THE PUBLISHING HOUSE
OF THE ROMANIAN ACADEMY

PROCEEDINGS OF THE ROMANIAN ACADEMY, Series A,
Volume 18, Number 2/2017, pp. 158–165

STRENGTHENING OF MASONRY SHEAR WALLS BASED ON METAL SOLUTIONS. PART II: FE-MODEL FOR THE PERFORMANCE EVALUATION

Adrian Ioan DOGARIU [1], Dan DUBINA [2]

[1] "Politehnica" University Timisoara, Steel Department and Structural Mechanics
[2] Romanian Academy – Timisoara Branch, Laboratory of Steel Structures
Corresponding author: E-mail: adrian.dogariu@upt.ro

Abstract. The strength and ductility of a proposed intervention technique for retrofitting masonry walls using metallic sheathing were experimentally evaluated. Due to the masonry-connector-plate system complexity, a detailed ABAQUS FE-model of masonry shear walls reinforced by metal sheathing, was set up. The numerical model was experimentally calibrated so as to replicate the behaviour of the retrofitted elements allowing parametric analysis. By using the finite element model, a "numerical experiment" was performed in order to set up the behaviour of different retrofitted wall types. An equivalent material able to replicate the numerical behaviour of the retrofitted model was applied in the global analysis. The seismic safety based on the performance approach was evaluated for a study case building. A multi-decisional matrix was proposed for the validation of the retrofitting techniques which combines structural, technical and economic aspects.

Key words: FE model, retrofitting techniques, performance-based seismic assessment, multi-decisional matrix, and numerical experimentation.

1. INTRODUCTION

Modern retrofitting philosophy encourages the new solutions to exploit the mechanical characteristics of different materials leading to mixed elements with a complex behaviour. The reversibility requirement limits the connection possibility between new and existing elements at ¨dry connections¨. These connections along with the mixed character of the work complicate the solution behaviour prediction [2].

Described in the first part of the article, the proposed solution [1] shows great deformation capacity improvement within the "structural damage"–"collapse prevention" range. The assessment of the positive effect of the intervention requires complex non-linear analysis and performance-based seismic evaluation tools without the possibility to apply rules of thumb. In addition to the element level improvement, the solution should be validated in the case of real case studies. The latest forty years have brought major developments in the field of numerical tools for structural analysis. In the field of existing masonry structures, there still are many shortcomings. The lack of data and scatter material behaviour are some of the difficulties that arise at numerical simulations. Nevertheless, useful information can be obtained from the numerical analysis.

Using local materials and simple technological procedures, brick masonry has been and still remains a popular material/technique for the construction of buildings in many areas of the world, including Romania.

Masonry structures are composed of transversal and longitudinal load bearing walls, which withstand both vertical and horizontal loads. Common earthquakes damage patterns can be classified in the following four categories: out-of-plane damage or collapse of the walls; in-plane shear or flexural cracking of walls; loss of walls to floor (or roof diaphragms) anchorage and damage or collapse of corners.

Depending on the horizontal load direction, the masonry panel shows different behaviour and different corresponding failure modes. A proper connection of walls and floors, commonly inhibit out-of-plane associated failure modes and the building laterally resist at earthquake actions, by the shear response of walls. The typical in-plane damage of the masonry walls subjected to earthquakes are the sliding failure, the flexural failure or rocking, and the shear failure. Shear diagonal is the most common failure mode. A nonductile failure characterized by a critical combination of principal tensile and compressive stresses resulting from combined shear and compression. The intervention strategy should select the effective retrofitting techniques against the most probable failure mode to occur. In most of the cases, old masonry buildings have shown weaker piers than spandrel. To get the most beneficial effect of the reinforcing technique, these areas are to be retrofitted.

1.1. Retrofitting strategies and performance-based assessment

Applying a strengthening technique is a multi-criteria task. Structural, technical, cultural, social, economic, and sustainability aspects are involved. The designer has several solutions at his disposition. They need to select the one which best matches the criteria of validation. The solution will represent a rational compromise among various criteria [4].

From the structural point of view, the intervention strategy should decide between increasing the strength and enhancing the deformation capacity, or a good balance of both. The effort to increase the resistance commonly leads to an increase of stiffness, so increasing seismic loads and demands. The major problem of masonry is the deformation capacity, hence the most proper solution in the case of masonry structures is to enhance the deformation capacity in order to get dissipation [4].

The role of Performance-Based Seismic Assessment (PBSA) is to propose methods for evaluating the building's ability to achieve the requested performance. When affected by earthquakes, structures should still sustain a certain amount of damage. Basic performance objectives are being adopted by performance-based engineering guidelines nowadays. These quantitatively defined parameters are expressed as acceptance criteria and are defined for almost all types of building typologies and component member. The acceptance criteria defined in FEMA 356 establish nonlinear relations force – displacement [7].

PBSA introduces multiple performance goals unified into a single Building Performance Objective (PBO). PBO appropriates the Building Performance Level with Seismic Hazard. The designer can show performance level for the structural and non-structural elements at a certain level of earthquake movement. Through different analysis procedures, acceptance criteria achievement needs to be checked. The criteria for the selection of the analysis method and of the specific requirements are available in most seismic design standards. The principles proclaimed above are valid for all typologies, old masonry buildings included.

2. FE-MODEL EXPERIMENTAL CALIBRATION

2.1. Finite element strategies for masonry elements

Various FE software are employed in order to make sophisticated simulations of the structural behaviour. Material and geometrical non-linearity can be accounted in the computation process. A description of the material behaviour, i.e. stress and strain tensor relation in a material point of the body, is necessary for this purpose. This mathematical constitutive law is the object of many research studies. An important aim is to get robust numerical tools, capable of predicting the behaviour of the structure from the elastic domain until total failure, due to excessive cracking and rigidity degradation, when brittle material is involved.

If the continuum model approach for nonhomogeneous materials like masonry is used, the following levels of approximation might be applied: micro-models (simplified or detailed) and macro-models [10].

A micro-model represents each component, capturing all the local failure modes of masonry, i.e. cracking or sliding on the contact surface, tensile cracking of the mortar or brick units, and crushing of masonry. When using a micro-model, two approaches are possible. First, a simplified micro-model or layer model, which doesn’t take into account the brick unit elements and the mortar elements interface (friction law); the second is a detailed or “interface” model, which delineates the normal and tangential contact surface instead of/or together with mortar layers. This modelling technique, even for small elements, will result in huge models. The reliability of the results is doubtful whether the input data are not really correct.

A simplified macro-model assumes an anisotropic continuum model. The constitutive law is based on average stresses and average strain relations, considering composite masonry as a homogeneous equivalent material. Components aren’t represented anymore and their failure modes aren’t caught.

Accounting for the nature of masonry, a discontinuous modeling based either on smeared crack or more complex discrete crack or interface smeared crack approach should be consistent [9]. The joint elements are the most appropriate to model the non-linear character of the masonry. Relative slip, debonding and opening and closing of the interface are shown, but aren't suitable for analysis of elements or entire structures.

The smeared crack approach considers the cracks and joints by modifying the material characteristics at the integration points of the finite elements. Smeared cracks are suited when the crack orientations aren't known beforehand. Cracks don't involve changes in the mesh or the introduction of new degrees of freedom. The method is recommended for the analysis of large-scale masonry elements or structures. It treats masonry as an equivalent homogeneous material. Joints and cracks are smeared out without differing bricks from the mortar joints. The smeared crack approach limitation is that discrete cracks are smeared out over an entire element. The crack opening is modelled by the continuous displacement approximation functions of the conventional finite element approach.

2.2. Description of the adopted numerical model

A detailed numerical model was built, calibrated on the experimental results. The model describes the behavior of both, the unreinforced masonry (URM) wall and the strengthened one. The steel plates were connected by chemical anchors. Referable to the proportions of the experimental specimen, a macro-model with 3D Deformable Solid (an 8-node linear brick) finite element has been applied. This approach requires an equivalent homogeneous material in order to model the masonry global behavior. The material parameters must be selected so as to reproduce the global behavior. The wall resistance and the deformation capacity are considered, along with the local behavior of the area around the connectors.

Various material models for brittle behavior available in ABAQUS Library [6] were studied. First, a "concrete smeared cracking" (CSC) model was applied. A good correlation was obtained for the unreinforced masonry panel. For strengthened masonry panels, the "CSC" model wasn't capable of obtaining the large ultimate displacement. Further, "Brittle cracking" (BC) and "Concrete Damage Plasticity" (CDP) [6] were considered. One of the advantages of these models is the possibility to use a dynamic formulation of equilibrium. This offers quicker results and a more robust numerical tool. For the "BC" model, the failure criterion is introduced in the tension range, considering an infinite compressive strength. This material model allows for very large displacement and the failure mode of the numerical masonry model is similar to the one observed in the experimental tests. Unfortunately, mesh instability occurs due to a very high energy unbalance in the post cracking state of the model.

"CDP" is used in the analyses of concrete elements. The brittle behaviour of masonry, along with the cracks development, can be simulated with full accuracy. The material model provides both crushing and cracking, but it doesn't introduce shear retention assumptions. A quasi-static analysis with an explicit solution has been performed (Fig 1). The mesh keeps a good stable state (Fig. 2) and doesn't produce energetic imbalances [1, 2, 5].

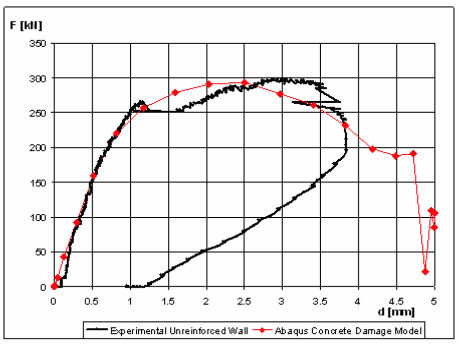

Fig. 1 – Experimental and numerical global behavior of URM.

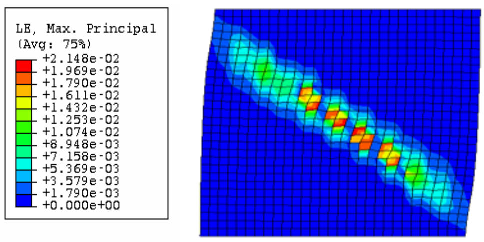

Fig. 2 – Shear diagonal failure mode of URM.

For the retrofitted masonry panel, the chemical anchor was modeled in a sequence of constraints. A nonlinear spring element (Fig. 3) was used. Nodes P1 and P2 were linked by a planar connector [6] with a constitutive law derived from experimental push-tests made on the chemical anchors (CA).

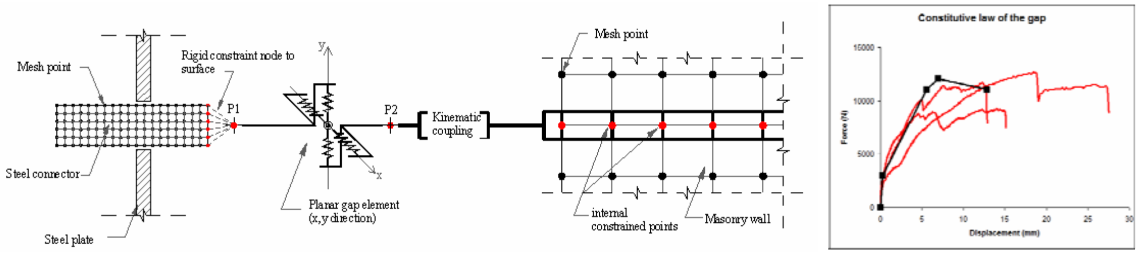

Fig. 3 – Numerical definition of chemical anchors and experimental based multi-linear constitutive law of CA.

The adopted numerical model exhibits a large ultimate displacement and the failure mode of the masonry wall reflects the observed experimental shear failure. The behaviour of the retrofitted model is described by a force – displacement curve (Fig. 4). The model showed a diagonal tensile failure mechanism (Fig. 5). The metal plate works at low stresses except areas around steel anchors [2].

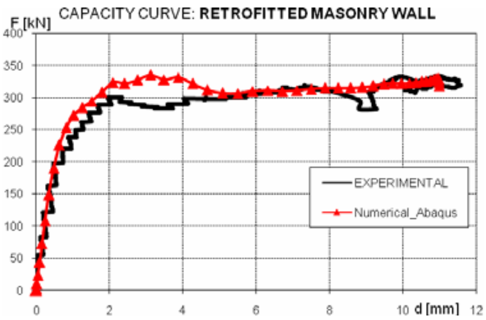

Fig. 4 – Experimental and numerical global behavior.

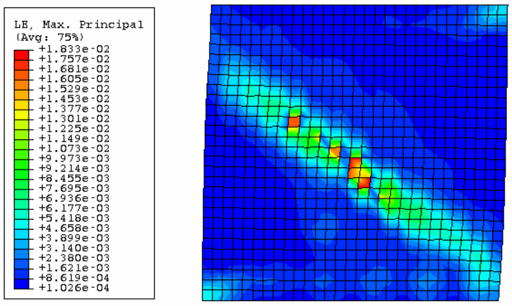

Fig. 5 – Diagonal tensile failure mode of reinforced wall.

Therefore, this FE model could be used to perform a parametric study. The aim is to check the effect of the main parameters involved in the design of the retrofitted solution. An “FE numerical experimentation” could prove the effect of this retrofitting technique on different types of masonry [5].

3. PERFORMANCE BASED EVALUATION OF A MASONRY BUILDING

3.1. Description of the masonry structure

A masonry building designed without anti-seismic criteria, selected as a reference benchmark structure within the framework of RFCS STEELRETRO [3], is reinforced and analysed. An intensity of 0.24g of PGA and type B soil has been considered. The building, composed of ground floor and two floors, is symmetrical in the horizontal plan and elevation. It has small and well-positioned openings, with an almost cubic shape of 15 m. The wall thickness varies on height from 350 mm to 650 mm. Stone masonry with a mean compressive strength f_m = 1.5 MPa, Young Modulus E_m = 1 500 MPa and unit weight w = 21 kN/m^3, is used for walls.

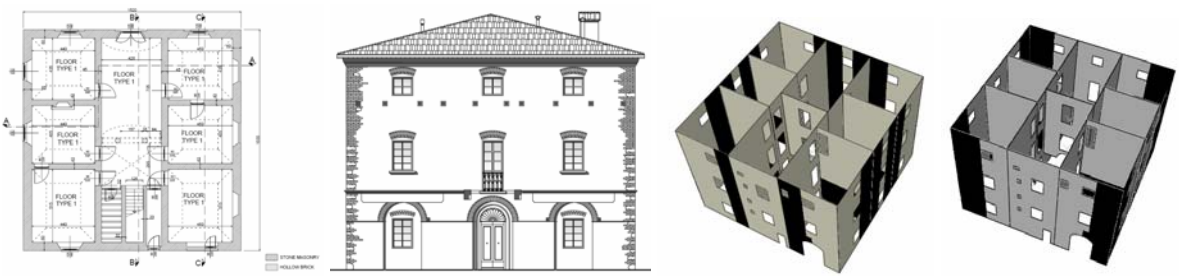

Fig. 6 – Horizontal section and main façade of the masonry building and placing of the reinforcing elements.

In order to apply the metal sheathing retrofitting to the walls, it was assumed that measures providing the diaphragm effect of floors and roof and the integrity of the wall junctions have been taken.

The building façade was reinforced on the entire height of the building. Two position possibilities were taken into account for external walls, as shown in Figure 6, simultaneously, with all the internal transversal ground floor walls. The applied techniques attempt to be minimal one and avoid affecting internal walls to not disturb the building occupancy.

3.2. Nonlinear model and specific acceptance criteria

A proper application of PBSA needs a reliable nonlinear analysis model in order to perform advanced analysis in displacement control. The ABAQUS numerical model calibrated on experimental results can fulfil this demand.

Performance based approach considers the structure as an assembly of its individual components. The building performance level should be defined in relation to its element performance. The evaluation of the effects of the damage on building performance must concentrate on how component properties change as the result of damage. The response of components is controlled by force – deformation properties (e.g. elastic stiffness, yield or cracking point, ductility, and ultimate deformation).

Damage occurrence affects the behaviour of elements in different ways. The behaviour of a wall depends on its relative strength in flexure to the one in shear. Cracks and other damage must be judged depending on the behaviour mode of each specific part.

A safety evaluation should consider the cracks width, placement, orientation, and their number and distribution pattern. In a simple way, cracks width is commonly used in order to decide the damage level and the performance of the wall. The performance acceptance criteria were established on the retrofitted wall panel model in connection with plastic strain at a certain performance level. The reinforced panel numerical model fails due to compressive load by crushing of masonry. If the unreinforced model fails at a level about 0.15% of plastic strain, the tensile strain in shear diagonal strip in the retrofitted models allows for reaching more than 3.5% strain before collapse prevention level and failure (Fig. 7).

The global behaviour curves (Fig. 7) come to sustain the possibility to enhance the deformation of the wall and prove suitability to apply the performance levels presented in Fig. 5, showing the benefit of the applied reinforcing.

To establish the performance levels, a parametric study has been conducted. A “numerical experimental procedure” was performed (Fig. 8). The drop in the complex numerical model behaviour (dotted lines in Fig. 8) is due to the discontinuity introduced by the gap between the connectors and metallic plate. The effect of the retrofitting solution for an existing masonry, with the mechanical characteristics presented above and wall thickness from 350 to 600 mm, was established [3].

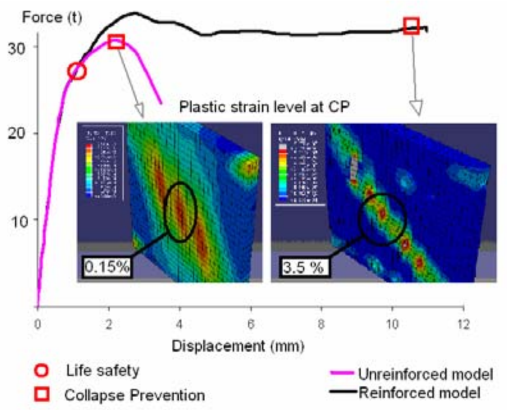

Fig. 7 – Performance criteria for un- and reinforced wall.

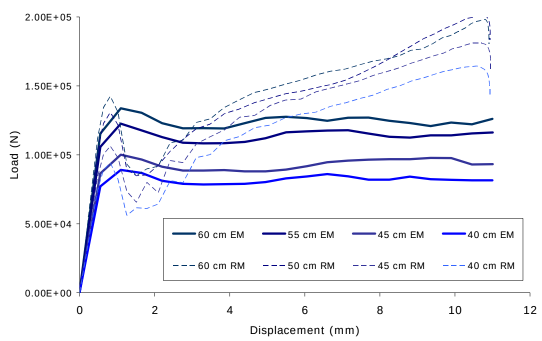

Fig. 8 – Effect of retrofitting for advanced model and simplified equivalent material.

Experimental tests on wall specimens have shown the behaviour improvement in the range of “Life Safety” (LS) – “Collapse Prevention” (CP), accompanying by a strength increasing. These results allow to use an equivalent material model for masonry. Removing the brittle component model by tension softening, the same global behaviour as in the case of "numerically tested" retrofitted specimens was obtained. This fact simplifies the numerical effort a lot. A simple change in the original material parameters replicate the effect of reinforcing (Fig. 8), allowing to apply this procedure for global analysis of real various building types. The retrofitted wall model reached at CP level 2.5% ultimate maximum plastic strain and –0.7% ultimate minimum plastic strain. In case of the equivalent model, at the same displacement of 10 mm, corresponding to a 1/150 drift, it was recorded 1.5% ultimate maximum plastic strain and –0.07% ultimate minimum plastic strain. These values will be used in the further evaluation as reference criteria.

3.3. Numerical analysis of existing building

A spatial model of the building using 3D shell elements and a material model of CDP has been built. Simplifications on the fixed base and rigid diaphragm behaviour of the floors has been made. The horizontal load was introduced quasi-statically, performing an explicit analysis. According to the first vibration mode, a triangular shape of forces were concentrated in the mass centre of the floors. The results of the pushover analysis are plotted in terms of base force – top displacement on both directions (Fig. 9). No torsional effects are expected for such a symmetric configuration. Generally, after reaching the maximum force, the masonry building behaves as fragile, losing much of the strength at small displacement. To establish the seismic response of the initial and the retrofitted structure, the procedure described in EN1998-1 was used [8]. The target displacement of 6.76 mm has been determined for the SDOF model at the capacity curve and inelastic spectrum intersection (Fig. 10). A constant ductility of 1.5 was considered.

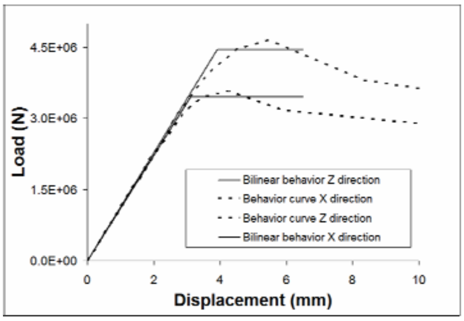

Fig. 9 – Numerical global behavior of the unreinforced building.

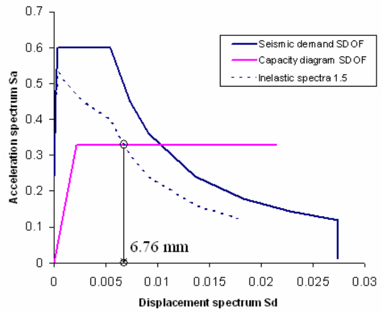

Fig. 10 – Demand spectra and capacity diagram (SDOF).

The damage level and evidence of the performance criteria attainment at 9.77 mm target displacement (Fig. 12) for the unreinforced model with regard to plastic strain is plotted in Fig. 11. One remark in the case of the unreinforced model at the level of the ground floor, all the diagonal cracks appear in between the openings and exceed the CP value of the plastic strain (0.15%).

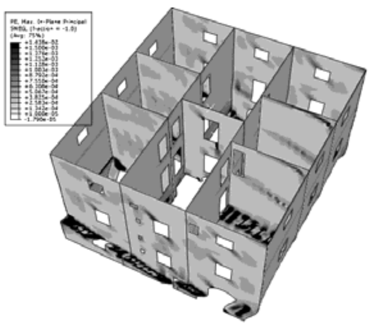

Fig. 11 – Plastic strain in unreinforced building.

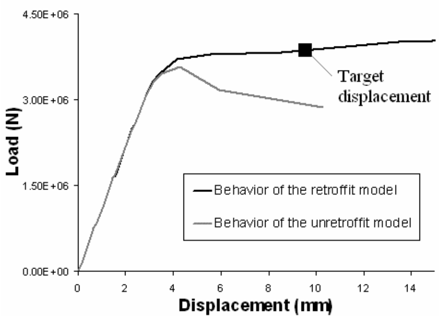

Fig. 12 – Behavior of unreinforced and retrofitted structure.

At 9.77 mm target displacement attained by the retrofitted model, similar to the unreinforced building, the level of the reference plastic strain is exceeded in the unreinforced walls, but not in the reinforced ones (Fig. 13b and Fig. 14b). Can be concluded that even if in the adjacent unreinforced walls the failure occurred, the reinforced walls keep the global safety of the building, by maintaining the level of strength.

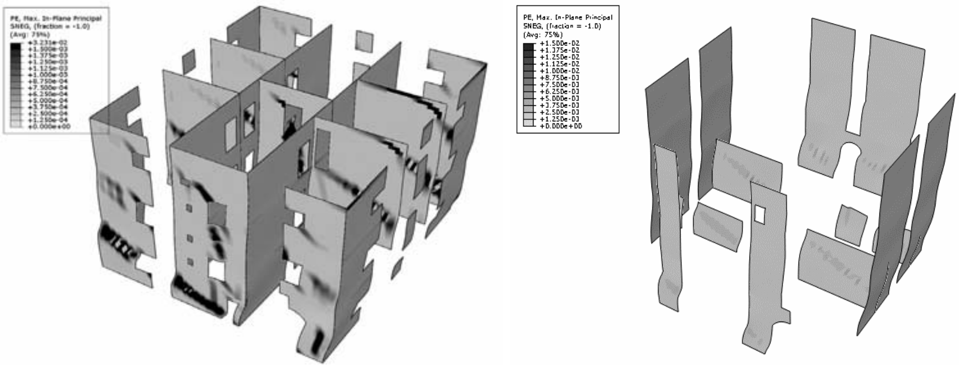

Fig. 13 – Damaged pattern for the reinforced building – unreinforced and (middle) reinforced walls.

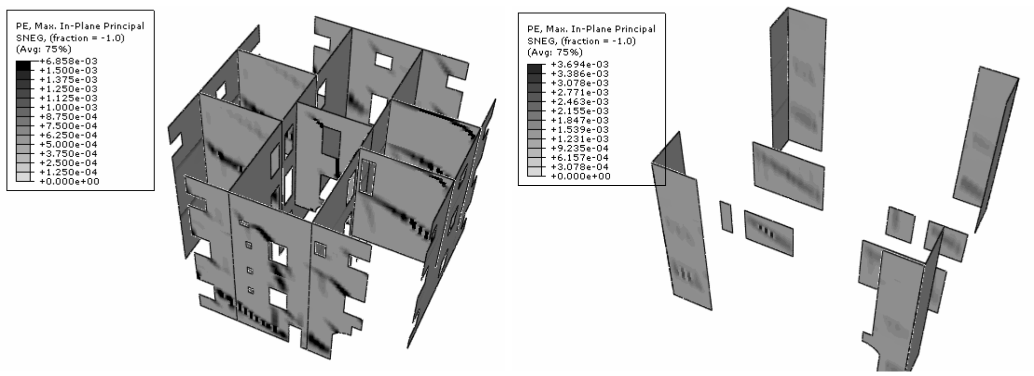

Fig. 14 – Damaged pattern for the reinforced building – unreinforced and (corner) reinforced walls.

The retrofitted building subjected to a seismic motion of PGA up to 0.16g behaves elastic and fulfils the IO performance level; for PGA between 0.16–0.44g, the LS performance level is attained. At a displacement larger than 30 mm, the building reaches CP level. Using the recurrence formulas for PGA given in the Romanian Code P100-3 [11], calibrated for the Vrancea earthquake, a matrix may be set up, showing the performance objective possible to meet by retrofitted building (Table 1) [3, 4].

Table 1

Performance objective

PL/IMR	30 y	50 y	100 y	225 y	475 y	975 y
PGA	0.072g	0.168g	0.24g	0.288g	0.36g	0.48g
IO – Immediate Occupancy	o					
LS – Life Safety		o	o	o	o	
CP – Collapse Prevention						o

However, to confirm the equivalent material simplifications operated in this case, for global analysis it is needed to extract the areas of important plastic strains concentration and to perform, using the advanced numerical model, a new local analysis, respecting the geometry and boundary condition [1].

4. CONCLUSIONS

Within the study, an FE model of the reinforced masonry panel was calibrated. The model replicates the experimental behaviour, allowing for a “numerical experiment”, in order to test the behaviour of the various types of masonry structures retrofitted with metallic plates based on the acceptance criteria and the performance requirements. As a simplified approach, an equivalent material that replicates the experimental results has been used for a global non-linear evaluation analysis on a real façade or the entire building, in order to assess the damage at certain seismic demands, based on the acceptance criteria established before.

The retrofitting solution has showed good behaviour, by preserving the initial capacity and allowing for considerable ultimate displacement of about 0.7% drift ratio, which corresponds to the collapse prevention level of the building.

In the end, a Decisional Matrix (Table 2) for the scoring of the intervention technique, could be applied. According to such matrix, even better intervention techniques are possible, but, nevertheless, the one applied in the present study is good enough [3].

Table 2

Decision matrix

		L	M	H	Mark
Structural aspects	Capability to meet requested performance objective (after building evaluation!)			O	9
	Compatibility with the real structural system (no need of complementary strengthening or confinement measures)		O		
	Adaptability to change of actions seismic typology (near field, far field, T<>Tic, etc.)			O	
	Adaptability to change of building typology			O	
Technical aspects	Reversibility of intervention		O		8
	Durability			O	
	Operational			O	
	Functionally and aesthetically compatible and complementary to the existing building		O		
	Sustainability		O		
	Technical capability			O	
	Technical support (Codification, Recommendations, Technical rules)		O		
	Availability of material/device			O	
	Quality control			O	
Economic aspects	Costs (Material/Fabrication, Transportation, Erection, Installation, Maintenance, Preparatory works)		O		8

Legend: L = low, M = medium, H = high, Mark – L (5-6), M (7-8), H (9-10)

ACKNOWLEDGEMENTS

This work was partially supported by the strategic grant POSDRU/159/1.5/S/137070 (2014) of the Ministry of National Education, Romania, co-financed by the European Social Fund – Investing in People, within the Sectoral Operational Programme Human Resources Development 2007–2013.

REFERENCES

1. DOGARIU A., *Seismic retrofitting techniques based on metallic materials of RC and/or masonry*, Politehnica Timisoara, 2009.
2. FP6 EC PROHITECH, *Numerical Analysis* (Vol. 4) and *Calculation models* (Vol. 5), Edit. Polimerica, 2012.
3. Publication Office of the European Union, *Steel solutions for seismic retrofit and upgrade of existing constructions* (*Steelretro*), Final Report (KI-NA-25894-EN-N), 2013.
4. A. DOGARIU, D. DUBINA, *Performance based seismic evaluation of a non-seismic masonry building of metal sheathed walls. Part I: PBSE and intervention strategy*, and *Part II: Study case*, Intl. Conf. PROHITECH09, 2009.
5. A. DOGARIU, F. CAMPITIELLO, Calibration *of a FE Model of Masonry Shear Panels strengthened by Metal Sheathing*, F-and-B ’10, Bucharest, 2010.
6. *** Dassault Systemes, ABAQUS 6.14 documentation, 2014.
7. Applied Technology Council, *Prestandard and commentary for seismic rehabilitation of buildings FEMA-356*, 2000.
8. European Committee for Standardization (CEN), *Eurocode 8: Design of structures for earthquake resistance*, 2003.
9. Rots J.G., *Numerical simulation of cracking in structural masonry*, Heron, **36**, *2*, pp. 49–63, 1991.
10. Lourenço P.B., *Computational strategies for masonry structures*, Delft University Press, 1996.
11. ASRO (Romanian Standardization Assoc.), *P100-3 Rules for evaluation and retrofitting of existing building*, 2008.

Received November 3, 2015