyses of the properties of constituent materials would have been previously carried out. Therefore, the values of mechanical characteristics needed for seismic resistance evaluation have been determined by testing the stone-masonry walls in situ.

Figure 2. Typical (left) residential and (right) public building, where the in-situ tests have been carried out

Taking into consideration the observed differences in the quality of existing masonry and structural configuration, tests have been carried out on traditional residential houses in villages as well on the public buildings in towns. Since many owners of buildings did not permit to use their houses for testing, it has not been possible to carry out tests in the most damaged part of the zone. A suitable building has been found in the village of Kal-Koritnica near the town of Bovec, which also suffered damage during the earthquake (building A - Figure 2, left). In the category of public buildings, however, the walls of two buildings have been tested (elementary school building in the village of Soča - building B, and Bovec Police station building - building C, Figure 2, right).

When selecting the suitable buildings for testing, attention has been paid that, for the specimens tested in the existing condition, a sufficiently large part of the masonry at a suitable location in the structure has not been damaged by the earthquake. As regards the cement-grouted walls, however, the masonry walls, from which the specimens in buildings A and B have been cut, have not been damaged before the grouting, whereas in the case of building C, the masonry suffered minor cracks during the earthquake.

PREPARATION OF SPECIMENS AND TESTING PROCEDURE

In the selected buildings and at an appropriate place, the specimen has been separated from the surrounding masonry by vertically cutting the wall at both sides by means of a saw. Attention has been paid that no cracks or other damage existed in the parts of the wall where the existing situation has been investigated.

In order to obtain data about the effectiveness of cement-grouting, cement-grouted walls have also been tested. To reduce the costs of testing, a pair of walls has been prepared and tested at the same time in each building. One of the specimens has been left as existing, whereas the other one has been strengthened by cement-grouting before the tests. For grouting, a mix consisting of Portland cement PC 35 and water in the proportion of weight varying from 1.5 : 1 to 1.22 : 1, with 0.3 % in weight of special additive to prevent shrinking, has been used. The quantity of the dry mix, needed to fill the voids in the walls, varied from 60 to 90 kg/m^3 of masonry.

After cutting, the contact surface between the wall and the steel beam where the lateral load has been applied has been strengthened with a concrete layer in order to prevent local crushing of stone-masonry. Then, a system of steel rods and supporting beams, accommodated to the actual situation in the building, has been installed to transfer the lateral force from hydraulic jack to the specimen. In order to prevent the occurrence of additional damage in the supporting part of the wall, attention has been paid that hydraulic jack has been laterally supported by a strong enough portion of the wall. In vertical direction, the surrounding part of floor structure has been supported with wooden posts. In this way, the accidental collapse of the floor in the case of possible collapse of the tested specimen has been prevented.

In the case of building A, the specimens have been tested as vertical cantilevers. Since the walls did not carry any vertical load from the upper part of the structure, compressive stresses have been induced by means of an additional hydraulic actuator, acting vertically. In order to allow the wall to freely deform when subjected to lateral load, the actuator has been placed below the wall and the vertical force transferred to the top of the wall by means of a system of supporting beams and steel rods. Horizontal load has been applied at the top of the wall. In the case of buildings B and C, however, the walls have been fixed at the bottom and top in the surrounding masonry and tested under the existing vertical load, caused by the weight of the upper structure and floors. In this case, horizontal load has been applied at the mid-height of the wall.

The specimens have been instrumented with displacement meters, placed horizontally and dilatometers, placed in the diagonal direction and fixed to the wall on one of the wall's surfaces. The force induced by the actuator has been measured by means of a load cell. The arrangement of tests in the case of the walls tested in building A is presented in Figure 3, left.

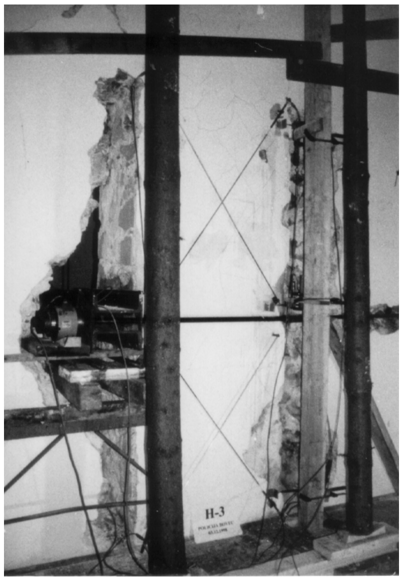

Figure 3. (Left) Arrangement of test - building A and (right) crack pattern in a tested cement-grouted wall at ultimate state - building C

Lateral displacements have been imposed, so that the specimens could be tested in the non-linear range. The displacements have been gradually increased, with unloading of the specimen at each step, until the resistance started to degrade and heavy damage developed. The tests have been terminated before the final collapse of the specimens, except in the case of walls A, which did not support the floor structure.

In order to obtain information about the compressive strength of existing stone-masonry in the region, an additional specimen has been prepared and tested by vertical compression in building A. In this case, the same system of vertical loading has been used as in the case of vertical cantilevers, tested by subjecting them to lateral lateral load acting at the top of them.

TEST RESULTS

As expected, all walls, existing and cement-grouted, failed in shear. Diagonally oriented cracks developed, beginning at the point where the lateral load has been applied and directed to the opposite corners of the wall at the top and bottom of the specimen. Typical crack patterns for the case of the fixed-ended specimen, tested in building C, are shown in Figure 3, right. Except in the case of building A, tests have been terminated before the final collapse of the specimens.

According to the assumptions of the shear resistance theory, tensile strength of the masonry f_t is defined as the principal tensile stress, developed in the wall, idealised as an elastic, homogeneous and isotropic element, at the attained maximum resistance H_{max} (Turnšek and Čačovič, 1971):

$$f_t = \sqrt{\left(\frac{\sigma_o}{2}\right)^2 + (b\,\tau_{H_{max}})^2} - \frac{\sigma_o}{2}, \qquad (1)$$

where

- σ_o = the average compressive stress in the horizontal cross-section of the wall due to vertical load,
- τ_{Hmax} = the average shear stress in the horizontal cross-section of the wall at the attained maximum resistance H_{max}, and
- b = the shear stress distribution coefficient, which depends on the height/length ratio and vertical/lateral load ratio at the attained maximum resistance. In the particular case of the tested walls, the value b = 1.5 can be assumed.

The values of shear modulus G have been evaluated on the basis of effective stiffness K_e of the specimens, determined from lateral load - displacement relationships (see Figure 4), and taking into account the value of elastic modulus E of stone-masonry, obtained by vertical compression test at 30 % of compressive strength, as well as geometric characteristics of each individual specimen. The following equation has been used to determine the value of shear modulus:

$$G = \frac{K_e}{\frac{A_w}{1.2\,h} - \frac{1}{1.2}\frac{K_e}{E}\left(\frac{h}{l}\right)^2}, \qquad (2)$$

where

- A_w = the area of horizontal cross-section of the wall,
- h = the relevant height of the wall,
- l = the length of the wall.

By vertical compression test of an existing stone-masonry specimen located in building A, the values of compressive strength f_c = 0.98 MPa and modulus of elasticity at 30 % of compressive strength E = 2655 MPa have been obtained. Although only one value of modulus of elasticity has been obtained experimentally, this value has been taken into account in all cases of evaluation of the shear modulus.

Typical relationships between the lateral load and rotation angle (displacement/half-height of the wall ratio), obtained by testing the existing and cement-grouted specimens in buildings B and C are shown in Figure 4a and 4b, respectively. The dimensions of the specimens and test results are summarised in Table 1, where the ratios of improvement of the tensile strength in increase in shear modulus due to cement-grouting are also indicated.

Table 1. Dimensions of tested walls and test results

Building	Wall	l x d x h (m)	σ_o (MPa)	f_t (MPa)	Grout./Exist.	G (MPa)	Grout./Exist.
A*	Existing	0.98 x 0.52 x 1.63	0.54	0.06	1.84	84	2.07
A*	Grouted	1.00 x 0.52 x 1.60	0.65	0.11		174	
B	Existing	1.00 x 0.65 x 2.54	0.20	0.06	2.83	181	1.86
B	Grouted	1.00 x 0.65 x 2.52	0.19	0.17		337	
C	Existing	0.98 x 0.64 x 2.51	0.18	0.10	2.20	151	3.11
C	Grouted	1.03 x 0.66 x 2.50	0.23	0.22		470	

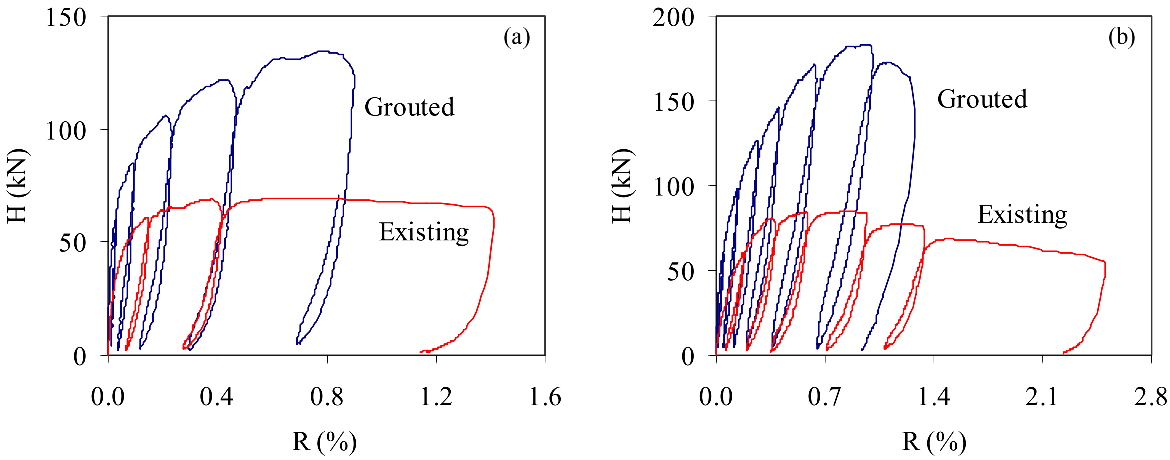

Figure 4. Lateral load - rotation relationships obtained by in situ lateral resistance tests of existing and strengthened walls in (a) building B and (b) building C

As can be seen, by cement-grouting, the tensile strength of the stone-masonry, which determines the shear resistance of walls, is significantly improved. It can also be seen, that the values of shear modulus, which determine the stiffness of the walls in the particular case of stone-masonry, is also increased. Taking this into consideration, it can be concluded that changes in distribution of seismic loads onto the walls may take place if only single walls in the plan of a storey are strengthened. This may result into an unexpected behaviour of the partially strengthened building during earthquakes. In order to avoid this, it is recommended that cement-grouting of stone-masonry walls, if needed, be executed uniformly over the entire floor area.

ASSESSMENT OF SEISMIC RESISTANCE

On the basis of experimental data, the seismic resistance of a series of typical houses in different locations has been analysed. For the analysis, a push-over method, developed at ZAG (Tomaževič, 1997), has been used. By this method, the resistance envelope of the critical storey is calculated by imposing the structure to step-wise increasing lateral displacements in the shape of the first mode of vibration. The results of calculation for existing and cement-grouted houses are summarised in Table 2. For easier comparison with the design values of ultimate base shear coefficient BSC_u, the seismic resistance is given in a non-dimensional form of the coefficient of seismic resistance CSR_u, i.e. the ratio between the calculated ultimate resistance (base shear) H_u and the weight of the building W, for both orthogonal directions of the building. To present the basic structural characteristics of the analysed houses, the wall/floor area ratio has also been evaluated for both directions.

Considering fact that all analysed buildings were 2 stories high, it can be concluded that, generally speaking, the seismic resistance increases with the increased wall/floor area ratio, as expected. It has to be noticed, however, that the type of buildings also plays an important role. In the case of public buildings where the quality of masonry was better, the height of floors was also higher than in the case of traditional residential houses. This resulted in greater weight of the building and, consequently, in reduced coefficient of seismic resistance, despite the same wall/floor area ratio and better quality of masonry.

In the case of seismic resistance verification of masonry buildings according to specifications provided in EC 8 (Eurocode 8, 1994), the ultimate coefficient of seismic resistance of the building CSR_u should be greater or equal to the ultimate design base shear coefficient (design seismic resistance/weight of the building ratio) BSC_u:

$$BSC_u = \frac{a_g \, S \, \beta_o}{q}, \tag{3}$$

where:

- a_g = the design ground acceleration, depending on the seismic zone,
- S = the soil parameter,
- β_o = the maximum spectral value, constant within the vibration period interval between T = 0.1 s and T = 0.4 s (which is the case of masonry buildings under consideration), and
- q = structural behaviour factor.

Table 2. Seismic resistance of existing and strengthened stone-masonry buildings in terms of coefficient of seismic resistance ($CSR_u = H_u/W$)

Building No.	No. of stories	Wall/floor area (%)		Existing			Strengthened		
		x-dir.	y-dir.	f_t (MPa)	CSR_{ux}	CSR_{uy}	f_t (MPa)	CSR_{ux}	CSR_{uy}
1	2	12.0	9.1	0.08	0.21	0.19	0.14	0.25	0.25
2	2	10.9	6.4	0.08	0.20	0.15	0.14	0.27	0.22
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

Assuming the values of S = 1, β_o = 2.5, and q = 1.5, as specified in EC 8 for plain masonry buildings, and considering the design ground acceleration value ag = 0.2, recommended to be applied for the seismic intensity zones VIII by EMS scale, the value of BSC_u = 0.33 is obtained. Comparing the values of CSR_u, given in Table 2 with the value of BSC_u = 0.33, it can be seen, that poor behaviour of the analysed existing houses could have been expected. On the other hand, however, thoroughly strengthened buildings, with cement-grouted walls and steel ties to prevent the separation of the walls, are not expected to suffer any substantial damage even when subjected to the expected earthquake of intensity VIII by EMS scale.

CONCLUSIONS

Seismic resistance of traditionally built stone-masonry houses in the area of Posočje, an earthquake-prone region in Slovenia, where recently two relatively strong earthquakes, occurring within 22 years time interval, damaged a significant number of buildings, has been investigated. In order to obtain reliable data about the mechanical characteristics of existing and strengthened local stone-masonry, a number of in situ tests has been carried out on both existing and cement-grouted specimens. It has been found that the quality of masonry varies by building typology (residential - public) as well as by location (rural areas - towns).

Generally speaking, two basic classes of quality of stone-masonry walls in the region have been distinguished on the basis of test results. In the case of stone-masonry of improved quality, specific for public buildings in towns, the values of tensile strength, which is the predominant parameter defining the seismic resistance, f_t = 0.08 MPa and f_t = 0.14 MPa can be considered for seismic resistance verification in the existing and strengthened situation, respectively. In the case of stone-masonry of poor quality, however, which is specific for rural houses, the corresponding values for existing and strengthened state are f_t = 0.06 MPa and f_t = 0.11 MPa, respectively.

The experimentally obtained results have been used as the input data for seismic resistance evaluation of a series of buildings in both existing and strengthened condition. As the correlation of earthquake damage and the calculated values of coefficients of seismic resistance of the analysed buildings indicate, poor behaviour of stone-masonry houses could have been expected, when subjected to EMS intensity VII - VIII earthquakes. However, if the integrity of buildings is provided by means of the tying of the walls with steel ties and anchoring the floor into the walls, and the walls are strengthened with cement-grouting, such buildings are not expected to suffer substantial damage. It can be concluded that in the zones, where the design (effective) ground accelerations up to 0.2 g are expected, which is the case of EMS intensity VIII earthquakes, the existing stone-masonry walls need to be systematically strengthened by cement-grouting in order to be capable to resist the expected seismic loads.

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