SEISMIC STRENGTHENING AND REDESIGN OF OLD MASONRY BUILDINGS: FOLLOWING THE NEW TRENDS

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ABSTRACT
The paper presents Slovenian experience regarding seismic redesign of old masonry buildings and application of traditional and up-to-date methods of strengthening. Among Eurocodes, European standards for structural design, a standard to regulate the seismic assessment and strengthening of existing buildings has been prepared, where the procedures and requirements to ensure the fulfillment of minimum demands are specified. Although the basic principles of earthquake resistant design are followed, some requirements regarding the seismic redesign of old masonry buildings of historic importance need to be modified, like the requirement to simultaneously use of confidence and partial material safety factors for the reduction of experimentally obtained values of mechanical properties of masonry in seismic resistance verification. As the analysis of damage to retrofitted masonry buildings, subjected to design-level earthquakes twice in just a few decades, indicated, such a requirement is too conservative and would lead to unacceptable (and unnecessary) structural alterations. The applicability and efficiency of various strengthening methods, using traditional and synthetic materials, is also discussed. Whereas the use traditional materials and methods (reinforced cement/concrete/ shotcrete coating, repointing, injecting) has been already verified by both, laboratory tests and earthquakes, it is not all clear as regards the application and efficiency of various methods based on the use of carbon or glass fiber reinforced polymers.

KEYWORDS: old masonry buildings, assessment, seismic redesign, strengthening methods, code requirements

INTRODUCTION
Old masonry buildings, including architectural cultural heritage buildings, have not been conceived to resist earthquake loads. They have been built in materials and systems which resist the compression, caused by the gravity loads, but not bending and shear caused by the earthquakes. In addition, materials are frequently deteriorated because of poor maintenance and environmental influence. Consequently, most of the damage to buildings and death toll due to earthquakes in the recent past occurred as the result of inadequate seismic performance of such buildings. Despite more or less systematic, coordinated research and preventive activities have been initiated already more than three decades ago, old masonry buildings remain that part of the building stock, which in developed countries carries the risk of being heavily damaged or destroyed during strong earthquakes, as a contrast to the new buildings designed according to contemporary principles of seismic resistant design. To improve the seismic resistance of such buildings, methods have been developed and codes have been prepared on the basis of experimental research and earthquake damage observations. However, there is still a long way to

go before the methods be systematically applied and seismic risk of masonry buildings in old urban and rural nuclei be preventively reduced to an acceptable level.

Monumental buildings are individual buildings, representing the most important part of architectural cultural heritage. They are either preserved as such (like remains of monumental buildings exposed to visitors) or they remain serving in their original intended use (like churches) or their service is modified (like industrial buildings modified to shopping malls). Technically, they represent individual cases, where the main principles of seismic strengthening and redesign are followed, whereas strengthening techniques need to be developed and designed from case to case. In most cases, strengthening methods strongly depend on the requirements of art historians and conservators.

Buildings in historic urban and rural nuclei, which today are also considered as architectural cultural heritage, are clusters of residential buildings, including also buildings of public character. They have been subjected to continuous process of modification and change during their life-time. Additional stories have been added to original buildings and new annexes built in the backyards and spare spaces between the individual houses along the streets. The buildings have been continuously adapted and reconstructed to follow the needs of inhabitants. If these buildings should be preserved as the vital part of cities and towns in the future, this process should not be interrupted by simply classifying the buildings into the category of architectural cultural heritage. Because of their intended use and occupancy, the same level of living standards and safety should be ensured for their inhabitants and users as in the case of the new construction. The interventions needed to achieve such requirements are not always in line with the requirements of preservation of architectural heritage. Therefore, the solutions represent a compromise between engineering demands and available technologies, economic considerations and principles of preservation. The philosophy "better something than nothing" is many times followed when deciding upon the technical solution to be applied for strengthening the structure. Experiences show that even partial interventions in the right place improve the seismic behavior.

Some technical issues regarding the seismic redesign and strengthening of such buildings and experiences obtained after the actual earthquakes in Slovenia will be discussed in the following.

TYPOLOGY OF BUILDINDS AND SEISMIC BEHAVIOR

Typology of old masonry buildings varies from region to region, from rural to urban areas. As is the case of other countries, traditional construction materials of heritage masonry houses in Slovenia are locally available limestone and slate, which in some parts of the country are replaced by clay brick. Stone masonry walls are made of rubble or river-bed stone, usually built as three-leaf walls 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. Lack of bond between the layers frequently results into the delamination and disintegration and subsequent collapse of the walls when subjected to in-plane and out-of-plane seismic loads.

The connection between the structural elements of historic buildings is adequate for gravity loads, but the elements which would ensure the monolithic behavior of the structure when subjected to seismic loading, like wall ties and rigid floors, are frequently missing. Masonry buildings are typical box-type structures, so that structural integrity should be ensured to utilize the available resistance of structural walls when subjected to lateral loads. If the walls are not well connected together, they separate at vertical joints and the walls orthogonal to the direction of earthquake motion start vibrating out of their planes. In some cases, vertical cracks in the walls develop due to out-of-plane bending as a result of large spans between the bracing walls. In the others, however, parts of such walls or the walls as a whole overturn and collapse because of the loss of stability (Figure 1).

Figure 1: Out-of-plane Collapse of Walls

Figure 2: Shear Failure of Walls

Old, historic masonry houses generally fulfill the requirements for structural regularity, such as uniform distribution of structural walls in both directions and along the height of the building, needed to achieve adequate seismic behavior. Wall/floor area ratio is relatively high, of the order of 10 %, and the distribution of load-bearing walls in both orthogonal directions is uniform. Unfortunately, the original adequate layout has been many times modified during the reconstructions in the recent past. Especially in the cities, new stories have been added to buildings and large parts of structural masonry walls have been removed in the ground floor to make place for shops and arcades along the streets. Since the removed parts have not been replaced with load-bearing elements, compatible in terms of resistance and deformability with the original masonry, and adequately connected with the remaining structural system, these buildings are extremely vulnerable to earthquakes. Heavy damage and even collapse of such buildings when subjected to design level earthquakes is almost inevitable.

Besides adequate structural integrity and layout, strong, sufficiently resisting walls are required, able to carry the seismic loads induced in the building during the earthquake and transfer them to the foundation system and soil. As a result of mechanical properties of masonry and characteristics of structural system, the resistance of buildings to seismic loads is governed by shear. Typically, diagonally oriented shear cracks develop in structural walls. In the case of the multi leaf stone masonry, structural walls may also delaminate and disintegrate if the duration of earthquake is long. Although such buildings are usually built without any specific foundation, the damage which might be attributed to foundation failure is rare. Loss of stability of foundation soil, such as land-sliding or soil liquefaction, may result into heavy damage of the upper structure.

SEISMIC LOADS FOR REDESIGN

The level of strengthening of existing buildings depends on the acceptable level of seismic risk. European standard for the assessment and retrofitting of buildings, EN 1998-3 [1] requires that the existing buildings be strengthened to achieve the same level of seismic safety as the new construction. Although the definition of design earthquake is slightly different, the requirements for the strengthened existing buildings are basically the same. Accordingly, the existing buildings should be (re)designed to withstand the earthquake with return period 475 years and 10 % probability of exceedance in 50 years, "without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic events" (no collapse requirement; ultimate limit state). As in the case of the new construction, the verification that the strengthened structure will resist an earthquake having a larger probability of occurrence than the design earthquake, i.e. earthquake with return period 95 years with 10 % probability of exceedance in 10 years, "without the occurrence of damage and limitation of use, the costs of which would be disproportionately high in comparison with the costs of the structure itself" (acceptable damage requirement; damage limitation state), should be also carried out [2]. Because of specific properties of masonry and small differences between story drifts at ultimate state and crack limit state, however, only the verification of ultimate limit state is usually sufficient.

For the range of vibration periods, T, of most masonry buildings, 0.05 s $\leq T \leq$ 0.25 s, where the design response spectrum is flat, the design spectral values, $S_{\mathrm{d}}(T)$ are determined by:

$$S_{\mathrm{d}}(T) = a_{\mathrm{g}} \cdot S \cdot \frac{2.5}{q}, \tag{1}$$

where a_{g} = the design ground acceleration on bedrock, expressed as a part of acceleration of gravity, g = 9.81 ms^{-2} , S = soil factor, and q = structural behavior factor (force reduction factor). Because it is non-dimensional, the design spectral value can be expressed also in the form of the design base shear coefficient, BSC_{d} ($BSC_{\mathrm{d}} = BS_{\mathrm{d}}/W$, where BS_{d} = design base shear, and W = the weight of the building above the base). Typical values are given in Table 1 for the case of the soil factor S = 1.2, a typical situation of foundation soil: several tens of meters thick deposits of dense sand and gravel or very stiff clay.

Table 1: Eurocode 8-1 Design Seismic Loads

Design ground acceleration a_g	0.05	0.10	0.20	0.25	0.30
Approx. intensity by EMS* scale	VI	VII	VII-VIII	VIII-IX	IX
Design ground acceleration Sa_g for S = 1.2	0.06	0.12	1.2	0.17	0.36
BSC_d for q = 1.5	0.10	0.20	0.40	0.50	0.60
BSC_d for q = 2.0	0.08	0.15	0.30	0.38	0.45
$BSC_{d,r}$ for q = 2.5	0.06	0.12	0.24	0.30	0.36

*European Macroseismic Scale [3]

According to official seismic hazard map of Slovenia [4], a_g = 0.25 should be considered in the design of buildings in the zones of highest expected seismicity, although higher values of ground accelerations have been already recorded during the recent seismic events [5]. As can be seen, the code required values of BSC_d for plain masonry structures and behavior factor q = 1.5 are high.

However, by analyzing damage to old masonry buildings, strengthened after the earthquake a few decades ago and subjected to a recent repeated earthquake of similar intensity, it has been found that adequately strengthened buildings survived the repeated design level earthquake with only moderate damage or even undamaged, although their calculated resistance did not meet code demands for collapse situation [e.g. 6, 7]. In the study of a group of 16 mainly two-story stone masonry buildings with average wall/floor area ratio 10 %, mean values of mechanical properties of masonry obtained by in-situ testing of walls in typical buildings have been considered. The average resistance of 16 buildings, expressed in the non-dimensional form of seismic resistance coefficient SRC ($SRC = R/W$, where R = the resistance of the ground floor and W = the weight of the building above ground), amounted to SRC = 0.33 (c.o.v. = 22 %). According to Eurocode 8 the design base shear coefficient equal to BSC_d = 0.45 (a_g = 0.225, S = 1.2, q = 1.5) should be taken into account in seismic resistance verification in that particular seismic zone. It should be mentioned that the resistance has been determined on the basis of resistance curves, calculated by a push-over method. In the case that traditional equivalent elastic static method had been used, the observed difference between the code demands would have been much greater.

Taking this and the observed earthquake damage into consideration, it can be concluded that the Eurocode's recommended value of behavior factor for plain masonry buildings (q = 1.5) can be increased to q = 2.0 in the case of adequately strengthened old masonry houses. This value is still within the range of Eurocode's suggested possible values of behavior factor for plain masonry structures (q = 1.5–2.5). The proposed reduction does not influence the safety of the building against collapse. Only a slight increase of damage when subjected to design earthquakes can be expected, which, however, will remain within the limits of the acceptable damage. It has been found [8] that, in order to fulfill the condition for damage limitation, the design ultimate state should be defined by either interstory drift value where the resistance degrades to 80 % of the maximum, or interstory drift value, equal to 3-times the value of interstory drift at the crack limit, whichever is less:

$$\Phi_{du} = \min \{\Phi_{0,8Rmax}; 3\Phi_{cr}\}, \tag{2}$$

where Φ_{du} = interstory drift (rotation) at design ultimate limit state, $\Phi_{0,8Rmax}$ = interstory drift (rotation), where the resistance degrades to 80 % of the maximum, and Φ_{cr} = interstory drift (rotation) at the occurrence of the first cracks, crack limit. The analysis of experimental results indicated, that the average value of interstory drift at crack limit amounts to approximately 0.3 % of the story height.

Numerical simulation of non-linear dynamic behavior of a few selected buildings, using the in-situ obtained data of mechanical properties of masonry and nearby recorded earthquake accelerogram (peak ground acceleration a_{max} = 0.47 g) yielded the same conclusions [7].

The reduction of design seismic loads for the case of the existing buildings of historic importance in high seismic zones has been already permitted in one on the previous versions of Eurocode 8-3 if "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, as well as where code required a_g values for redesign of a monument would lead to completely unacceptable architectural alterations" [9]. The reduction, however, should not exceed 1/3 of the design base shear coefficient value for new construction.

DESIGN RESISTANCE AND MECHANICAL PROPERTIES OF MATERIALS

Despite significant advances in modeling and computer capabilities, reliable and user friendly models of seismic analysis of existing masonry buildings of historical importance have yet to be developed. Depending on structural characteristics and importance of the building, different methods can be used for the verification of limit states. Non-linear dynamic response (time history) analysis is used in specific cases of the most important structures. In the case of the usual design practice and regular structures, however, either linear-elastic methods, such as lateral force method and modal response spectrum analysis, or non-linear static push-over type methods are used. In the case where the linear-elastic methods are used, the ductility and energy dissipation of the structure are taken into account implicitly by reducing the elastic seismic loads with the so called structural behavior factor, q. In such a case, only the design resistance of the structure and structural elements is compared with the design seismic shear and action effects, respectively. In the case of masonry structures, very limited redistribution of action effects from more to less loaded elements is permitted.

In the case of the push-over methods, where the resistance curve of the structure is calculated on the basis of the mechanism models and by taking into account the redistribution of lateral loads to structural walls, the first step of verification is the same as in the previous case: the calculated resistance of the structure is compared with the design seismic shear. However, in addition to resistance, ductility and displacement capacity of the structure should be also verified and compared with the code demands. In this regard, damage limitation criteria are essential: the structure should not be designed (or the existing structure redesigned) for design seismic loads, which would cause excessive damage to structural system.

Numerical model used for the seismic resistance verification of existing masonry buildings should reflect the actual seismic behavior of the structure under consideration. In the case of residential buildings with regular structural configuration, the models, developed for earthquake resistance verification of modern masonry structures, can be used, providing that the basic assumptions of such models, like rigid floor diaphragm action, are fulfilled also in the case of the analyzed historic building. Otherwise, the models should be modified to take into consideration the actual structural behavior.

In the case of monumental buildings with complicated structural layout, finite element methods can be used. However, it should be borne in mind that by using elastic finite element models, only the potential weak points can be identified, where the structure is prone to damage because of concentration of stresses. Elastic models cannot provide reliable information regarding the actual seismic resistance and seismic behavior. Since the use of the non-linear models for the analysis of seismic response of masonry structures is time consuming and requires specific skills, these models are used only exceptionally.

In the case of the existing masonry buildings with regular structural configuration, where measures have been taken for the tying of the walls and rigid floor diaphragm action, shear mechanism and shear resistance of the walls govern the seismic behavior. Because of specific relationships between the moduli of deformation at compression and shear, shear deformations prevail. Consequently, the stiffnesses of resisting walls are proportional to their horizontal section areas and do not significantly depend on the boundary conditions. The idea to use a relatively simple story mechanism model [11] has been significantly improved in recent years [12, 13].

For seismic vulnerability studies of existing buildings, where the walls are not tied with steel ties and connected with floor diaphragms, out-of-plane vibrations are critical, which cause separation and subsequent local failure of walls, located orthogonal to the main seismic motion. To estimate the resistance of the building, acceleration values which cause the separation of the assumed portion of the building, are calculated [14, 15]. In the calculations, actual mechanical properties of masonry are taken into consideration. Several possible mechanisms are verified: the critical one, which determines the seismic resistance of the building as a whole, is the mechanism, where the ratio between the acceleration, causing the mechanism, and acceleration of gravity attains the minimal value.

Besides adequate numerical model, which requires reliable information regarding the type of the structure and constituent elements, adequate information regarding the mechanical characteristics of structural materials is needed. Such information can only be obtained by means of inspection of the building in-situ and testing of materials and elements in-situ and/or in the laboratory. Depending on the thoroughness of inspection and amount of testing, three knowledge levels are defined in Eurocode 8-3, which determine the admissible method to be used for structural analysis and estimate the reliability of values of mechanical properties of materials taken into account in the calculations. According to knowledge levels, material strength values are reduced by so called confidence factors, *CF*, namely:

- knowledge level KL1: limited knowledge, $CF = 1.35$;
- knowledge level KL2: normal knowledge, $CF = 1.20$;
- knowledge level KL3: complete structural knowledge, $CF = 1.00$.

A detailed definition of each knowledge level is given in the code.

As recommended by Eurocode 8-3, mean values, obtained by testing, and not characteristic values of mechanical properties of materials as in the case of the newly designed structures, are considered in the redesign. Depending on the knowledge level, these values are reduced by confidence factor, *CF*, the value of which depends on the thoroughness of inspection of the building and reliability of data needed for structural evaluation. However, besides the reduction by confidence factor, *CF*, the code requires that partial safety factors for material, γ_M, be also taken into account to calculate the design values of material strength:

$$f_d = \frac{f}{C_F \gamma_M}, \tag{3}$$

where f_d = the design value of material strength, f = mean value of material strength, determined by testing, C_F = confidence factor, depending on knowledge level (KL), and γ_M - partial material safety factor for masonry, as specified by Eurocode 6-1, European standard for the design of masonry structures [16]. According to Eurocode 6-1, the values of partial safety factor for masonry, γ_M, depend on the production control and inspection of works on the site. In normal situation, the values within the range from 1.5 (optimum production control and severe inspection on the site) to 3.0 (no proof regarding the production control and inspection) are considered. In seismic situation, the chosen value can be reduced by 1/3, however in no case γ_M should be smaller than 1.5.

There is no reason that besides confidence factors, *CF*, partial safety factors of materials, γ_M, be considered in the calculations. It is not possible to assess the uncertainties regarding the values of mechanical properties of materials, which result in partial materials safety factor and depend on the factory control and inspection at the site, as in the case of the new construction. At the time of construction of old masonry buildings, there has been no factory quality control of materials (what about stone?) and inspection on the construction site. Therefore, to obtain the data needed in redesign, the actual structural materials are tested and the actual values of mechanical properties are determined. According to code, test determined values should be reduced by $\gamma_M = 3.0$ in normal and $\gamma_M = 2.0$ in seismic situation. Consequently, only one half of the mean value of masonry strength, obtained by testing the actual materials, can be considered in redesign.

In the case of the previously mentioned group of 16 buildings, which survived a repeated design level earthquake with only moderate damage or even undamaged, the average design resistance, calculated by taking into consideration partial material safety factor $\gamma_M = 2.0$, would drop from $SRC = 0.33$ (no reduction of tested material strength: $CF = 1.0$, $\gamma_M = 1.0$) to $SRC = 0.21$ ($CF = 1.0$, $\gamma_M = 2.0$). This represents only about 60 % of the design seismic load. Nevertheless, the buildings resisted the last design level earthquake undamaged or only slightly damaged.

Moreover, it should be mentioned that the design ground acceleration value in the particular zone studied is $Sa_g = 0.27$, whereas measured peak ground acceleration value amounted to 0.47 g. The example is only an indication. Final conclusions should be made on the basis of thorough parametric analyses.

On the basis of such indications, modifications of code requirements can be proposed also regarding the determination of design strength of materials. Although the mean values, obtained by testing, should be taken into consideration, there is no recommendation regarding the number of specimens to be tested. Therefore, on the basis of experience and taking into consideration the usual scattering of results, obtained by the in-situ testing of the same type of existing masonry (± 20 %), it can be recommended, that in the case where at least two specimens are tested, the actual mean value of all test results be taken into account, $f_t = f_{t,m}$, as specified in the code. However, if only one test result is available, the value, reduced by considering the expected scattering of test results, should be considered: $f_t = f_t/1.2$.

The values of confidence factor, *CF*, given in the code for each knowledge level, seem to be too optimistic. On the basis of experience and earthquake damage studies, it is proposed that, in the redesign, the requirements of Eurocode regarding the confidence factor *CF* be modified as follows:

- knowledge level KL1: limited knowledge. No testing. The values of mechanical properties of masonry are taken from the literature for masonry type, corresponding to masonry type under consideration. Identification inspection only is carried out. $CF = 1.7$;
- knowledge level KL2: normal knowledge. Mechanical properties are obtained by testing at least one specimen in the cluster of buildings of the same typology. General identification of a given type of masonry in the cluster is carried out by removing plaster and opening the walls at least in one location in each story of the building under consideration. $CF = 1.20$;
- knowledge level KL3: complete structural knowledge. Mechanical properties of masonry are determined either by in-situ tests or in the laboratory by testing specimens, taken from the building under consideration. At least one specimen of the specific masonry type should be tested in the building and the composition of the masonry should be verified by removing plaster at least in one location in each story. No reduction. $CF = 1.00$.

PRINCIPLES AND METHODS OF STRENGTHENING

Numerous criteria should be considered when deciding if and how to retrofit the building. The basic criteria are of technical nature. The type, location and amount of interventions depend on the resistance of the building in the existing state, evaluated by the assessment of building characteristic on site and calculations. Structural type and quality of materials represent the main parameters, upon which the decision is made regarding the method and technology of intervention. Any information regarding the effectiveness of the selected method is of relevant importance in the decision-making process. In addition to technical, some general criteria, like costs of interventions with regard to importance of the building, availability of technology and skilled workmanship, duration of works and usability of building, among others, should also be considered. Last but not least, efficient quality control system and inspection should be available.

To improve the resistance and reduce seismic vulnerability, structural deficiencies, identified by earthquake damage analysis, should be remedied. To ensure the integrity and box-type seismic response of the building, structural walls should be adequately connected with wall ties, whereas the floors should be strengthened to ensure floor diaphragm action and anchored into the walls for uniform distribution of seismic loads onto resisting walls. If necessary, structural layout is improved by filling the openings in the existing walls, or building new walls on new foundations, adequately located in plan of the building and connected with existing floors. Masonry, compatible with the existing one as regards the mechanical properties, should be used, and new masonry walls or infills should be adequately bonded with the existing masonry. The increased resistance of the upper structure requires verification and improvement in the load-bearing capacity of the foundation system, if necessary.

STRUCTURAL INTEGRITY

Structural integrity is needed to fully utilize the available resistance of masonry walls to dynamic seismic loads. Therefore, the tying of the walls of historic buildings with wall ties represents the basic step of interventions in the structure for improving the seismic resistance. In most cases, the tying of the walls is sufficient for providing structural integrity, in the others, however, existing wooden floor structures should be strengthened, anchored and connected with masonry walls. The analysis of earthquake damage and experimental research already confirmed the effectiveness of the simple tying the wall with steel ties, where the reinforcing steel bars are placed at the floor levels on each side of the walls, and anchored at the ends on steel plates. The bars are threaded at the ends so that they can be prestressed after placing and fixed with nuts. By a series of shaking table tests of models of stone and brick masonry houses, the mechanism of action of the ties has been studied and ties design procedures proposed [17, 18].

Figure 3: Upper Corner Failure and Separation of Walls of Model Without Wall Ties [19]

Figure 4: Model with CFRP Ties Resisted 3.5-times Stronger Earthquake [19]

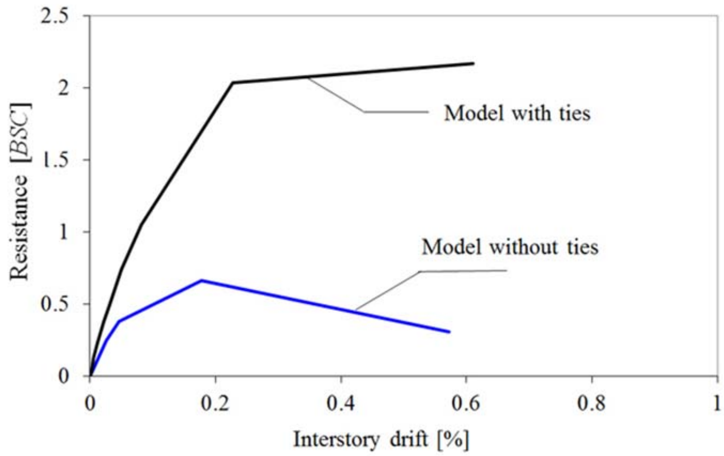

Figure 5: Relationships Between the Base Shear and Story Drift, Measured During the Shaking Table Tests of Brick Masonry Building Models With and Without Confinment. After [19]

Recent experiments indicated the possibility of replacing steel ties with reinforced polymer laminate strips (Figures 3 and 4 [19]). By confining the models of building with vertically and horizontally placed strips, separation of walls and out-of-plane collapse was prevented. As a result, the available resistance and displacement capacity of the structure has been fully utilized (Figure 5).

Figure 6: Rigid R.C. Sheared and Delaminated Stone Masonry Walls at Supports [10]

Figure 7: Similar Mechanism has Been Observed During Shaking Table Test of a Brick Masonry Model with R.C. Slabs

Theoretically, the replacement of wooden floors with massive reinforced concrete slabs represents the optimum solution for improving the structural integrity. Horizontally rigid slabs ensure good connection of the walls and prevent excessive out-of-plane vibrations. However, experimental investigations and post-earthquake observations indicate, that the replacement of wooden floors with rigid r.c. slabs sometimes adversely affects the seismic response [10, 18]. There have been numerous cases observed where the inadequately anchored and connected new slabs caused severe damage to existing structural walls, by shearing the walls and causing the delamination of stone masonry (Figure 6). The same kind of mechanism has been observed also in the laboratory (Figure 7). To prevent negative effect, new floor slabs should be sufficiently supported and adequately anchored into the walls.

STRENTHENING OF WALLS

Whereas most efficient method of strengthening of multi-leaf rubble stone masonry remains the injecting of cementitious grout into the voids in the wall, several traditional methods are available for strengthening the brick masonry walls, such as reinforced cement/concrete/ shotcrete coating and repointing, In many cases the efficiency of traditional strengthening methods has been already verified not only by laboratory and in-situ testing, but also in actual seismic situations.

The mix of bonding and filling materials, which is injected under pressure into the numerous voids of stone masonry walls, fills the voids and bonds the materials into a monolithic structure after hardening. As a result, delamination and disintegration of stone masonry when subjected to seismic loads are prevented and the integrity of the walls is ensured, which significantly improves the resistance.

Originally, the dry mix of consisted of 90 % of Portland cement and 10 % of pozzolana added to ensure the plasticity and injectability of the grout. Although the intervention is not visible after application, which suits the requirement of protection of architectural cultural heritage, the hardened cement grout undesirable effects. Namely, with hardened cement grout capillary active fabric is inserted in the masonry, which soaks the water from the ground and environment. Because of humidity, impurities in cement which dissolve in the water may damage frescoes and other decorations, frequently found on the surface of historic stone-masonry walls. Dampness may also appear on the walls after injecting the cement grout. Therefore, water-repellent additives in the form of specially prepared inorganic salts or stearic acids, are added to the mix. Since the strength of the hardened grout mix does not significantly influence the shear strength of the injected masonry, part of cement is replaced by inert aggregates in the form of fine-grained sands to improve the characteristics of the grout [20]. Bonding mechanisms and possibilities of replacing cement with masonry-friendly lime have been recently studied [21]. As a result of these studies, the composition of the grout mix can be designed for each particular type of masonry and for each particular problem to be solved. Locally available materials compatible with the original texture of historic walls can be used as a replacement of part of the cement in the grout in order to reduce the undesirable side effects to an acceptable level. However, some problems of injectability of mixes and hardening of the lime-based grouts, have yet to be resolved.

In the last couple of decades, synthetic materials, such as carbon (CFRP) or glass fiber reinforced polymers (GFRP) are replacing the traditional ones in the case where the strengthening of brick and stone masonry walls is needed. Various techniques of strengthening the masonry walls with polymers have been developed and their efficiency tested in the laboratories [e.g. 22−25]. The interest of using such methods of strengthening, which are time-effective and relatively clean, is growing with the decreased costs of polymers, especially GFRP.

In a recent study, carried out at Slovenian National Building and Civil Engineering Institute, a large series of brick and stone masonry walls, strengthened with different types of application of CFRP or GFRP mesh and/or fabric, laid in different ways in different types of matrices, have been tested [26 and 27]. In the case of the brick masonry, 24 walls have been strengthened by applying 10 different types of strengthening solutions. In addition, two control, not strengthened walls have been tested for reference. The walls have been strengthened by different types of coating, namely GFRP grid laid in fiber reinforced cementitious, 15 and/or 25 mm thick mortar, GFRP or CFRP uni-directional fabrics laid in 2 mm thick epoxy matrix, or by CFRP strips (plates), glued to the masonry and anchored into the foundation blocks and bond-beams with epoxy resin. The number of anchors which connected the coating to the masonry also varied. Typical layouts of representative strengthening types are schematically presented in Figures 8−11.

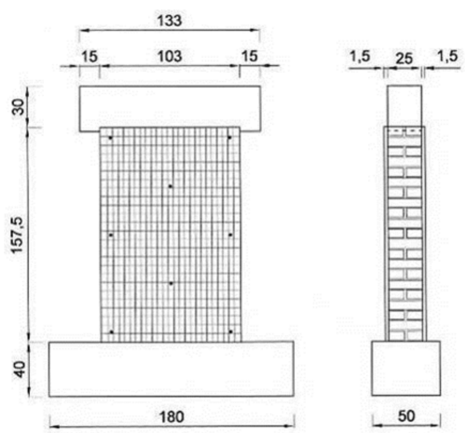

Figure 8: Brick Masonry, Vertically Placed GFRP Grid, 8 Anchors [26]. Measures in cm.

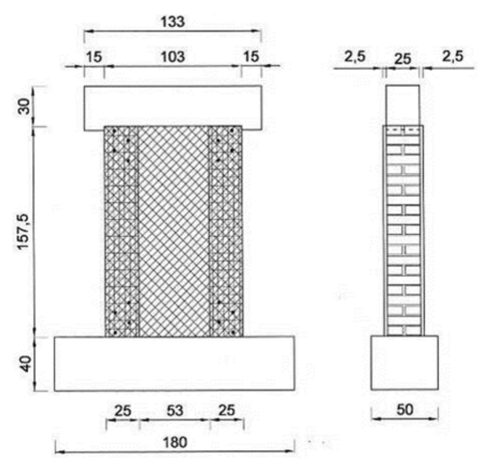

Figure 9: Brick Masonry, Diagonally Placed GFRP Grid Reinforced With Vertical Boundary Grid Strips, Anchored in Corners [26]. Measures in cm.

In the case of the stone masonry, 8 walls have been strengthened by applying 4 different types of strengthening types, whereas 2 walls have been tested for reference. The coating consisted of vertically or diagonally paced GFRP grid as reinforcement and 15−20 mm thick fiber reinforced cementitious mortar as a matrix. The coating, anchored to the wall in the corners, was placed on one or both sides of the wall. In one case, 30 cm wide GFRP fabric strips, placed vertically and diagonally on both sides of the wall and anchored to the wall in the corners, have been used as reinforcement, laid in epoxy resin matrix. Before application of coating, the surface of the walls has been leveled with fiber reinforced cementitious mortar. Typical examples are schematically presented in Figures 12 and 13.

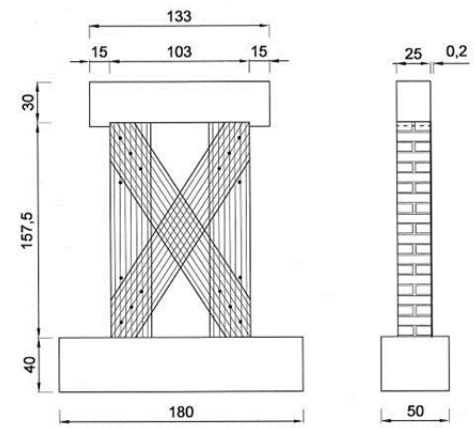

Figure 10: Brick Masonry, Diagonal And Vertical Fabric Strips, Anchored in Corners [26]. Measures in cm.

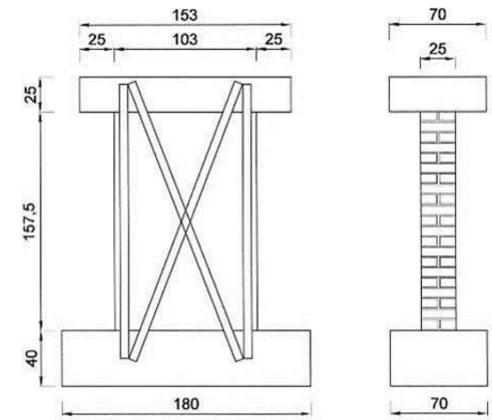

Figure 11: Brick Masonry, Diagonal And Vertical CFRP Plates/Strips [26]. Measures in cm.

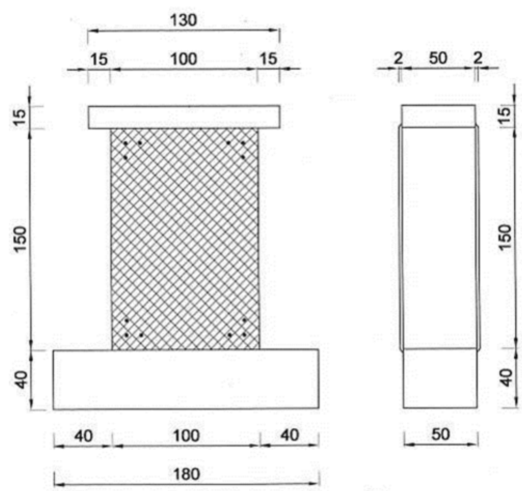

Figure 12: Stone Masonry, Diagonally Placed GFRP Grid, Anchored in Corners [27]. Measures in cm

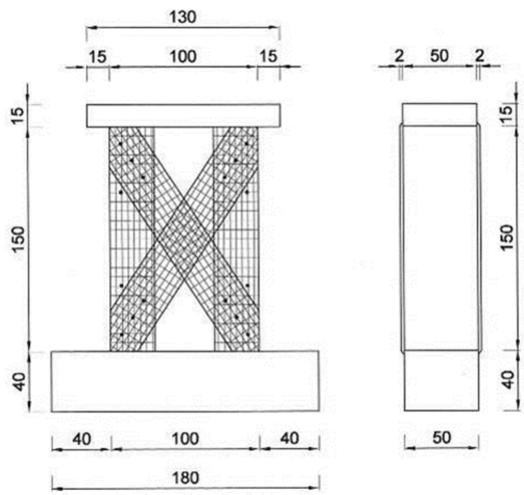

Figure 13: Stone masonry, Diagonal and Vertical GFRP Fabric Strips, Anchored [27]. Measures in cm

All walls were subjected to cyclic in-plane lateral load reversals at constant preloading of 25−30 % of the compressive strength of masonry. Shear behavior was predominant in all cases.

In the case of the brick masonry walls, the in-plane resistance was improved by 20–130 %, depending on the strengthening type. In the case of diagonally placed grid with vertical boundary reinforcement, shear rupture of the coating took place (Figure 14). In all other cases, the delamination and buckling of coating or strips was critical. The phenomena of premature delamination have been emphasized in the case of strengthening the walls with rigid CFRP strips/plates, directly glued to the masonry (Figure 15). Because of delamination, the displacement capacity was not significantly improved. Typical lateral load-displacement hysteretic relationships are shown in Figures 16 and 17.

Surprisingly, the strengthening of traditional three-leaf stone masonry walls by application of CFRP coating significantly improved both lateral resistance and displacement capacity of the tested walls. The efficiency did not depend much on the type of coating (vertically or diagonally placed CFRP grid in fiber reinforced mortar; GFRP fabric in epoxy resin matrix), but depended mainly on the method of application. Analyzing the test results, no indication can be obtained regarding the influence of damage state of the wall at the time of application of coating (previously damaged, undamaged) on lateral resistance and displacement capacity.

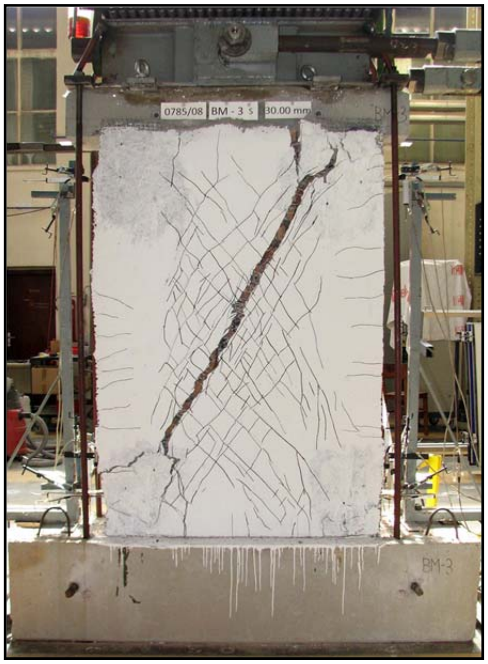

Figure 14: Brick Masonry. Diagonally Placed GFRP Grid Reinforced With Vertical Boundary Grid Strips [26]

Figure 15: Brick Masonry. Delamination of CFRP Strips at Ultimate State [26]

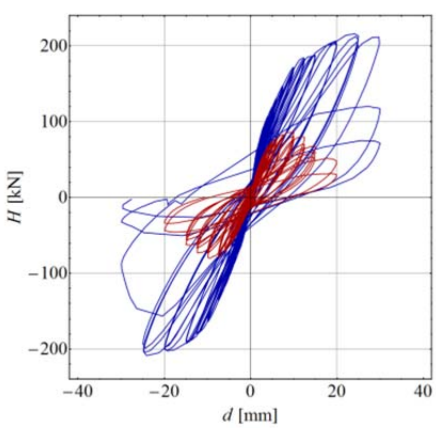

Figure 16: Brick Masonry. Diagonally Placed GFRP Grid Reinforced With Vertical Boundary Grid Strips Was Most Efficient. Red: Control Wall [26]

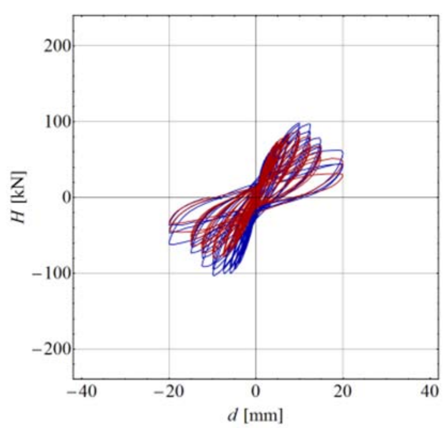

Figure 17: Brick Masonry. Strengthening With CFRP Strips Had No Effect. Red: Control Wall [26]

The application of coating on only one side of the wall, although anchored in the corners, improved the resistance to a lesser degree than the application of coating on both sides of the wall. In addition, such solution did not improve the displacement capacity. However, the way of anchoring the coating, at least in the corners of walls, is important. Typical damage pattern of a coated stone masonry wall is presented in Figure 18, whereas lateral load-displacement relationships for the same wall are shown in Figure 19.

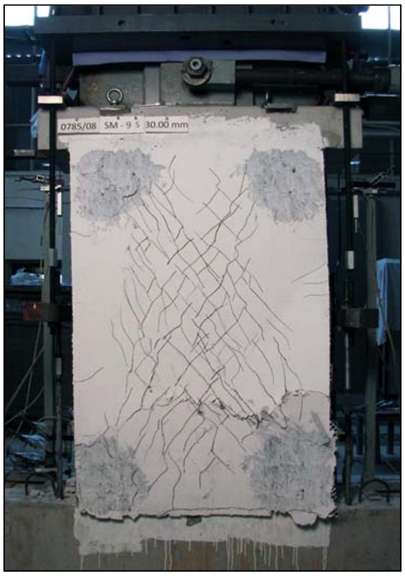

Figure 18: Stone Masonry. Cracks in the Coating at Ultimate State. Delamination Took Place at the Bottom [27]

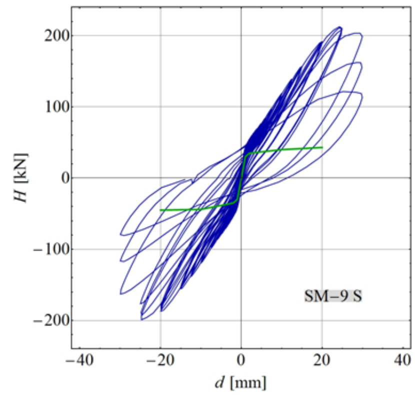

Figure 19: Stone Masonry. Strengthening With CFRP Coating is Efficient. Green: Control Wall [27]

CONCLUSIONS

Significant development has been made in the last decades in all aspects of strengthening and redesign of old masonry buildings. Although the basic principles did not change and the efficiency of traditional methods of strengthening has been verified in repeated seismic events, new methods have been developed for structural assessment, improved numerical models and techniques make possible reliable structural evaluation, new and improved technologies are available for structural strengthening. In addition, even a special standard for the assessment and retrofitting of buildings has been prepared as part of the family of structural Eurocodes.

Observations and analyses of damage to buildings in historic urban and rural nuclei after strong earthquakes have indicated that, by selecting adequate technical solutions and carefully executing the works, the required degree of seismic safety can be achieved. The number of cities and villages, where the repeated earthquakes in the last decades either confirmed the effectiveness of interventions or indicated the errors made in the retrofitting process, is increasing. Although the seismic resistance verification in accordance with the rules of the code may indicate that the seismic resistance is not sufficient, good seismic behavior of adequately retrofitted heritage buildings has been observed in most cases. The research results have been used to analyze recent code requirements as regards the seismic demand and structural resistance. It has been found that the requirements of the code are severe and would sometimes require unacceptable structural interventions. On the basis of the observed seismic behavior, experimental simulation and actual mechanical properties of masonry materials, obtained by in-situ and laboratory testing, modifications have been proposed, which would lead to more realistic code demands for architectural heritage buildings. The proposed modifications do not reduce the generally required safety against collapse, but will only slightly increase the level of the expected damage. To confirm the proposal, the seismic behavior of buildings, subjected to design level earthquakes for the second time in just a few decades, has been analyzed and correlated with the actually observed earthquake damage

Technologies and procedures for retrofitting are constantly improved and optimized. New materials are developed and new lessons are learned after each earthquake. However, the number of old masonry buildings in seismic prone cities and towns, vulnerable to earthquakes, is still large. Many buildings have been already renewed, however no intervention in the structural system to improve their seismic resistance has been carried out - despite repeated bad experiences after strong earthquakes. To implement the results of research into practice, awareness of seismic risk from the side of inhabitants and users, as well as political readiness to carry out preventive measures as well as to enforce the accepted rules and regulations in the case of building's renewal, is still needed.

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