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Seismic Performance Assessment of Masonry Buildings

Blerim Nika, Nikolla Vesho, Aguljeln Marku, Merita Guri
POLIS University Tirana, Albania

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ABSTRACT

In Albania, as a country with high seismic risk, seismic design and assessment of structures is very important. In most cases, the seismic design situation is crucial in structural solution and in the dimensions of elements. In addition to designing new structures seismic assessment of existing structures is an ever-increasing need due to the existence of old structures built with design codes that reflect knowledge and accumulated experiences up to the time of their design and construction. It can be said that in compared to 30 - 40 years ago, the changes in design codes are significant. The building that will be used as a case study for analyses purpose is named as type 77/11 according to the previous institute of construction. This can serve as a reference for the social masonry buildings built in the communism era before the year 1980. Before this year, the design code used was the KTP-63. The analysis will be performed with ETABS software, and for the seismic performance will be used the KTP-89 spectre and EC-8 spectre (since Albania is trying to implement this code as a national standard). For strengthening technique will be used TRM.

Keywords: Performance point, strengthening, TRM, capacity curve

I. INTRODUCTION

Like many other countries that have gone through or are still in the process, Albania is facing development challenges and the constant transformation of the economic, social, cultural and environmental context. The rapid pace of this transformation, confusion, lack of planning but spontaneous development as well as the difficulties encountered during a chaotic transition process, have created a development model oriented to the present and which does not guarantee in most cases, meeting the needs of future generations. It began to build massively in Albania long after the establishment of democracy. This came from the huge shortages that were in the housing field, but unfortunately the technology left much to be desired. Lack of legislation as well as in most cases there were constructions without the opinion of specialists and this was especially in the suburbs of urban and rural areas. The structural stability of the building stock, 70-80s, may be at risk, taking in consideration structure degradation over time; as the 26th November earthquake showed. In Albania, until the end of the communist era in 1990, the masonry was used for residential and public buildings because it was low cost for that time. Nowadays, these types of buildings are still in use.

Structures designed with previous codes have suffered severe damage due to insufficient capacity to cope with seismic load and limited ductility. Concerns about the suitability of old codes with new ones can be answered more accurately through input of new methods of analysis [1]. In addition to designing new structures seismic assessment of existing structures is an ever-increasing need due to the existence of old structures built with design codes that reflect knowledge and accumulated experiences up to the time of their design and construction. It can be said that in compared to 30 - 40 years ago, the changes in design codes are significant.

II. ALBANIAN CONTEXT

2.1. Seismic design of masonry buildings

In Albania over half of the residential building stock was built before the 1980s and hence does not comply with modern energy efficiency and seismic safety requirements. Low to mid-rise brick masonry buildings constitute the typical traditional buildings widespread Albanian, with very limited construction of RC structures before 1985. Most mid-rise and high-rise buildings, in turn, were constructed after the 1980s using reinforced concrete. Design standards have not been the same since the first day they were introduced, but instead they have evolved during the years along with the evolution of the science and the accumulated experience.

In Albania, the first national-wide code was implemented in 1963, which was afterwards further enriched in 1979 and 1989. However, since the majority of the buildings is older than most of the contemporary regulations, it is clear that their seismic resistance is also much less compared to newer structures. This is due to the fact that the seismic actions considered in old buildings are less than 50% compared to the ones accounted for nowadays. The obvious consequence of the above is the under-designing of the bearing elements of a building. For these reasons these structures do not meet with requirements of new construction codes. In Europe, most countries have introduced the "Structural Eurocodes", which reflect a high level of knowledge in field of Structural Engineering. The last update of KTPs (Albanian construction code), was made in 1989 with approval of KTP-N.2-89 [2]. On the other hand, many existing buildings were realized before this year, designed in accordance with even older design codes. Especially the buildings treated in this study have been designed with codes in force at the time of construction starting in 1963 (KTP-63, KTP-78). The figure 1, shows the schematic design of the buildings regarding the changes of KTP after major earthquakes event.

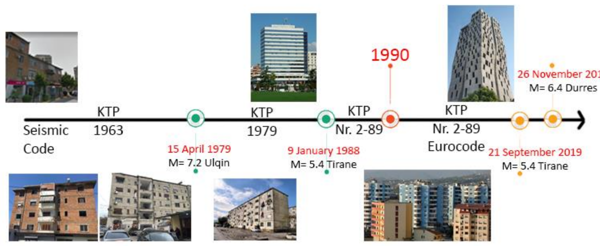


Figure 1 Schematic time period of buildings typologies according to National Codes and seismic events

Social buildings are spread throughout the Republic of Albania and are of different types from a structural point of view. However, there are common elements between them and this fact can serve to carry out a structural assessment of buildings based on a study performed on a limited number of them. In this context, it can be mentioned the fact that most of the buildings in Albania were built using brick masonry as a building material.

The purpose of this study is to evaluate and improve seismic performance of typical projects of residential masonry buildings stock, selected in Albania which are designed in accordance with the codes (KTP-63, 1963; KTP-78, 1978). This assessment will be performed according to the instructions of EC8, ATC40 and FEMA 440, given the nonlinear behavior of masonry. Among the typical residential building projects, one has been selected as representative coded 77/11. Performance seismic assessment will be performed in accordance with FEMA 440 guidelines [3]. Examination of capacity curves will identify structural deficiencies.

2.2. Seismic risk assessment

The first seismic map of Albania dates back to 1952 as a product of the work done by experts of the Institute of Sciences and the Ministry of Construction of that time. Since then, studies on assessing the seismic risk in our country has continued with numerous publications to the present day. The map of seismic zoning that is still in force dates back to 1979 (figure 2, middle). In figure 2 (left) is the map of 1963 [4].

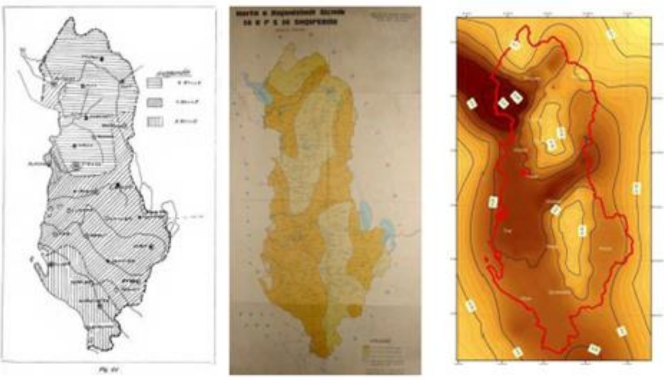


Figure 2 The proposed map of seismic zoning 1963(left) an actual map of seismic zoning 1979 (middle) Source: Baballëku, 2014) Seismic risk assessments in Albanai (right) (Source: Aliaj.Sh; Koçiu.S; Muço.B; Sulstarova.E, 2010

Thus, since 1952, seismic risk has been estimated always increasing. A good part of the buildings, object of this study, were built before the year 1979, which means that not only the technical conditions were old, but also the zoning map seismic has had low values of the seismic intensities of the expected earthquakes. Among the recent works [5] we can single out the map shown in figure 2 (right), in which it is noticed that seismic risk assessments in Albania tend towards an increase in values compared to earlier editions.

Although the use of "design spectra" for seismic analysis has been present in design codes in our country, the values have been much lower in compared to today. In figure 3, the spectra of KTP-N.2-78 are presented in the same coordinate system and Eurocode 8, EN 1998-1 (according to the seismic map of ‘04, not in force). The increase in the values of spectral accelerations between periods is clear. If we compare today's demand (for an area with ag = 0.25g) with that of 1978 for an area of intensity VIII point, an increase of about 5 times the spectral acceleration is observed [6] for the low-rise buildings.

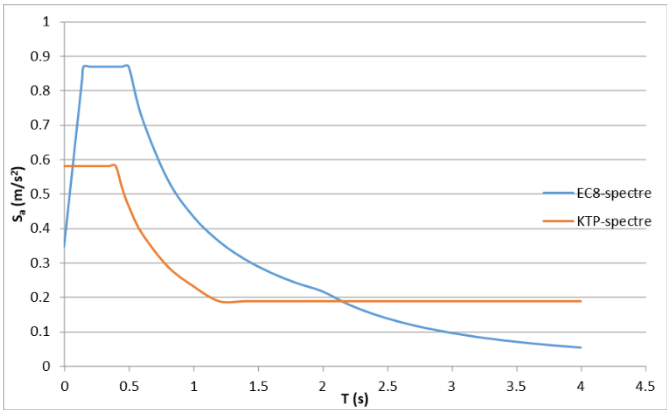


Figure 1 Comparison of spectral accelerations between KTP and EC-8 for Tirana (soil category II by KTP, and B be EC-8)

III. METHODOLOGY FOR SEISMIC ANALYSIS

Masonry is a material consisting of masonry units (such as bricks, blocks, stones) and mortar. Mortar, being the bonding material represents an essential role in the behavior of masonry as a whole. For Computer analysis, we can distinguish two major of masonry modeling techniques, respectively micro and macro modeling [7].

Through the procedure known as "homogenization" it becomes possible that the three components (masonry unit, mortar and their interface) to be merged into a single unit for calculation purposes. The two-dimensional element obtained (detached representative unit in figure 4), can be conceived in different sizes. The representative unit can be considered as orthotropic, with different properties according to three directions. The orthotropic behavior of the masonry corresponds to reality. It arises as a consequence of the technique of its construction, given here the different geometry of the bricks according to the different directions and their connection with the mortar. Representing these properties by numerical values requires that the values of stresses, deformations, and other characteristics to be determined according to the three main directions. Also, the relationship between these characteristics needs to be defined.

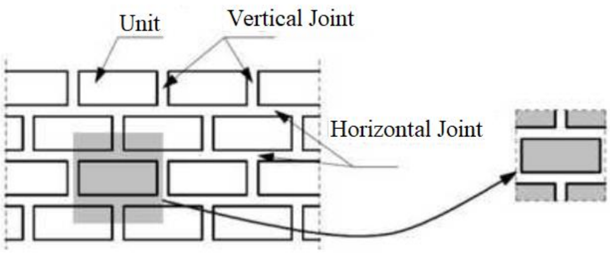


Figure 4 Homogenization procedure (Source: Lourenco, 1996)

3.1. Calculation of the capacity curve

For the calculation of the masonry building in the program ETBAS will be used nonlinear pushover analysis. In this type of analysis, the building is subjected to a horizontal loading until destruction. The pattern of destruction is determined by the purpose of analysis. This model may consist of one or several horizontal force or by displacement of modal shapes. Low-rise buildings, such as those with masonry in Albania, during the action of seismic forces vibrate mainly according to the first modal forms [8]. Given that with pushover analysis we will evaluate their seismic performance, it is reasonable to use the first modal forms as loading models (figure 5).

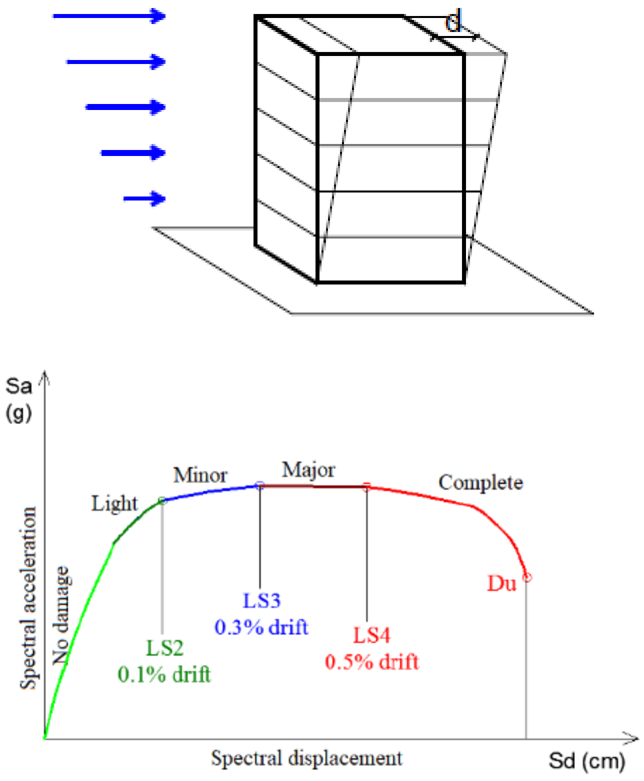


Figure 5 Pushover analysis and Building capacity curve with service states

3.2. Determining service states

By service states we mean a limit point on the capacity curve which is used to classify building damage. These limit states are a function of the type of construction and the material with which they are built. Researcher Calvi [9] has proposed assigning service states in function of relative displacement between floors (interstorey drifts). Relative displacement is directly related to shear stress from the seismic force absorbed by each floor. Given that the destruction of the building comes from this shear force, it is convenient to use these limit service states for all cases of masonry buildings. So, in the following calculations will use these limits. Calvi [9] set three limits for service states (figure 5, right) as follows:

LS2 - Minor structural damage and / or moderate non-structural damage; building can be used after the earthquake, without the need for significant reinforcement or repair of structural elements. The suggested limit of relative displacement is 0.1%.

LS3 - Major structural damage and major non-structural damage. The building cannot be used after earthquake without significant repair. However, repair and reinforcement are feasible. The suggested limit of relative displacement is 0.3%.

LS4 - Complete collapse; repair of the building is neither possible nor economically reasonable. The structure will have to collapse after the earthquake. Beyond this limit state is expected complete collapse with risk to human. The suggested limit of relative displacement is 0.5%.

IV. SEISMIC RETROFIT

As has been learned from all recent earthquakes, modern (including masonry buildings) designed according to requirements of state-of-the-art earthquake engineering, successfully resist strong ground motion, whereas many existing, non-engineered as well as engineered buildings, "old" by the standards of earthquake-resistant design, and not merely by the age of construction, collapse or suffer severe damage. Since old masonry buildings are typical representatives of traditional non-engineered construction, their seismic vulnerability is, in general, relatively high. Indeed, most earthquake damage and loss of life in these regions is caused by inadequate seismic behavior of existing masonry buildings, in most cases residential houses in urban and rural areas which are of traditional type of construction.

Seismic retrofitting requires considering some peculiarities in contrast to the usual procedure followed in strengthening for static loads or energy efficiency upgrade. Three distinctive features of a structure should be considered and well-coordinated for a successful seismic upgrade: stiffness, ultimate resistance and deformation capacity [10].

There is a wide variety of intervention techniques for strengthening and repair of masonry structures, which have suffered damage due to degradation, overload, basement sinking, change of temperatures, natural disasters such as wind, earthquake, etc. These techniques are divided into "traditional" and "modern". "Traditional" techniques use materials and construction processes that are originally used for structure construction, while modern techniques aim more efficient solutions using new materials and technologies. Some of the most commonly used reinforcement techniques are [11]:

Traditional - Demolish-rebuild intervention, Placement of concrete belts, Installation of steel tie rods, Reinforced injections, Reinforcement of openings with metal profiles in the form of boxes.

Modern - Reinforcement with composite materials FRP (Fiber reinforced polymer), Reinforcement with composite materials TRM (Textile-Reinforced-Mortar)

V. SEISMIC ANALYSIS OF THE STRUCTURE

The building that will be used as a case study for analyses purpose is named as type 77/11 according to the previous institute of construction. This can serve as a reference for the social masonry buildings built in the communism era before the year 1980. Before this year, the design code used was the KTP-63. It had little knowledge for the seismic risk and design. Therefore, the 77/11 building, designed in 1977, has no seismic concrete belts. The thickness of the bearing walls is 38 cm for the first three storeys, and 25 cm for the upper two. The slab are ribbed slabs, with thickens 15 cm, and ribs every 20 cm. The analysis will be performed with ETABS software, and for the seismic performance will be used the KTP-89 spectre and EC-8 spectre (since Albania is trying to implement this code as a national standard).

In the figure below is shown the plan and elevation view of the reference building 77/11.

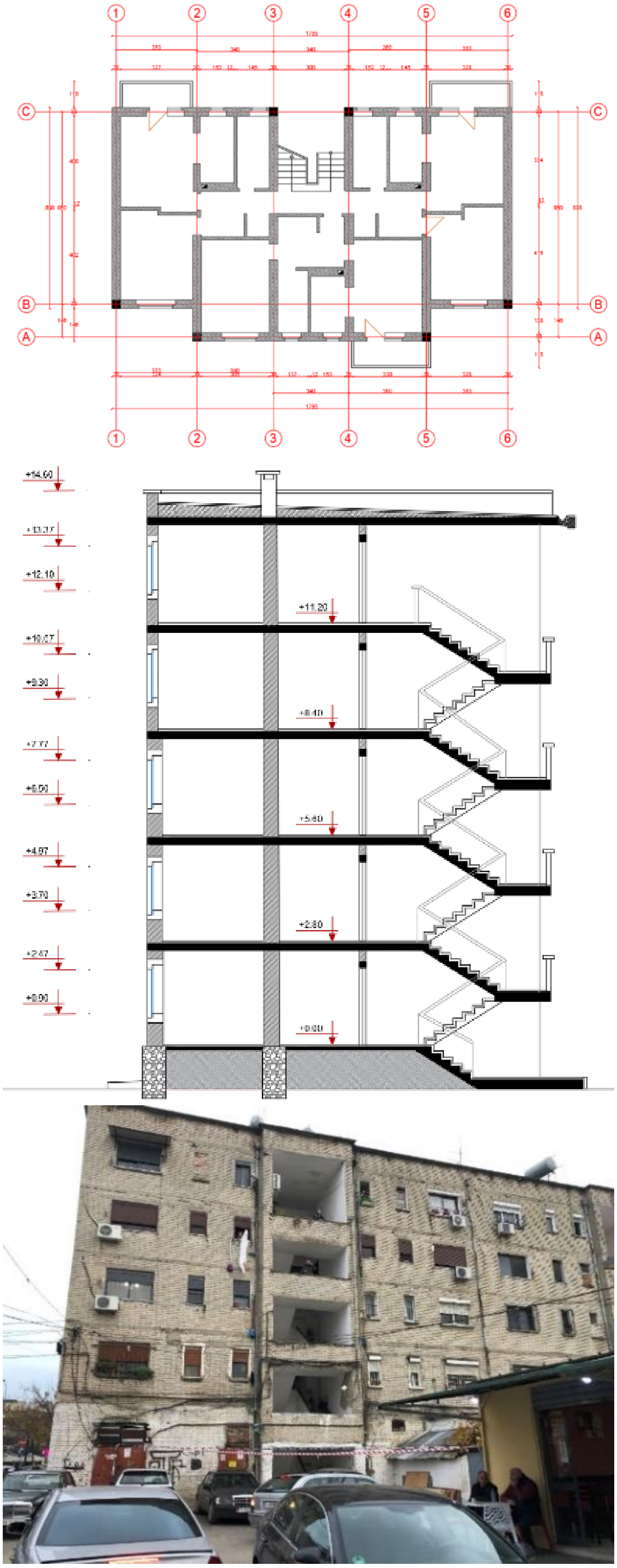


Figure 6 Plan and elevation view of the buidling type 77/11 and Photo of the building

5.1 Static loads and seismicity

Will be taken in consideration the dead and live loads. The dead loads include the self-weight of the building (slabs and internal non-bearing walls with their respective layers). The dead load calculated is g = 6 kN/m². The live load is according to EC-1, and will be taken q = 1.5 kN/m² for the first four storeys, and q = 2 kN/m² for the roof.

Seismic spectre

Will be considered the analysis with two different spectres, KTP-89 and EC-8. (figure 3)

For the seismic spectre it has been chosen the city of Tirana.

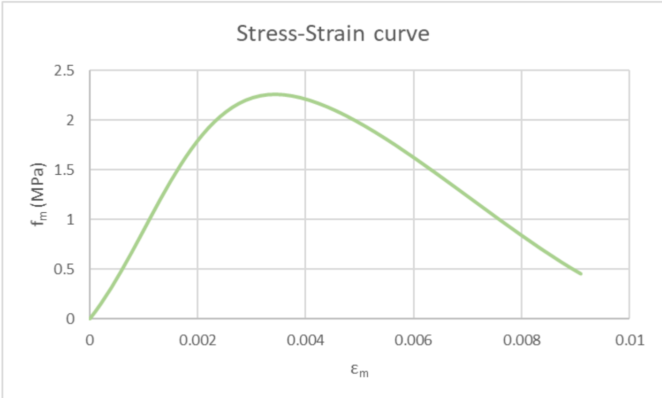


Figure 7 Stress-Strain curve for compression S11 and S12

5.2 Masonry properties

To properly analyze the structure, it is needed to determine the mechanical properties of masonry. The compressive strength of brick is f_b = 7.5 MPa and of mortar is f_j = 2.5 MPa (according to the original design specifications). We can determine the compressive strength of the masonry wall with the equation below:

$f'_m = 0.63 * f_b^{0.49} * f_j^{0.32} = 0.63 * 7.5^{0.49} * 2.5^{0.32} = 2.26$ MPa

and the elastic module $E = 550 * f'_m = 550 * 2.26 = 1240$ MPa

The respective strain for the compressive strength is:

$$\varepsilon'_m = C'_j \frac{f'_m}{E_m^{0.7}} = \frac{0.27}{f_j^{0.25}} * \frac{f'_m}{E_m^{0.7}} = \frac{0.27}{2.5^{0.25}} * \frac{2.5}{1240^{0.7}} = 0.0033$$

The ultimate stress: $f_{mu} = 0.2 * f'_m = 0.2 * 2.26 = 0.453$ MPa

The ultimate strain: $\varepsilon_{mu} = 2.75 * \varepsilon'_m = 2.75 * 0.0033 = 0.0091$

With these values, we can generate the stress-strain graph as in figure 7:
For the shear strength (S12 stresses), KTP reccomends the for mortar with 2.5 MPa comprensive strength, the shear strenght is 0.11 MPa.

With these values, we can generate the stress-strain graph for the shear stresses as in figure 8. The curve is considered ideal bilinear with maximum shear strength as the cohesion between mortar and brick. This assumption is made because of true behavior is very close to the bilinear shape according to experimental tests. Elastic maximum deformation is obtained from the first part of the almost linear curve of experiments on masonry. In order for the program to calculate the maximum displacement of the building, the last part of the graph is added which reduces the shear resistance to zero. In this way the calculation stops when the bearing masonry is destroyed to the extent that overall stability is compromised.

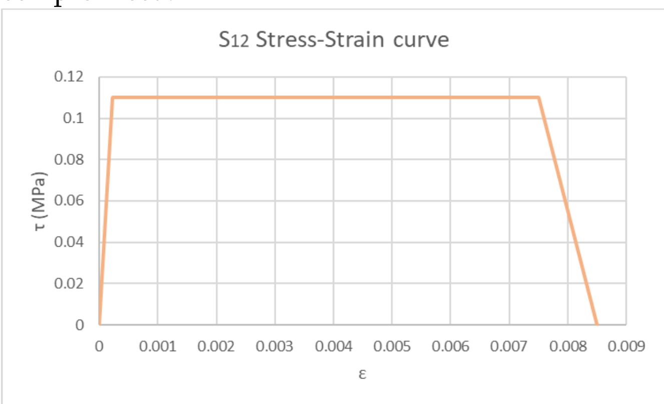


Figure 8 Stress-Strain curve for shear S12

5.3 Nonlinear analysis

For the nonlinear analyses is considered the first modal vibration of the structure. The first mode is in X-direction. The period of first mode is T_1 = 0.69s, and for the second mode is T_2 = 0.5s. As for controlled displacement is considered a joint in roof. The first analysis is done in the X direction:

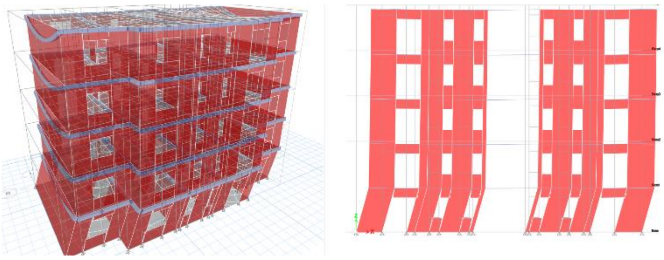


Figure 9 Maximum nonlinear displacements direction

To analyse the structure according to the Calvi service limit states, it is necessary to calculate the interstorey drifts and compare them to limit drift as we discussed in the previous paragraphs. The drifts for the service limit states are as below (story height is 2800 mm): LS2 =0.1% * 2800 = 2.8 mm; LS3 =0.3% * 2800 = 8.4 mm; LS4 =0.5% * 2800 = 14 mm
Below are presented the interstory drift for each story corresponding to the steps of the pushover analysis.

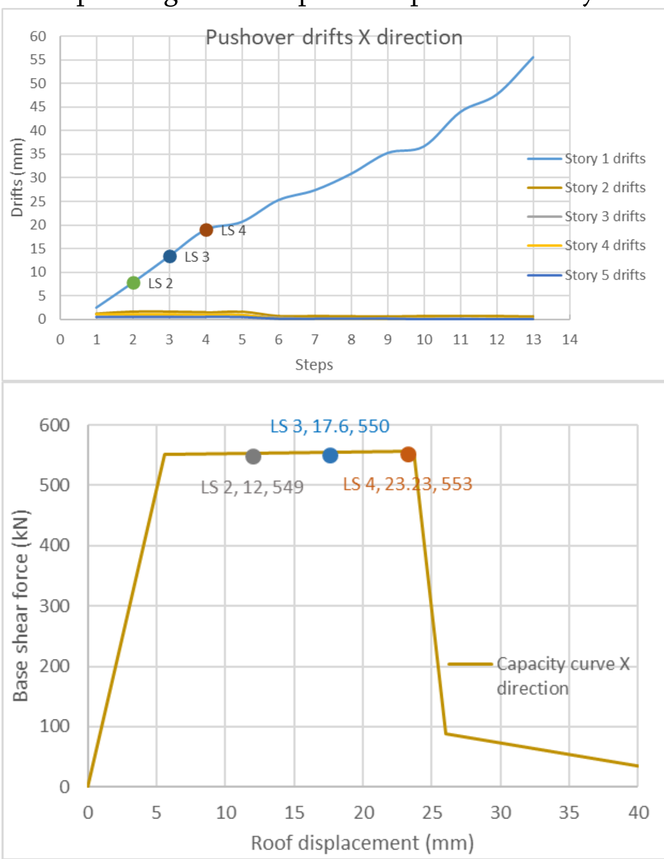


Figure 10 Pushover drifts and Limit states in the capacity curve X direction

As we can see from the above graph, the maximum drifts are in the first storey. This is attributed to the shear force which is greater in the first floor. Therefore, the first story acts as a soft storey, and will be needed to strengthen. All other storeys do not suffer plastic deformation since relative displacements are less than 0.1%. To understand in which direction the structure is weaker, the same analysis was done also according to the Y direction.

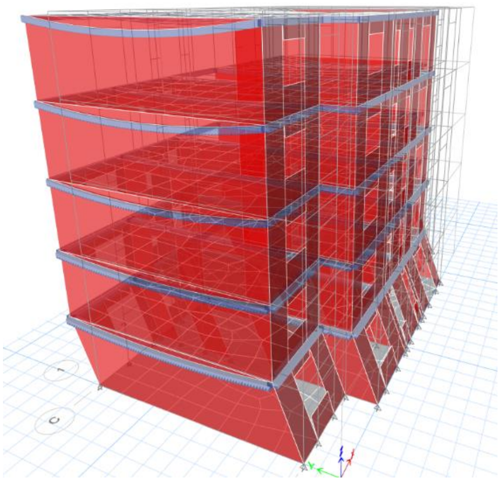


Figure 11 Maximum nonlinear displacements Y direction

The interstory drift for Y direction for each story corresponding to the steps of the pushover analysis are presented below:

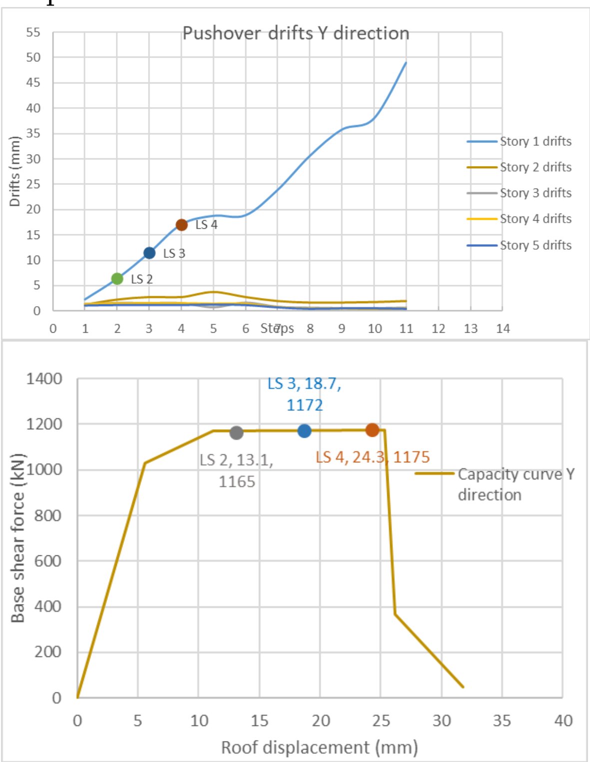


Figure 12 Pushover drifts and Limit states in the capacity curve Y direction

Also,in Y direction, the maximum drifts are in the first storey. This is attributed to the shear force which is greater in the first floor. Therefore, the first story acts as a soft storey in both directions. The seismic performance of the 74/11 type building will be calculated below according to the improved "Equivalent Linearization" procedure found in the document FEMA 440. Its steps were explained in the previous chapters. The spectres used, are according to KTP-89 and EC-8. Below we present the building performance degree for both X directions and Y. The same graph shows the performances of both KTP and Eurocode spectra.

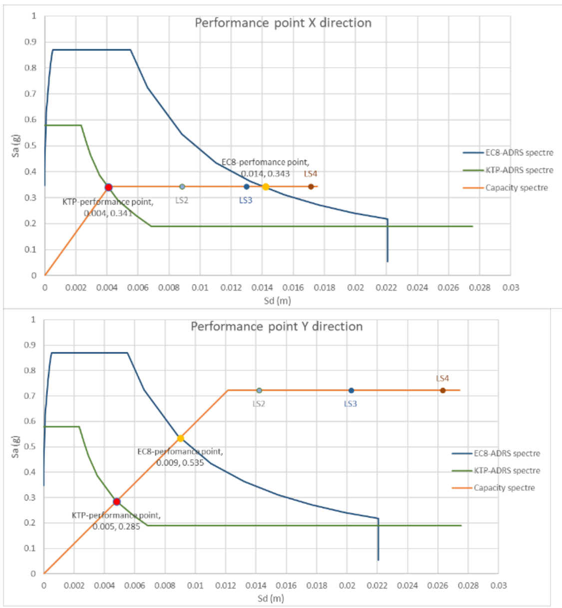


Figure 13 Performance point X direction (left) Performance point Y direction (right)

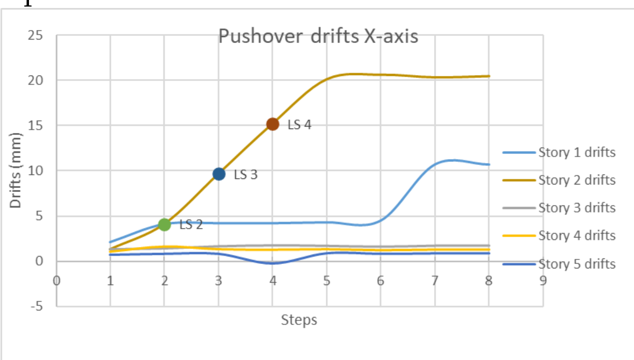
Eurocode 8 spectrum causes greater damage to the building, as seen from the graph. This means greater values of displacements, accelerations, vibration periods and ductility. This thing is to be expected given that graphically the spectrum according to Eurocode 8 has higher values than that of KTP. About calculating risk according to both methods, Eurocode is much more advanced than KTP in terms of parameters and analysis of ground accelerations. The latter are calculated by taking into account the old earthquakes, geology and geotectonic of the area. Although, in X direction, according to KTP, the building is safe, it still needs to be also performed the analysis according to Eurocode 8, as the most advanced and the most unfavorable.

However, it is important to show how much the two calculations differ from each other.

In the X direction, according to EC-8, can withstand the earthquake, but it has a limit state of LS3. This mean that after the earthquake, the building must have deep structural repairs which are going to be costly. The building type 74/11 is destroyed by seismic shear force on the ground. The reason is the greater shear force in the first floor. As noted by the interstoreys drifts, the first floor suffers major damage from horizontal forces, while the other floors are hardly damaged at all. This phenomenon is same for both X and Y directions. According to KTP, the performance point is achieved without any minor damage. In Y direction, since building does not have any damages, as KTP and EC, when withstanding the design earthquake, it does not need strengthening in that direction. Therefore, the further analysis and strengthening will be done only in X direction.

5.4. Strengthening of the building

For the nonlinear analyses is considered the first modal vibration of the structure. The fist mode is in X-direction. The period of first mode is T1 = 0.69s, and for the second mode is T2 = 0.49s. As for controlled displacement is considered a joint in roof. The analysis is done in the X direction. Below are given the results of the nonlinear analysis. The interstory drift for x direction for each story corresponding to the steps of the pushover analysis are presented below:



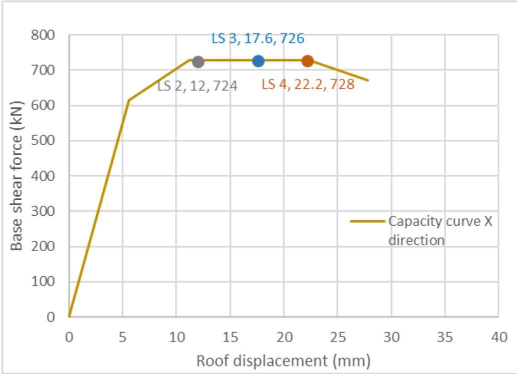


Figure 14 Pushover drifts and Limit states in the capacity curve- TRM

It is noted that interstitial displacements are more comparable to each other for the first two storey. This shows a better redistribution of shear stresses in masonry. Now the second storey floor is weaker and therefore acts as a soft storey. The service limit states are then presented in the capacity curve as above. The seismic performance of the reinforced building will be calculated the same as for the case without reinforcement. So, the nonlinear standard procedure will be applied FEMA 440. It will then be judged on improving the seismic capacity of the building type 74/11. Below is presented the building performance degree with TRM reinforcement. The same graph shows the performances of both KTP and Eurocode spectra.

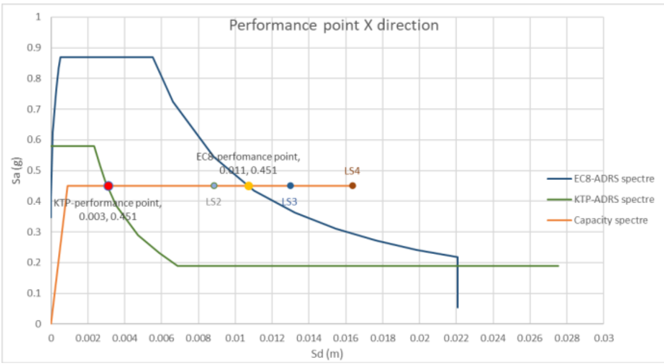


Figure 15 Performance point – TRM

The spectrum according to Eurocode 8 causes significant damage to the building without reinforcement. With the reinforcement the performance point is achieved for higher displacements and higher spectral accelerations. So, in general the building more ductile and is safer. In terms of shear force capacity, we notice an increase of it. This increase is attributed to the increase in the effectiveness of masonry as a result of redistribution of stresses on the two floors. Also, according to EC8, the building withstands without collapse the design earthquake. The service limit state is LS2 that mean it suffers minor damages, while without TRM it was the LS3 level. The creation of the soft story in the second floor might be a problem, but since the structure is in LS2, it means that would not be major damages and therefore isn't a significant problem.

VI. CONCLUSION

The selected building is modeled on the original design variant but also by applying reinforcements to its perimeter. From the analysis of the building without reinforcement resulted the first floor as a soft storey.

For reinforcements were chosen polymer materials equipped with fibers with high resistance. It was concluded that reinforcement for this category the building should be located up to the first floor of the building, for him avoid the phenomenon of soft storey.

The reinforcement showed good results in upgrading the capacity of structure. The improvement was observed in two main directions, in bearing capacity and in shifts from the seismic spectrum. Bearing capacity in shear is increased. The increase in bearing capacity is explained by redistribution of forces in the masonry lined with reinforcing.

In addition to the bearing capacity in the shear, the shape of deformed ductility was achieved.

Ductility in itself does not explain it enough the state of plastic deformation of the building. This as it is considered only by compared the displacement of the roof point with the elastic displacement in the curve of capacity. The relative displacement of the other floors is not taken into account here. Building has more ductile behavior if all floors have relative displacements comparable to each other. Conversely the ductility is smaller if we have a soft storey that deforms further.

It was noted that current seismic code in Albania shows many deficiencies regarding the other codes, therefore it's an urgent need its upgrade. The structures built with this code do not perform accordingly with the present European codes.

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