Assessment and Improvement of Seismic Performance of the Masonry Bearing Building Stock in Albania

MSc Merita Guri[1]
Department of Applied and Human Science,
Faculty of Architecture and Design
POLIS University
Tirana / Albania

MSc Diana Lluka[2]
Department of Building Constructions and Transport Infrastructure, Faculty of Civil Engineering,
Polytechnic University of Tirana
Tirana / Albania

MSc Egla Luca[3]
Department of Applied and Human Science,
Faculty of Architecture and Design
POLIS University
Tirana / Albania

Abstract:- **In Albania, as a country with high seismic risk, it is very important the design and seismic evaluation of buildings. From the economic perspective there are two opsions: repair or demolition. The opportunity to choose is given by the assessment of their actual condition. In the new and old Albanian design codes, there is no specified procedure about the seismic performance of the existing buildings. This situation becomes even more serious when considering the degradation over the years and structural interventions. For this reason, it is necessary to assess and improve the seismic performance of the choosen typology projects of residential buildings with load bearing masonry in Albania, which are designed in accordance with [KTP 78, 1978; KTP-89, 1989], based on contemporary theories as EN1996, FEMA 440, ATC40.**

For the improvement of the seismic performance of this type of residential stock with load bearing masonry, we will focus on polymer materials such as FRP (Fibre-reinforced polymers). More specificlly on Glas Fibre-reinforced polymers GFRP.

Key Words : Load bearing masonery buildins; Seismic reinforcment; Seismic performance; FRP; GFRP; KTP-89; EC-8; EC-6

I. INTRODUCTION

Albania is one of the most prone to seismic oscillations in the Balkans [1]. Recent devastating earthquakes in neighboring countries[1] have shown that masonry buildings have suffered maximum damage and are responsible for the loss of life. Due to reasons such as age, interventions made by people and the design code of the time, these types of buildings are vulnerable to earthquakes. It is therefore important to evaluate the seismic performance of these buildings and on the basis of this assessment techniques must be developed to strengthen these buildings in order for them to resist potential earthquake damage [2].

The chosen type building to analyze has the code 74/4 according to construction archaive [3]. The elected building has no anti-seismic columns. Therefore they are more vulnerable to seismic action [4]. It has been 40 years since the design and construction of the building type 74/4. This time undoubtedly has contributed to the degradation of masonry and reduces its load bearing capacity. As a type building, it can be found in many cities across the country. Consequently it may be subjected to different climatic conditions and may have suffered various degradations. This degradation also depends on human activity which may have adversely affected by interfering in the structural elements. **The aim of this study is to analyze the building type 74/4 assuming as hath undergone no alteration or damaged over time.** With this analysis we estimate the type project 74/4 to the action of seismic spectrum of the EuroCode and that of the KTP-N2-89. The masonry was modeled with nonlinear behavior. The modeling was carried out in the program SAP2000. The masonry is designed with two layers representing its behavior in compression and shearing. For the compression behavior is used graph Kaushik (2007), and for the shearing behavior is used the ideal bilinear graph in accordance with maximum resistance in shear as cohesion between the mortar and brick. Furthermore is explained the performed nonlinear analysis and the results processed by FEMA 440. The service conditions are taken from a study from Calvi (1999). This methode calculates them as proportionate to the relative displacement of the interstory, describing the damage to the building pretty well in the local and global level from seismic shear forces [5]. These results further elaborated by FEMA440 to find the performance point of the structure. To increase the carrying capacity of the building type 74/4 are used the reinforcment techniques against global collapse mechanisms [6], applying the reinforcement layer on the outside of the building sites for the three bottom floors. Computer analyzes were conducted respectively for two situations: without reinforcement and FRP reinforcement. The conducted analysis "pushover" analyze. Capacity curve was developed together with seismic spectrum to find the performance points.

II.DESCRIPTION OF STRUCTURE

A.Technical specifications and loads.

The constructive project and technical specifications for the 74/4 building are taken from the technical archives office in Albania (figure 1). Loads for slab are taken 6.94 kN / m2 -7 kN / m2. The bricks are M75 with a resistance of 7.5MPa and mortar of M25 with a resistance of 2.5MPa. The load bearing wall thickness is 380mm on first two storeys and 250mm on the remaining three storeys.

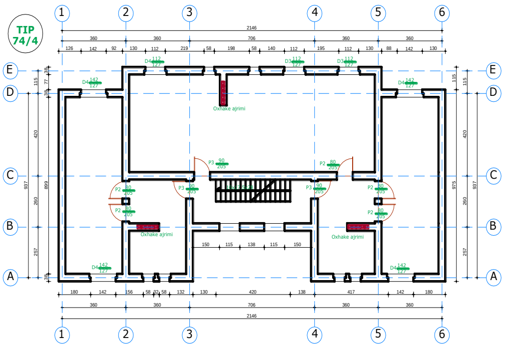

Figure 1 –The floor plan of the building type 74/4

B.Seismic spectrum.

Since the objective of this research is to evaluate the seismic performance of the building and improve it, we will consider two types of spectrum: KTP-N2-89 which is in use and Eurocode 8 which is not approved yet (figure 2)

a) Seiscmic spectrum according to KTP-N2-89

Given that type buildings in Albania, as it 74/4 that we are studying, are built almost in the whole territory, they are exposed to different seismic risk.

This is because Albania has a variety of seismic intensity from 7-9 degree. To represent the majority of the buildings type 74/4, it is reasonable to choose the seismic intensity that has the greater surface in the seismic map of Albania. This is the intensity 8. Furthermore the category of soil will be average, with the same logic as above. The ductility will be taken 1 because in the nonlinear analysis this role is played by the material curve.

Eventually we have:

- Soil category: 2 (medium)
- Seismic intensity: I=8.
- The ductility: $\psi = 1$ (the spectrum is elastic)

B.*Seiscmic spectrum according to EC-8*

The EC - 8 have another way to calculate the seismic spectrum. In this case it depends on several factors: peak ground acceleration PGA, the category of the soil, the predicted magnitude (in the case of our country is M> 5.5) and the behavioral factor. This last one as a concept is comparable to the inverse of ductility. Given the studies, Albania has a variety of seismic peak ground acceleration from 0.15-0.3g. With the same reasoning as above, we choose the seismic acceleration that has greater surface in the seismic map. Eventually the selected parameters are:

- Soil category: C
- Spectral acceleration: 0.2g=1.96m/s2
- Behavioral factor: 1 (the spectrum is elastic)

Both spectra are built on the same graph to compare with each other.

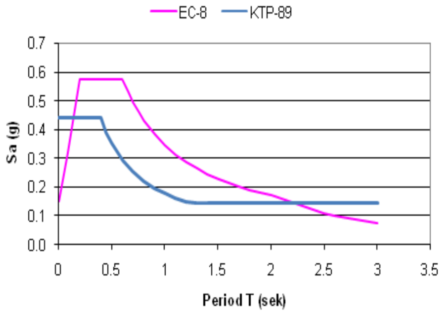

Figure 2 – Elastic response spectra

III. MATHEMATICAL MODELLING OF REPRESENTATIVE BUILDINGS

Computer calculations will be performed with finite element program SAP2000. To model the masonry we have referred to the study [7] which is using the element type "shell" consisting of layers with nonlinear behavior. Layers will represent the properties of masonry in axial compression and shear strengths.

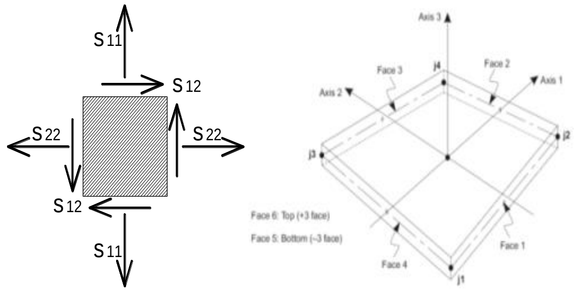

Figure 3- Plan element with 4 knots and strains in plan. (Sap2000 reference manual)

The masonry behavior will be modeled by two different stress-strain graphics. They will represent the vertical stresses S11, S22 horizontal stresses and shear stresses S12. These behaviors are in fact the main ones describing the masonry material.

It is very important to predict the best possible stress - strain graph for each direction [7].

IV. NONLINEAR MATERIAL PROPERTIES MASORNRY

A.Nonlinear Modeling of masonry.

a) Stress - strain graphs for S11 and S22 directions.

This behavior is defined by researchers as Kaushik (2007) (figure 4) based on many laboratory tests. We present the following calculations.

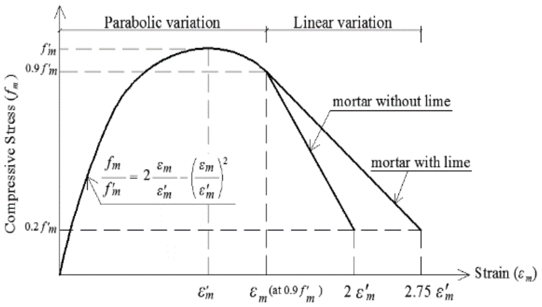

Figure 4 – Stress - strain graph for axial compression deformation (Kaushik, 2007)

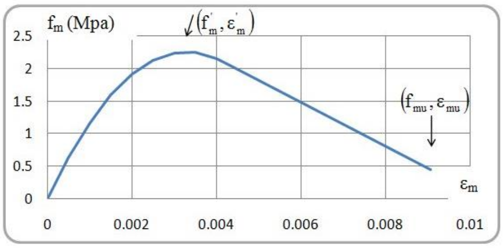

Figura 5 – Stress - strain courve for axial compression (used for S11,S22)

b) Stress - strain graph for S12 direction.

This curve represents the nonlinear behavior unto destruction in the horizontal direction of a masonry element. According to the literature when the masonry is subject of the terrain horizontal movement (earthquake) horizontal resistant force is represented by the cohesion and the friction between the brick and mortar.This force is shear stress of Mohr-Coulomb:

$$\tau = c + \sigma \cdot tg\varphi \qquad (8)$$

In this equation, "σ" is the vertical stress and tgφ represents the friction between elements. It should be noted that in order to activate the sliding friction it should be destroyed the cohesion between elements.

The equation expresses behavior of interdependence between the vertical stress and friction

Different design codes have different values of cohesion which serves for the graphic material for the S12 direction. Albanian code has a cohesion value which depends on the mortar marks. According to the Albanian code KTP, mortar resistance 2.5 MPa, the cohesion is c = 110 kPa [6]. Eurocode 6 has two alternatives for the resistance to shearing and is choosen the minimum of two [8] [9].

$f_{vk0} = 200kPa$, for the mortar resistance f_m=2.5MPa.

Or $f_{vk0} = 0.065 \cdot f_b = 0.065 \cdot 2267 = 147kPa$

Seeing that the Albanian code has the lowest value, it is more appropriate to consider that value (110 kPa). Also over time this value could be even lower due to the degradation of buildings. According to EuroCode 6 the sheare module can be 40% of the module of elasticity.

E=1247 Mpa, so: G=0.4*E=499 Mpa (1)

$$\varepsilon_{el} = \frac{\tau_{el}}{G} = \frac{0.11MPa}{499Mpa} = 0.00022 \qquad (2)$$

The idealized graph of shear strength S12 is given below:

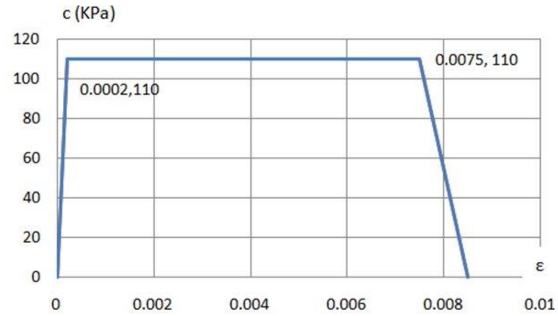

Figure 6 – Stress - strain graph for shear (for S12)

B.Nonlinear Modeling of FRP.

SAP2000 reinforcements will be modeled as a layer which overlaps on the outside of the masonry. For this will be used the existing modeling of the masonry whith nonlinear layers. To the layered element we will add a layer representing the reinforcment. Since the reinforcements are of various types, there is a wide range of options in the selection of reinforcements [10]. We will concentrate on polymers reinforced with glass fiber GFRP. This layer of glass fiber is applied with epoxy adhesive to the masonry. The behavior is elastic fragile and works only in tension. Once the material reaches the maximum in tension it breaks. Same as above we should drop off to zero the carrying ability as they are weaned. Strengthening with GFRP will be modeled using the material curve. The behavior is linear and fragile, anyway it is not expected that the reinforcement to be destroyed before the masonry . This is due to their permitted deformation. Seeing the materials curves, we note that the masonry collapse after 0.75% deformation, glass fibers after 1.69% deformation. Note that you can not use all the carrying capacity of the reinforcement, because the masonry is destroyed earlyer. According to the M. R. Valluzi study 2012, the equivalent thickness is 0.12mm. The same study also gives details on the carryig ability in tenssion (1310 MPa) and the maximum deformation (1.69%). With these data was built the following graph. [11].

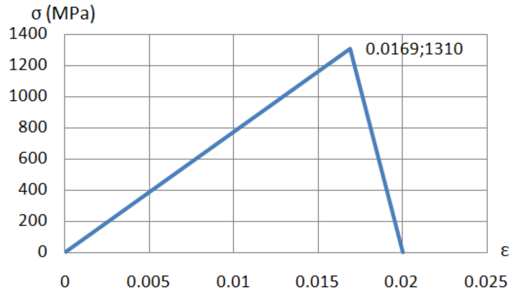

Figure 7 – Stress - strain courve for glass fibers GFRP

V. NONLINEAR ANALYSIS FOR THE BUILDING TYPE 74/4 CAPACITY CURVE OF THE STRUCTURE

To perform the nonlinear analysis type "pushover" of this model is considered the model of loading it according to

the first two quakes modes that correspond to two main directions of the structure.
The first moda results along the direction + Y or the short direction of the building. The period for this mode is T = 0.70s. The second moda results along the direction -X or the longist direction of the building. The period for this mode is T=0.33s. So the two nonlinear analyzes were performed for these two models of loading. For the displacement monitoring point, is selecting a point on the terrace of the building. Below we present the results of the nonlinear analysis.

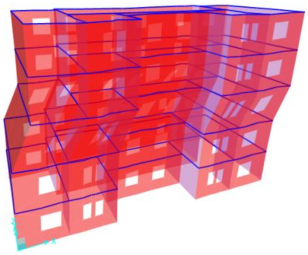

Figure 8 –Maximum nonlinear displacement according the X direction.

In order to determine the boundary service conditions according to Calvi [5], it is necessary to get from program the relative interstory drifts. They then are compared with the limit values as described above. Since the height of one floor is 3.06m, then we have:
LS2 =0.1% * 306 = 0.31 cm
LS3 =0.3% * 306 = 0.92 cm
LS3 =0.5% * 306 = 1.53 cm

Below is showed graphically the interstory drift. For convenience the ordinate axis is scaled by 0.1% displacement. The horizontal axis shows the steps of the nonlinear analysis unto destruction of the building.

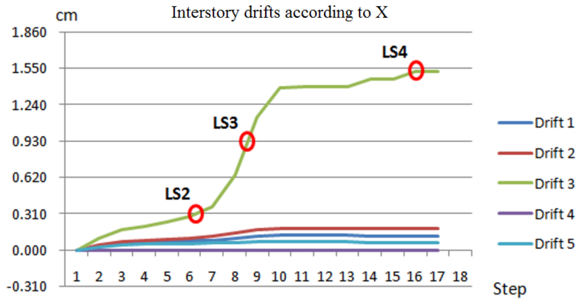

Figure 9 – Interstory drifts according to X.

Then the boundary position appears in the capacity curve. It is observed that the interstory drifts are generally small except the 3rd floor. This floor is critical because there we have a reduction of the wall thickness from 38 to 25cm. It seems fairly obvious in Figure 9. In order for the values to become more tangible, we estimate with interpolation (where necessary) the absolute displacements and the base shear force for each service position. These values are added at the capacity curve shown below.

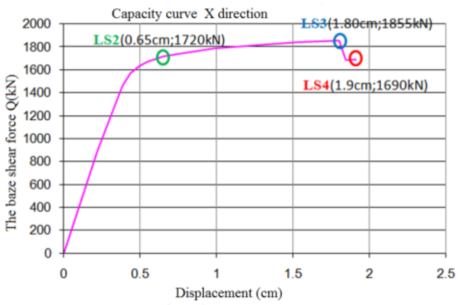

Figure 10 – Capacity curve in the limit conditions.

The Y direction has another capacity curve due to asymmetry of the building. Although the baze shear force is approximately equal, the final movements are not the same. 3rd floor again results as the weak floor.

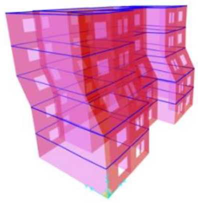

Figure 11 – Maximum nonlinear displacement according the Y direction.

Below is showed graphically the interstory drift.

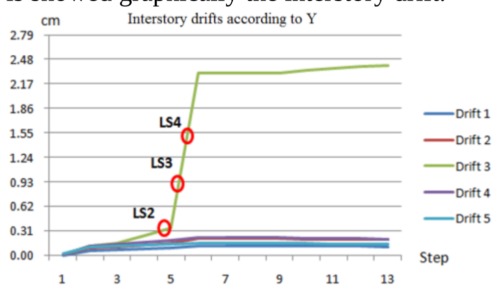

Figure 12 – Interstory drifts according to Y.

As expected, judging by the form of the deformation at the time of destruction, there is a weak 3rd floor. All other floors do not undergo plastic deformation as the relative displacements are less than 0.1%. Also in this direction plays an important role the reduction of the section from 38cm to 25cm on the 3rd floor.
Service conditions are as follows:

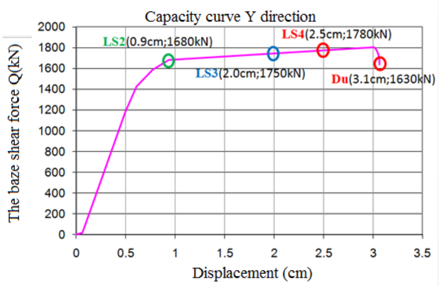

Figure 13 – Capacity curve in the limit conditions.

Discussion on the results of the analysis for the capacity curve For the 74/4 tip building we see a almost balanced behavior in both directions. While shear force is almost the same for both directions, we see a greater ductility in the Y direction. This is reflected in the biggest displacement in the moment of destruction for the Y direction. In Table 1 we present the results of the nonlinear analysis.

Table 1. Results of the analysis for the study case without reinforcement

	LS2		**LS3**		**LS4**		**Du**	
X	0.65 cm	1720 kN	1.8 cm	1855 kN	1.9 cm	1690 kN	1.9 cm	1690 kN
Y	0.9 cm	1680 kN	2 cm	1750 kN	2.5 cm	1780 kN	3.1 cm	1630 kN

The 74/4 type buildings destroyed by the shear seismic force not on the ground floor but on the third floor. The reason is the immediate reduction of the thickness of the load bearing wall. As noted from the interstory drifts, the third floor suffered major damage from horizontal forces, while the other floors are almost not damaged at all. This phenomenon is the same for both directions X and Y. This is evident in 3 dimensional view of the model at the point of destruction (figure 11).

VI. SEISMIC PERFORMANCE OF THE BUILDING TYPE 74/4 WITH EC-8, KTP

Determination of the performance point of the structure according to FEMA440

Below we present the performance points of the building in two directions X and Y (figure 14, 15). At the same graphic are featured the performances of both spectra KTP and EC - 8 [12].

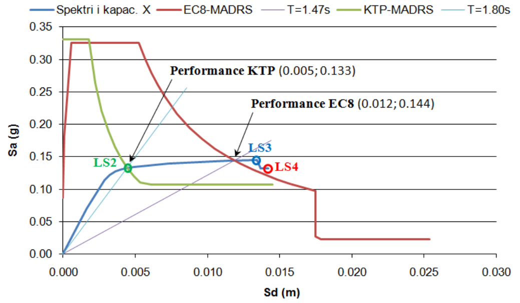

Figure 14 – Performance point for the X direction.

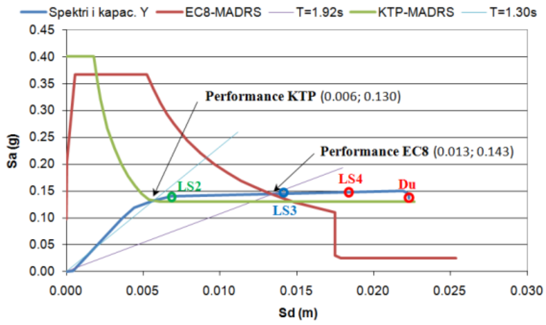

Figure 15 – Performance point for the X direction.

A.Comparison of the preformance according to KTP and EC8

The Eurocode 8 spectrum causes largest demages to the building, as seen from the graph. This means greater value of displacement, accelerations, period of vibrations and ductility. This is expected since graphically of the spectrum according to The Eurocode 8 has a higher value than that of KTP. This has to do with the risk calculation according to both methods. Eurocode is quite certainly more advanced than KTP in terms of parameters and analysis of the accelerations of the soil. The last ones are calculated taking into account previous earthquakes and the geology of the area. Yet in some cases certain buildings according to KTP results safe, again we should be performed the analysis according to The Eurocode 8, as the most advanced and most disadvantaged. However, it is important to show how the two calculations differ from each other. We are giving summarized in the following tables.

Table 2 - Performance point for the building without reinforcement according to KTP.

	Sd (cm)	Sa(m/s2)	Displacement (cm)	Shear force	Ductility µ	Periode T (sek)
X	0.5	1.30	0.7	1715	1.5	1.47
Y	0.6	1.28	0.8	1570	1.3	1.30

Table 3 - Performance point for the building without reinforcement according to EC8.

	Sd (cm)	Sa(m/s2)	Displacement (cm)	Shear force	Ductility µ	Periode T (sek)
X	1.2	1.41	1.6	1840	3.7	1.80
Y	1.3	1.40	1.8	1730	3.1	1.92

B.Comparison of the performance of the building with and without reinforcement

The nonlinear analysis according to FEMA 440 serves us to determine which will be the performance of the building given a seismic spectrum. In our case we found the performance of the building from the KTP and Eurocode 8 spectrum. The reinforcement that we modeled changes the performance of the building making it safer. It is necessary to assess the quantitative and qualitative role of the applied reinforcement

a) Displacements

The allowed displacements of the building to the point of destruction rose about 2 times in the X and Y direction. Also the deformed shapes improved a lot.

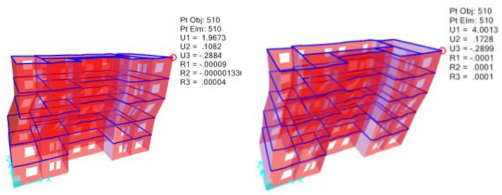

Figure 16 – Comparison of displacements fot the cases without reinforcement (left) and with carbon fiber reinforcement (right) for the X direction

Relative interstory drifts are significant because they are in proportion to the shear stresses in the masonry. Judging on them it is possible to justify the horizontal distribution of shear forces. Below (Tab.4) are presented the maximum relative interstory drifts for the pushover analysis in the X direction.We note that all floors except the third floor have extra displacements, which confirms the distribution of forces on all floors. The third floor has no changes because this floor still remains the weak one.

Table 4 - Change of the relative interstory drifts for the cases with and without GFRP reinforcement for the pushover analysis in the X direction.

Cases	Floor 1 (cm)	Floor 2 (cm)	Floor 3 (cm)	Floor 4 (cm)	Floor 5 (cm)
Without Reinforcement	0.12	0.18	1.52	0.00	0.07
With GFRP	1.09	0.71	1.51	0.51	0.13
Difference	+0.97	+0.53	-0.01	+0.51	+0.06

b) Carrying capacity

Capacity curve gives us a clear idea of the change that occurs in the reinforced building with glass fibers. In Figure 17 are given the capacity curves for the building without reinforcement and with glass fiber reinforcement GFRP. The increase of the carrying capacity is visible both in the shear forces values, as well as in the displacements values. While the model without reinforcement collapses on the third floor, the reinforced model resistes more by engaging other floors. For the X direction, the maximum shear force has increased from 1855 in 2335 kN or 26%.

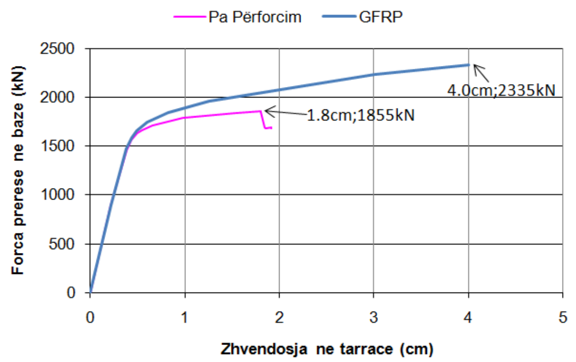

Figure 17 - The capacity curves for the X direction for the cases without reinforcement and with glass fiber reinforcement GFRP

c) Seismic performance

Calculation of the building's seismic performance with GFRP by FEMA440

The seismic performance is the most important aspect of the nonlinear analysis. Through graphical and analytical procedures,is intended to predict the effect of a seismic spectrum on a specific building. In the case of the building reinforced with glass fiber we notice an improvement of the seismic performance. This means less demages and greater stiffness for the building. Below is a comparison in tabelar form of the seismic performance by the KTP and EC-8 spectrum in the cases without reinforcement and with glass fiber reinforcement GFRP (Table 5 and 6).

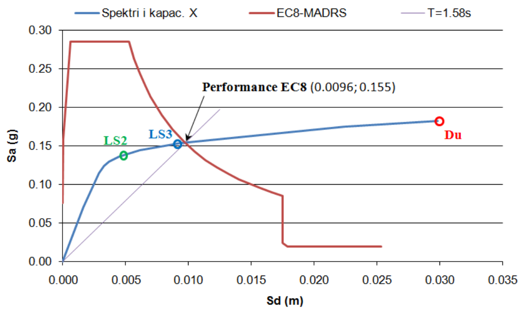

Figure 18 - Performance point for the X direction.

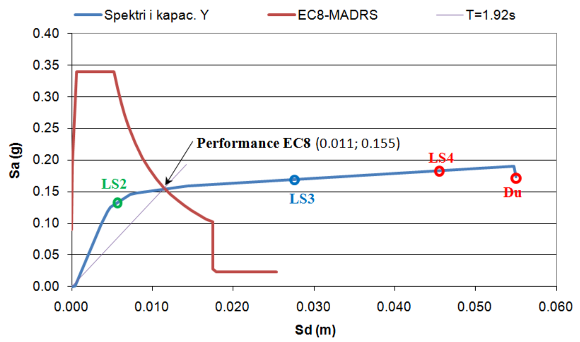

Figure 19 - Performance point for the Y direction.

Although the building has greater stiffness, again there are non uniform drifts especially to the fourth floor.

Table 5 - Performance point for the situation without reinforcement and with GFRP reinforcement by KTP in the X direction.

	Sd (cm)	Sa(m/s2)	Displacement (cm)	Shear force (kN)	Ductility µ	T (sek)	Service condition
KTP – without reinforcment	1.2	1.41	1.6	1840	3.7	1.80	LS2
KTP-GFRP	0.96	1.52	1.4	1980	2.4	1.58	LS3

Table 6 - Performance point for the situation without reinforcement and with GFRP reinforcement by EC-8 in the X direction.

	Sd (cm)	Sa(m/s2)	Displacement (cm)	Shear force (kN)	Ductility µ	T (sek)	Servic condition
EC8-X without reinforcment	1.3	1.40	1.8	1730	3.1	1.92	LS2
EC8-X - GFRP	1.1	1.52	1.5	1840	2.1	1.92	LS2

The EC-8 specter causes considerable damage to the building without reinforcement. The ductility reaches 3.7 with the KTP spectrum in the X direction and 3.1 with the EC -8 spectrum. These values are significantly reduced when the building is reinforced with glass fiber from outside. After the reinforcmet with GFRP the ductilicty recpectivlly reaches the values 2.4 and 2.1 for the KTP and EC-8 specturm. So generally the building undergoes less plastic deformations in both directions and is more secure. In terms of carrying shear force we note an increase of 6-7%. This increase is attributed to the increased effectiveness of the masonry due to the redistribution of stresses in all reinforced floors. Although displacements are smaller by about 10-20%, spectral acceleration is higher by about 7% in both cases. This is because the shape of the spectrum of EC8, which provides greater accelerations for the buildings, increases the stiffnes and reduces the oscillation period.

VII. CONCLUSIONS

Reinforced building resulted in a better performance than that without reinforcement. Improvement was observed in two main directions, the carrying capacity and displacements form the seismic spectrum. The increase the carrying capacity is explained by the redistribution of forces in masonry clad with reinforced layer by activating better the floor, avoiding local colapses. Besides carrying capacity in shear, we should also justify the deformed shape and the achieved ductility. The ductility in itself does not explain enough the plastic deformation state of the building. This is because it is taken into account only by comparing the displacement of the roof with the elastic displacement of the capacity curve. This does not take into account the relative displacement of the other floors. The building has a more ductile behavior if all floors have relative displacements comparable to each other. Conversely, the ductility is smaller if we have a soft floor wich is deformed more. We have soft floors in cases without reinforcement at the third and fourth floor.

Finally it is recommended that for strengthening the masonry building with specifications as 74/4 studied above, to use GFRP glass fiber reinforcement. This, if applied to the bottom three floors of five-storey load bearing masonry buildings, provide a better behavior of the structure under seismic action. On the reinforced floors they increase the rigidity and carrying capacity. Also avoid the phenomenon of soft floor on the third floor where the wall thickness changes from 38cm to 25cm. The distribution of shear stresses throughout the building is significantly improved. This means better utilization of the masonry material and a more effective suppression of seismic energy.

BIBLIOGRAPHY

[1] Aliaj, Sh. Sulstarova, E. Peci, V. Muco, B. (2004). "Probabilistic Seismic Hazard Maps for Albania".
[2] Tomazevic, M. (2007). "Damage as a Measure for Earthquake Resistant Design of Masonry Structures: Slovenian Experience", J. Civ. Eng. (Canada), 122, 1040-1047.
[3] National Buildings Archive of Albania. (1974).
[4] Tomazevic, M. (1999). "Earthqauke-resitant design of masonry buildings, Series on Innovation in Structures and Construction", Vol. 1, Imperial College Press, London.
[5] Calvi, G. M. (1999). "A displacement-based approach for vulnerability evaluation of classes of buildings". J Earthquake Eng 3(3):411–438.
[6] Kodet Shqiptare te Projektimit (1952, 1963, 1978, 1989) dhe versionet e permirësuara
[7] Bilgin, H. and Korini, O.: Seismic capacity evaluation of unreinforced masonry residential buildings in Albania, Nat. Hazards Earth Syst. Sci., 12, 3753-3764, doi:10.5194/nhess-12-3753-2012, 2012.
[8] EN 1996-1. European masonry design code. (2005). "Design of masonry structures. Part 1: General rules for reinforced and unreinforced masonry structures".
[9] EN 1998-1. European seismic design code. (2004). "Design of structures for earthquake resistance. Part 1: General rules, seismic actions and rules for buildings".
[10] Lorenzis,L.(2008)" Strengthening of masonry structures with fibre-reinforced polymer (FRP) composites" www.woodheadpublishing.com
[11] Valluzzi, M. (2012). "Round Robin test for composite to brick shear bond characterization", RILEM, Materials and Structures, DOI 10.1617/s11527-012-9883-5.
[12] ACI 440M 2004: ACI 440M Guide Draft-1, "Guide for the Design and Construction of Externally Bonded FRP System for Strengthening Unreinforced Masonry Structures", ACI Committee 440, USA, 2004, ACI, 80 pp.