



Seismic capacity evaluation of unreinforced masonry residential buildings in Albania

H. Bilgin and O. Korini

Department of Civil Engineering, Epoka University, Tirana, Albania

Correspondence to: H. Bilgin (hbilgin@epoka.edu.al)

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Abstract. This study evaluates seismic capacity of the unreinforced masonry buildings with the selected template designs constructed per pre-modern code in Albania considering nonlinear behaviour of masonry. Three residential buildings with template designs were selected to represent an important percentage of residential buildings in medium-size cities located in seismic regions of Albania. Selection of template designed buildings and material properties were based on archive and site survey in several cities of Albania. Capacity curves of investigated buildings were determined by pushover analyses conducted in two principal directions. The seismic performances of these buildings have been determined for various earthquake levels. Seismic capacity evaluation was carried out in accordance with FEMA (Federal Emergency Management Agency) 440 guidelines. Reasons for building damages in past earthquakes are examined using the results of capacity assessment of investigated buildings. It is concluded that of the residential buildings with the template design, with the exception of one, are far from satisfying required performance criteria. Furthermore, deficiencies and possible solutions to improve the capacity of investigated buildings are discussed.

1 Introduction

Masonry is the most important construction material in the history of humankind. This term is used in a variety of forms such as stone, clay brick, cellular concrete block and adobe for the construction of building structures. The combination of heavy weight and high stiffness along with the lower tensile strength makes masonry structures prone to earthquakes. Since many urban settlements are in located in moderate to severe seismic zones of the world, seismic vulnerability

assessment of masonry buildings requires special consideration. Even though a large percentage of loss of life during the past earthquakes have occurred due to the poor performance of masonry buildings, the efforts to measure and to increase their seismic performance are not adequate when compared with current advances in the area of reinforced concrete structures (Tomazevic, 1999; Erberik and Yakut, 2008)

Recent devastating earthquakes have emphasised the inadequate seismic performance of unreinforced masonry (URM) buildings to the worldwide community. In literature, several studies related to performance of URM buildings in past earthquakes are available (Calvi, 1999; Decanni et al., 2004; Jagadish et al., 2003; Kaplan et al., 2010; Klingner, 2006; Pasticier et al., 2008; Yilmaz et al., 2012; Yoshimura and Kuroki 2001). Many of URM buildings were affected by severe earthquakes due to poor quality of construction, poor workmanship, aging and the lack of maintenance.

Following observed damages in past earthquakes (i.e. 1999 Kocaeli and Duzce in Turkey, 2001 Gujarat, India; 2002 Molise and 2009 L'Aquila, Italy; 2010 Haiti and 2010 Chile), there have been significant efforts to mitigate the earthquake hazards on URM buildings in many countries (Decanni et al., 2004; Jagadish et al., 2003; Kaplan et al., 2010; Klingner, 2006; Lagomarsina and Penna 2003; Yoshimura and Kuroki 2001). Seismic safety of URM buildings has been questioned in the wake of L'Aquila, Italy (6 April 2009), Haiti (12 January 2010) and Chile (27 February 2010) earthquakes because there was a widespread conviction that these buildings experienced considerable damage compared to reinforced concrete buildings.

In Albania, template designs developed by the governmental authorities are used for many of the buildings intended for residential purposes as common practice to save on architectural fees and ensure quality control during the

communist period (Korini, 2012). The representative typology of the country corresponds to URM buildings. There are standard URM buildings all over the country for residential purposes, 5 stories with different plans. The majority of existing residential masonry buildings in Albania, like in many other European countries, has been designed considering earlier codes (KTP-9, 1978; KTP-N2, 1989) when seismic loads were not required or the design was to lower level of seismic loads of what is currently specified.

Masonry systems can be either engineered or non-engineered and be classified as unreinforced, confined and reinforced masonry. Each system has been built through different construction technologies and exhibit different earthquake responses; URM exhibits brittle failure whereas the other two have enhanced strength and ductility. This study aims to evaluate seismic capacity of the existing masonry residential buildings constructed per pre-modern seismic code (KTP-9, 1978) in Albania considering nonlinear behaviour of URM components. Three residential buildings with template designs constructed in accordance with Albanian Code (KTP-9, 1978) were selected to represent an important percentage of existing residential buildings in Albania of moderate seismicity. Selection of template designed buildings and material properties were based on field investigation, archive study and the blueprints of these structures. Mechanical characteristics for the case buildings were taken from their blueprints and adopted for the analysis. Capacity curves of investigated buildings were determined by pushover analyses conducted in two principal directions. Seismic capacity evaluation is carried out in accordance with FEMA (Federal Emergency Management Agency) 440 (FEMA 440, 2004). Reasons for building damages related to URM in past earthquakes are examined using the results of capacity assessment of investigated buildings. Deficiencies and possible solutions to improve the capacity of URM residential buildings are discussed.

2 Description of structures

Until the end of communist period in 1990s, masonry buildings continued to be built using template designs. Masonry was used for public and governmental buildings as a low cost construction method for that time. Today these buildings are still in use and the main functions are mostly for residential purposes. Hence, a considerable number of buildings have the same template designs in different parts of Albania (Korini, 2012).

A field and archive survey were carried out in Tirana city to select the most common template designs among residential buildings. Being the capital city of Albania, Tirana represents a medium-size city in a seismic part of Albania (Aliaj et al., 2010). According to survey results, there were about 30 types of residential buildings with template designs. It is observed that the most common templates are TD-83/3,

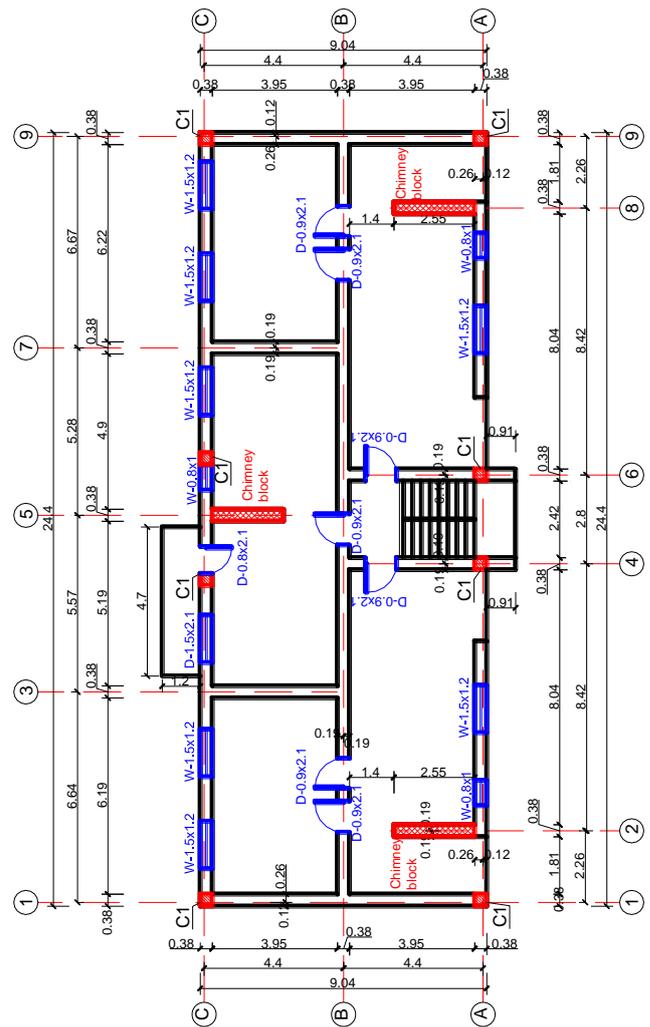


Fig. 1. Template design 83/3, (National Archive of Albania, 1983).

TD-72/3, and TD-72/1 which covers nearly 15 % of the total building stock (Korini, 2012). According to the blueprints of each template design, selected buildings are built with clay bricks of M75 with a resistance of 7.5 MPa and mortar of M25 with a resistance of 2.5 MPa. These mechanical properties taken from the blueprints of respective template designs are used and adopted for the analysis. Unlike many residential reinforced concrete buildings, URM buildings generally have a uniform distribution of mass and stiffness in horizontal and vertical plane because of similar architectural features due to similar purpose of use in all storeys. Therefore, they are less prone to structural irregularity effects such as, heavy overhangs, great eccentricities between mass and stiffness centres, etc. All of them have five floors. The load bearing wall thickness is 380 mm on first two storeys and 250 mm on the remaining three storeys. Representative plan views of the three buildings for the ground storey are shown in Figs. 1–3. All dimensions are in m.

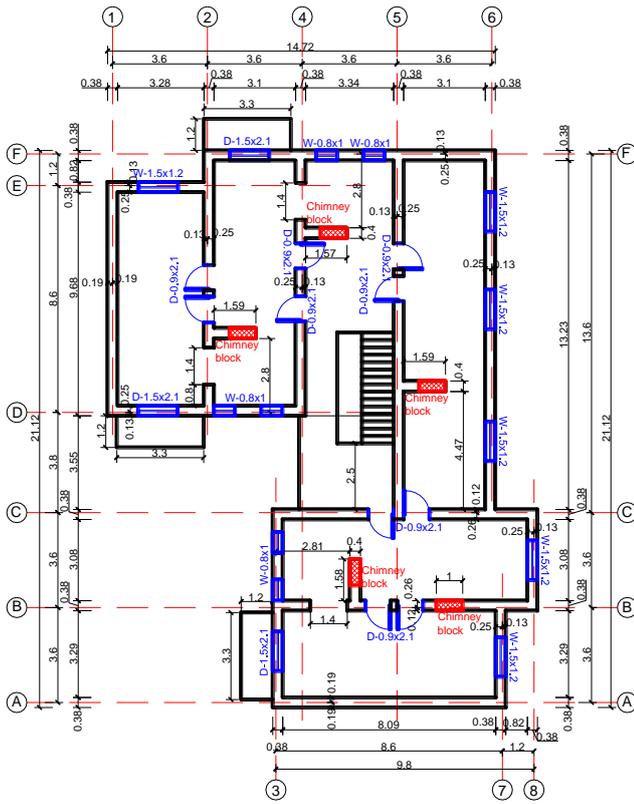


Fig. 2. Template design 72/3, (National Archive of Albania, 1983).

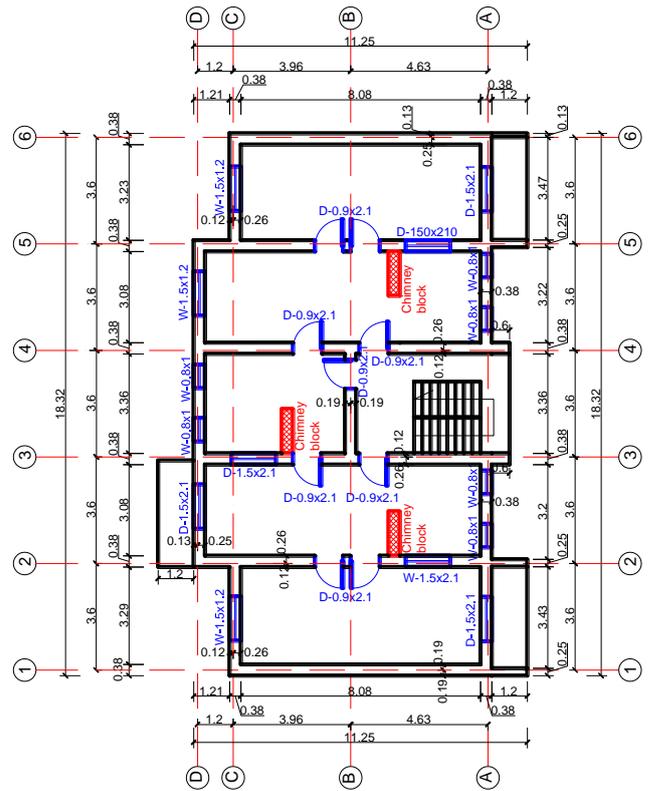


Fig. 3. Template design 72/1, (National Archive of Albania, 1983).

3 Mathematical modelling of representative buildings

SAP2000 (CSI, 2011) program has been used for modelling the considered building typologies. The 3-D modelling of URM buildings starts from the hypotheses on their structural and earthquake behaviour; the load bearing structure under horizontal and vertical loadings is defined, with walls and floors. The walls are the load bearing elements, while the floors are considered as planar stiffening elements (rigid diaphragm), on which the horizontal effects are distributed between the walls connected. Presence of ring beams in masonry prevents out-of-plane failure (Magenes, 2010) and provides the development of global structural behaviour governed by in-plane response of walls. This fact was also observed in previous shaking table tests (Benedetti et al., 1998; Mazzon et al., 2009). Experimental tests on masonry infilled concrete frames (Fardis, 1997) have revealed that severe acceleration levels are required to trigger an out-of-plane collapse due to increased natural period of vibration of the panel. In this study, the local flexural response of floors and out-of-plane behaviour of walls are not computed since they are considered as negligible with respect to the global building response dominated by their in-plane response.

For the modelling of the selected buildings two types of issues should be considered: correct representation of the mathematical model and inelastic characteristics of

materials. URM is a composite construction material which consists of masonry units and mortar. Brick and stone are the usual elements of masonry units. Mortar is used to make the connection between these units. Under vertical and horizontal loads, load-bearing of masonry considered as the assemblage of the masonry units and mortar is influenced by the compressive, shear and flexural strengths, durability, water absorption and thermal expansion.

To model this anisotropy, two different approaches have been offered in literature: “micro modelling” and “macro modelling”. Each modelling technique requires the adoption of different constitutive models. Modelling of masonry due to its anisotropic behaviour has been a very difficult task for several years. As a first approach, the finite elements methods can be used to model the masonry constitutive elements (mortar and unit elements). They are discretized into a certain number of finite elements then suitable constitutive nonlinear laws are adopted. A second approach is based upon the justification of “equivalent frames”. The structure is described by an assemblage of piers and spandrel elements. These elements are connected by rigid offsets and modelled by proper constitutive laws in order to take into account the mechanical nonlinearity (Dolce, 1989). Several studies have been done by different researchers; (Lagomarsino et al., 2006; Gamberotta and Lagomarsino, 1997; Penelis, 2006; Calderini and Lagomarsino, 2008; Belmouden, 2009). In these approaches,

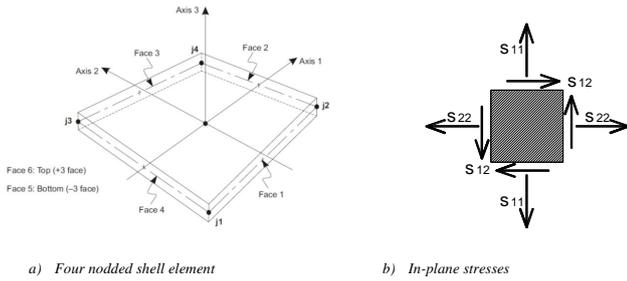


Fig. 4. A four noded shell element and in plane stresses. (SAP2000 reference manual).

a nonlinear macro-element model, able to reproduce earthquake damage to masonry structures and failure modes observed during experimental testing, has been implemented with similar approximations.

For nonlinear analysis of the selected URM residential buildings, material properties determined from the blueprints of the designs were taken into account. As aforementioned, many of the buildings intended for residential services have similar construction procedure supervised by governmental authorities. Material properties considered in this study were determined based on an archive study of 30 buildings.

4 Nonlinear material properties

Pushover analyses have been performed using SAP2000 Nonlinear Version 15 (CSI, 2011) that is a general-purpose structural analysis program. Member sizes in the template designs were used to model the selected buildings for nonlinear analysis. No simplifications are made for members; like rounding-off or grouping members with close properties. All structural elements are modelled as given in the template design. Three-dimensional model of each building is created in SAP2000 to perform pushover analysis. Walls are modelled as nonlinear layered shell elements. The anisotropy of masonry is modelled by two different stress-strain curves. Each of them represents respectively vertical stresses S11, horizontal stress S22 and shear stress S12 (Fig. 4). S11 and S22 stress-strain curves are determined using the relation provided by Kaushik et al. (2007) (Fig. 5). Parabolic part of the curve is predicted by Eq. (1) which is valid until stress drops to 0.9 f_m in the descending part.

$$\frac{f_m}{f'_m} = 2 \frac{\epsilon_m}{\epsilon'_m} - \left(\frac{\epsilon_m}{\epsilon'_m} \right)^2 \tag{1}$$

$$f'_m = 0.63 \cdot f_b^{0.49} f_j^{0.32} \tag{2}$$

where f_m is the mortar compressive strength, f'_m is the masonry compressive strength, ϵ_m is strain in masonry and ϵ'_m is the peak strain corresponding to f'_m . In Eq. (2), f_b and f_j ; brick compressive strength according to Eurocode 6 (EN

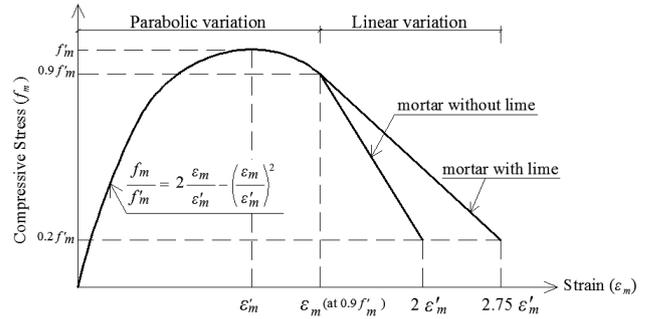


Fig. 5. Masonry idealised stress-strain curve for compression (Kaushik et al., 2007).

1996-1, 2005) and mortar compressive strength, respectively. Then a linear part is proposed for the curve. It is assumed that masonry may have a residual strength at 20 %, due to friction between detached parts. In this study, the Eq. (3) is used to estimate ϵ_m .

$$\epsilon'_m = C'_j \times \frac{f'_m}{E_m^{0.7}} \tag{3}$$

$$C'_j = \frac{0.27}{f_j^{0.25}} \tag{4}$$

On the other hand, to take into account the shear resistance, shear stress-strain curve should be defined. This curve needs to represent the horizontal failure. In reality, when a masonry member is subjected to lateral ground motion the horizontal resisting strength is represented by the cohesion and friction between brick and mortar which can be expressed with Coloumb friction (Eq. 5):

$$\tau = c + \sigma \times tg\varphi \tag{5}$$

where σ is the vertical stress and $tg\varphi$ stands for friction between elements. In this study, shear resistance is represented by a material nonlinear curve (cohesion) and the friction is neglected. Annex C of EN 1998-3 (EC-8) provides drift limits for in-plane response of existing URM buildings. For the S12 stress-strain material curve, an approach which was used by researchers (Lagomarsino et al., 2006, 2007; Korini and Bilgin, 2012) have been taken into consideration and adopted for the analysis.

5 Seismic demand

Seismic loads are commonly represented by response spectrum functions which are derived from time history records of earthquakes in specific areas. Albania, located in the Balkan Peninsula, has a moderate seismic hazard and tectonic activity (Aliaj et al., 2010). Microzonation of the country allowed classifying the soil of the country in three types.

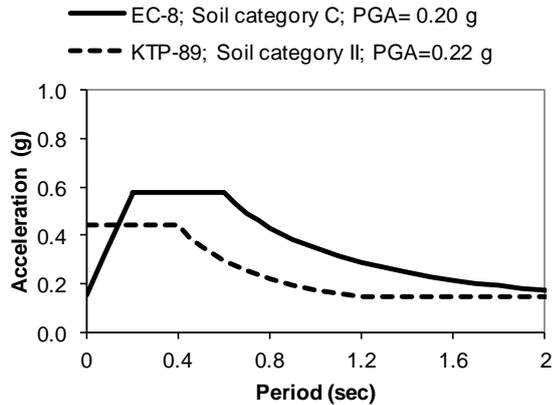


Fig. 6. Comparison of elastic response spectrums.

KTP-N2-89 normative design response spectrums are still used, since Eurocode 8 is not yet legally approved. In this study both Eurocode 8 and KTP-N2-89 spectrums are used in order to make a comparison and question the adequacy of current design spectrum. It is obvious that the Eurocode 8 spectrum has higher demand than the other (Fig. 6). Since the following existing structures have been constructed in different parts of the country, both ground conditions and seismicity is variable. In this study, the demand calculations for the seismic assessment of the considered buildings are performed considering the soil Type C with a moderate seismicity (0.2 g) according to Eurocode 8 (2004) and its corresponding spectra considering soil category II and medium seismicity (0.22 g) in KTP-N2-89 (1989).

6 Identification of damage limit states

A performance level is a limit state on the pushover curve that is used to classify the damage. As recommended by several researchers (e.g. Priestley, 1997), deformation thresholds may be the best indicators of identifying the limit states corresponding to structural and non-structural performance damage levels. In order to define these damage limits or performance levels of the URM template designs, there are neither experimental results based on laboratory tests nor available values calibrated from observed damages during the earthquakes. On the other hand, values of the mechanical properties of the materials used in these template designs have been taken from the blueprints of these projects and the actual values are not completely known. Considering all these aspects, there are different approaches to damage limit states classification for URM. Calvi (1999) and Lagomarsino and Penna (2003) have introduced different thresholds of the spectral displacement for discrete damage states based on the bilinear representation of the capacity spectrum. In this study for the performance assessment of the considered template designs, both thresholds have been employed.

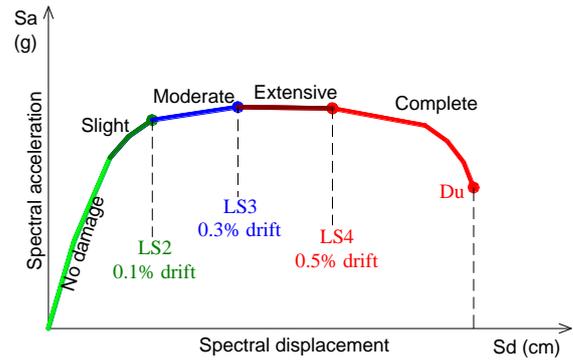


Fig. 7. Performance levels on pushover curve.

Calvi (1999) proposed four damage limit states for masonry structures (Fig. 7). Lagomarsino and Penna (2003) identified yield point and ultimate displacement for a structure and then split the capacity curve into 5 parts. Following the outlined criteria (see Tables 1 and 2); the thresholds of the spectral displacements are obtained for the damage limit states.

7 Pushover analysis and capacity evaluation

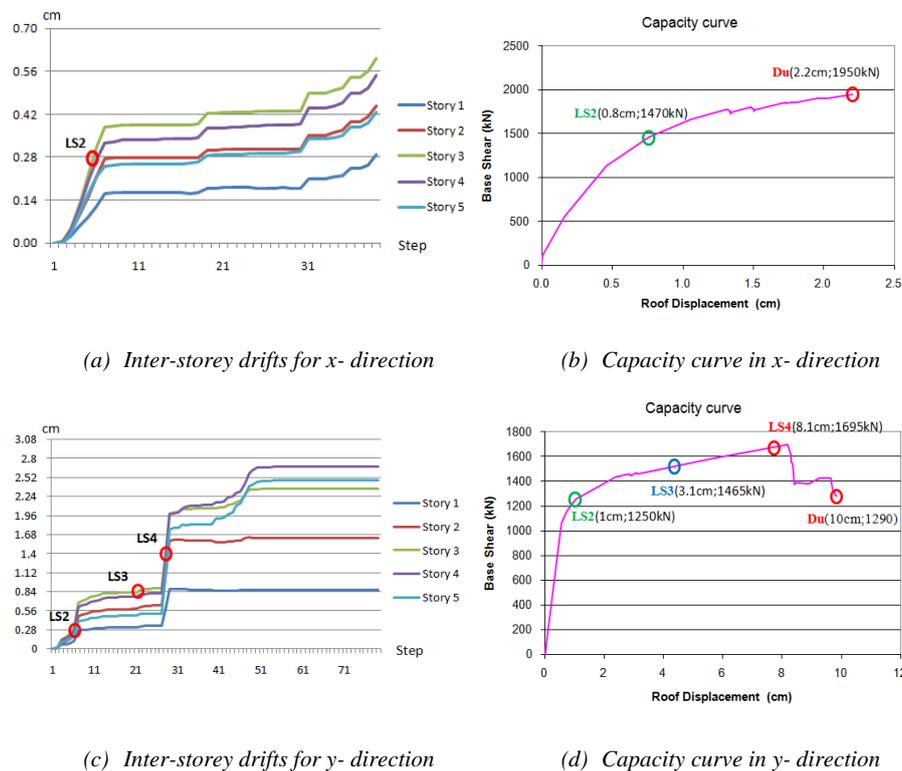
The pushover analysis consists of the application of gravity loads and a representative lateral load pattern. Gravity loads were in place during lateral loading. In all cases, lateral forces were applied monotonically in a step-by-step nonlinear static analysis. The applied lateral forces were proportional to the product of mass and the first mode shape amplitude at each story level under consideration.

In pushover analysis, the response of structure is characterised by a capacity curve that represents the relationship between the base shear force and the displacement of the roof. This useful demonstration is very practical and can easily be visualised by practising engineers. Roof displacement is commonly used for capacity curve.

Capacity evaluation of the investigated residential buildings is performed using damage limit states suggested by Calvi (1999). Pushover analysis data and criteria of Table 1 were used to determine inter-storey drift ratios of each building in both directions. Identification of damage limit states and its representations on capacity curves for each building is given in Figs. 8–10b and d. Small displacement capacities at different performance levels are remarkable for the buildings with greater openings in the respective directions due to failure of masonry elements. Also, TD-83/3 x-direction and TD-72/3 in both directions do not have the expected displacement capacity due to lack of continuous walls (window openings) and irregularity in plans and elevations. The reductions in wall thickness cause a jump in inter-storey drift ratios at the third floor as obviously seen below (Figs. 8–10).

Table 1. Performance levels and criteria provided by Calvi (1999).

Performance level	Performance criteria
LS1	– No damage
LS2 (Minor structural damage and/or moderate non-structural damage)	– Structure can be utilised after the earthquake, without any need for significant strengthening and repair to structural elements. – The suggested drift limit is 0.1 %.
LS3 (Significant structural damage and extensive non-structural damage)	– The building cannot be used after the earthquake without significant repair. Still, repair and strengthening is feasible. – The suggested drift limit is 0.3 %.
LS4 (Collapse)	– Repairing the building is neither possible nor economically reasonable. The structure will have to be demolished after the earthquake. Beyond this LS global collapse with danger for human life has to be expected. – The suggested drift limit is 0.5 %.

**Fig. 8.** Inter-storey drift ratios and capacity curve representation of the TD-83/3 obtained by pushover analysis.

The accurate prediction of inter-storey drift ratio and its distribution along the height of the structures is very critical for the seismic performance evaluation purposes since the structural damage is directly related to this parameter. The inter-storey drift ratios and their corresponding profiles along the height of the template designs are illustrated in Figs. 8–10a and c. As this is the case, the inter-storey drift ratio

over the height of the structures become non-uniform as wall thickness changes.

Pushover analysis data and criteria of Table 2 were used to determine damage limit states according to Lagomarsino and Penna (2003) in both directions. Identification of damage limit states is given Table 3.

The displacement capacity values are solely not meaningful themselves. They need to be compared with displacement

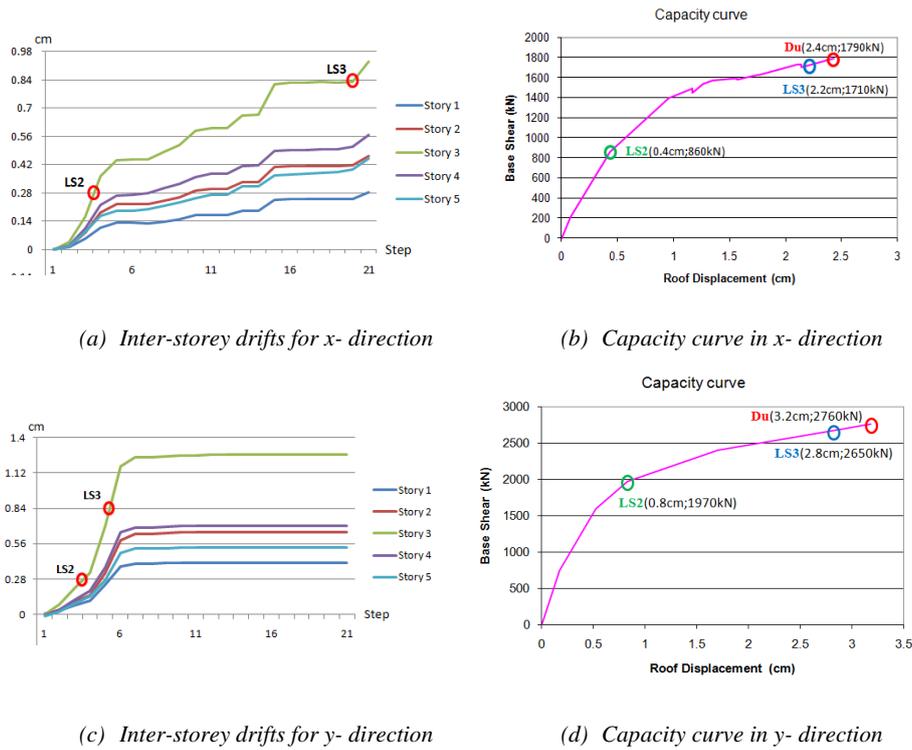


Fig. 9. Inter-storey drift ratios and capacity curve representation of the TD-72/3 obtained by pushover analysis.

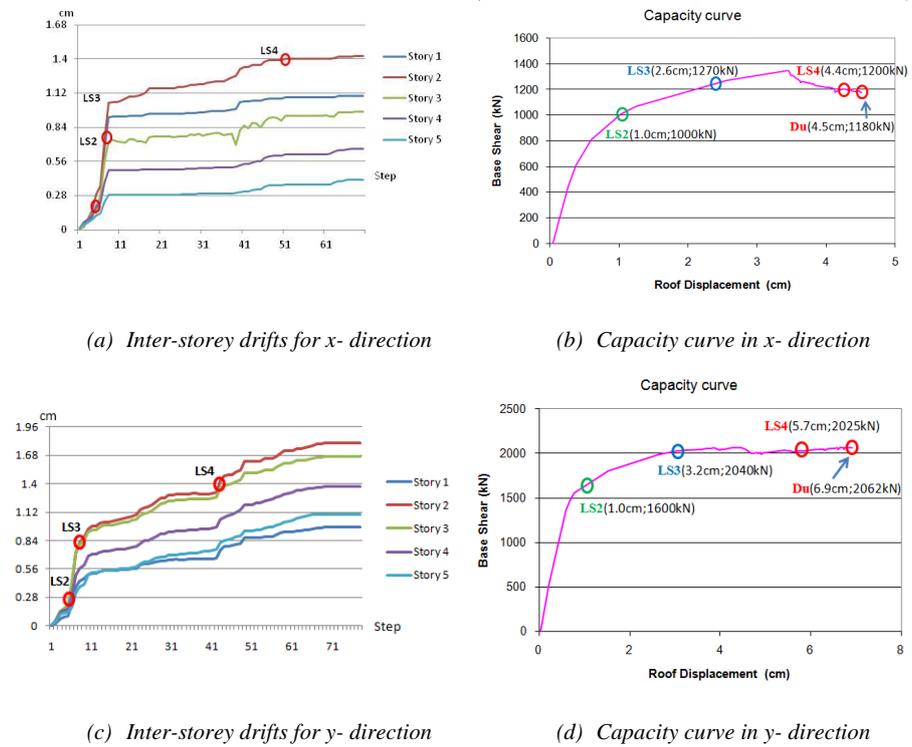


Fig. 10. Inter-storey drift ratios and capacity curve representation of the TD-72/1 obtained by pushover analysis.

Table 2. Performance levels and criteria provided by Lagomarsino and Penna (2003).

Damage state	Spectral displacement, S_d
No damage	$S_d < 0.7 D_y$
Slight	$0.7 D_y < S_d \leq D_y$
Moderate	$D_y < S_d \leq D_y + 0.25(D_u - D_y)$
Extensive	$D_y + 0.25(D_u - D_y) < S_d \leq D_u$
Complete	$S_d > D_u$

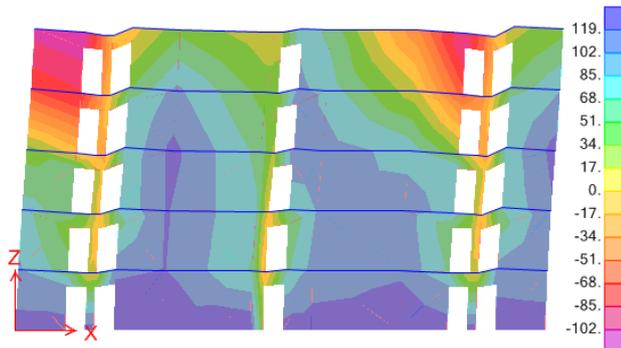


Fig. 11. Ultimate level of shear stress distribution for one wall element of TD-83/3 building obtained from pushover analysis (x-direction – kPa).

demand values. According to modern codes, residential buildings are expected to satisfy a life safety performance level which corresponds to LS3 according to Calvi (1999) under design earthquakes, corresponding to 10% probability of exceedance in 50 yr (Aliaj et al., 2010). Performance point estimates and damage limit states of building capacities corresponding to calculated performance levels are compared to see whether the residential buildings have adequate capacity. Performance points of the structures were obtained as described in FEMA 440. Table 4 lists performance levels of each building under both KTP-N2-89 and EC8 spectrum.

The seismic performance assessment is made for the three template design masonry buildings. For two of them the performance was insufficient and the risk of collapse was very high. Significant damage (LS3) is caused in all the obtained performances. The Albanian and European seismic codes produced different performance levels. European code was mostly unfavorable as expected. Only the third structure may be considered safe under the moderate seismic conditions taken into account. Performance assessment results are summarised in Table 5.

8 Remarks on building responses

TD-83/3 shows a brittle behaviour in x-direction. This is probably because of the greater area of openings in this direction. Even though most of the masonry does not reach their

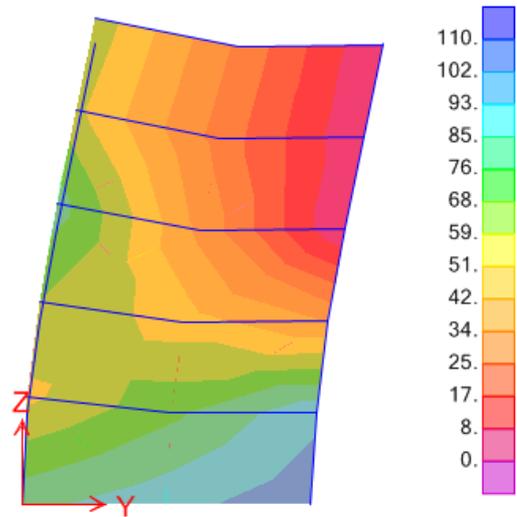


Fig. 12. Ultimate level of shear stress distribution for one pier element of TD-83/3 building obtained from pushover analysis (y-direction – kPa).

ultimate shear capacity, some of the spandrel elements go failure (Fig. 11). The performance point is obtained by using FEMA 440 procedures only for KTP spectrum. Regarding Eurocode 8 spectrum, performance is not reached because of insufficient capacity in this direction. The y-direction behaviour is more ductile. Pier elements are more efficient (Fig. 12). The performance point is found under both spectrums (Table 4). The performance is lower than LS3 damage level for this direction and the damage is moderate in case of this type of earthquake happens.

TD-72/3 has a very brittle behaviour in the x-direction. According to the pushover curve, it fails at the small range of displacements. Due to greater openings and plan irregularity, the performance point is obtained only for KTP-N2-89 spectrum with moderate damage (before LS3 level), but very close to collapse. The peripheral walls carry most of the load and they fail while most of the masonry does not reach ultimate shear resistance. Figure 13 shows a peripheral wall and another one with large openings at failure. The performance in y-direction is better than x- due to the efficiency of pier elements (Fig. 14). Performance point is found and LS3 level is satisfied only for KTP spectrum (Table 4). Although the response in y-direction is more ductile, it is not adequate to satisfy EC8 spectrum.

TD-72/1 shows a ductile behaviour in x-direction. The regularity in plan and elevation makes a good distribution of stresses and increases energy dissipation capacity (Fig. 15). LS3 damage limit state is assured. Due to the low lateral load capacity, extensive damage is expected under EC8 spectrum. Performance point is reached only for EC8 spectrum and it is close to LS3 (Table 4). This building has a higher capacity and resistance in y-direction. Stresses are uniformly

Table 3. Damage limit states according to Lagomarsino and Penna (2003).

Structure Type	Direction	Damage Limit state thresholds (cm)			
		Slight	Moderate	Extensive	Complete
TD-83/3	X	0.53	0.75	1.13	2.20
	Y	0.70	1.00	3.20	9.80
TD-72/3	X	0.63	0.90	1.28	2.40
	Y	0.56	0.80	1.40	3.20
TD-72/1	X	0.42	0.60	1.58	4.50
	Y	0.56	0.80	2.13	6.90

Table 4. Performance points according to Fema 440 for the considered buildings under both spectrums.

Design spectrum	Performance points according to Fema 440						
	Direction	TD-83/3		TD-72/3		TD-72/1	
		Base shear (kN)	Displacement (cm)	Base shear (kN)	Displacement (cm)	Base shear (kN)	Displacement (cm)
Eurocode 8	x	NA	NA	NA	NA	1230	2.3
	y	1430	2.4	NA	NA	1900	2.1
KTP-89	x	1800	1.5	1630	1.8	NA	NA
	y	1430	2.4	2410	1.7	1650	1.1

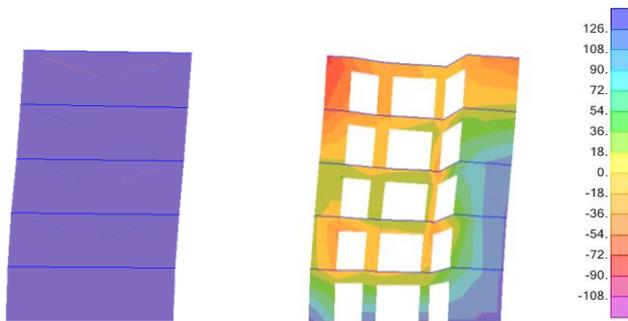


Fig. 13. Ultimate level of shear stress distribution for (a) a peripheral wall element; (b) one wall with opening of TD-72/3 building obtained from pushover analysis (x-direction – kPa)

distributed (Fig. 16) and global response is satisfactory. Performance is found for both spectrums with a medium to high ductility (Table 3). LS3 damage level is satisfied for both spectrums. Moderate damage is caused by both of them, which is higher for EC8 seismic load. Also it is obvious the big distance between the performance point and ultimate LS4 damage level. This means that this structure is safe under both spectrums in y-direction.

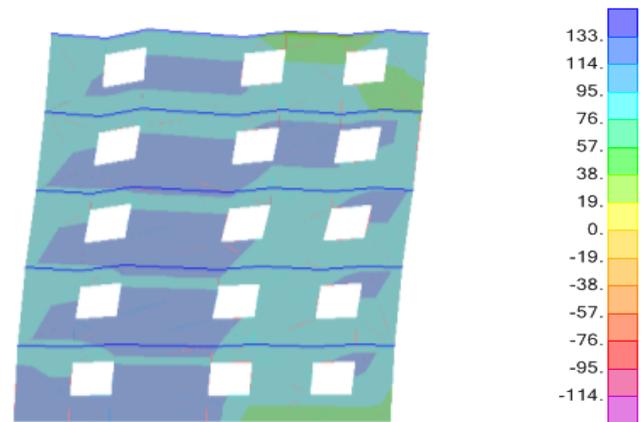


Fig. 14. Ultimate level of shear stress distribution for TD-72/3 building obtained from pushover analysis (y-direction – kPa).

9 Summary and conclusions

This study evaluates seismic capacity of residential buildings with the selected template designs constructed per pre-modern code in Albania considering nonlinear behaviour of masonry. Three residential buildings with template designs were selected to represent an important percentage of residential building stock in mid-sized cities located in the seismic region of Albania. Selection of template designed buildings and material properties were based on field

Table 5. Analysis results for all structure.

Building ID	Direction	KTP-N2-89 Albanian	EC 8	Comment
TD-83/3	x	Safe LS3	Risky*	Low stiffness according to Eurocode
	y	Safe LS3	Safe LS3	Moderate damage under both spectrums
TD-72/3	x	Safe LS3	Risky*	Low stiffness according to Eurocode. Performance is close to collapse for KTP
	y	Safe LS3	Risky*	Low stiffness according to Eurocode but safe for KTP
TD-72/1	x	Safe LS3	Safe LS3	Low stiffness according to KTP but safe for EC
	y	Safe LS3	Safe LS3	Moderate damage under both spectrums

* Risky means no performance is found due to low stiffness.

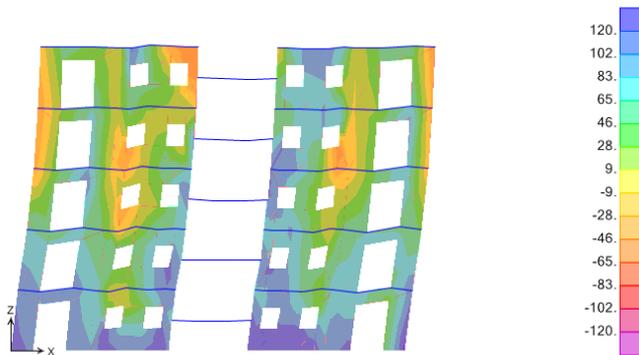


Fig. 15. Ultimate level of shear stress distribution of TD-72/1 building obtained from pushover analysis (x-direction – kPa).

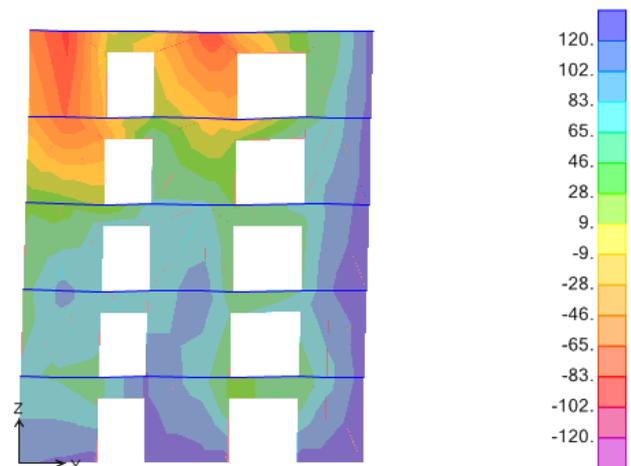


Fig. 16. Ultimate level of shear stress distribution of TD-72/1 building obtained from pushover analysis (y-direction – kPa).

investigation and survey in the governmental archives of Albania. Capacity curves of investigated buildings were determined by pushover analyses conducted in two principal directions. Seismic performance evaluation was carried out in accordance with Fema 440 provisions. Damage limit states thresholds suggested by Calvi (1999) and Lagomarsino and Penna (2003) have been used. Reasons of building damages in past earthquakes are examined using the results of capacity assessment of investigated buildings. Deficiencies and possible solutions to improve the capacity of residential buildings are discussed. The observations and findings of the current study are briefly summarised in the following:

1. Based on archive investigations according to the blueprints of each template design, selected buildings are built with clay bricks of M75 with a resistance of 7.5 MPa and mortar of M25 with a resistance of 2.5 MPa.
2. Evaluation of the capacity curves for the investigated buildings points out that those storeys where the thicknesses of the walls are reduced may cause a deficiency in the seismic performance. Deformation demands are concentrated at the floor where the change occurs. Such abrupt changes in stiffness and strength may lead to failure at the level of change, since the load above and

below the floor is similar. Observations on capacity curves considering the damage limit states thresholds generally being maximum in these stories (inter-storey drift ratios) shows this fact.

3. Regarding the stress distribution and inter-storey drift ratios, stress concentrations and inter-storey drifts are lumped at third story levels where a reduction from second storey to third storey was made. This type of sudden reduction in wall thickness cause deficiencies for the upper part of the building as it is observed in this study. Excessive inter-storey drift and inadequate shear strength may result in moderate to severe damage to these brittle structures. As a conclusion, wall thickness should be reduced in a gradual manner for new buildings.
4. Masonry shear walls are pierced by window and door openings. Above and below the opening, spandrels connect the walls. In direction where significant amount of openings, the capacity curves show the effects of discontinuity in masonry. The observations on the template designs indicated that although windows are located in

both directions, openings are intensified in one orthogonal direction (longitudinal). Considering the fact that amount of openings is a significant parameter in the seismic performance of URM buildings, this situation can cause notable differences in lateral strength and displacement capacities in two orthogonal directions and clearly be observed from the capacity curves. Although the difference on lateral strength capacity is somewhat limited, the difference in displacement capacity is noteworthy. The displacement capacity in direction where the number of openings is intensified is less than half of that other. This study shows that the considered amount of openings decreases energy dissipation capacity by around 50% and therefore increases the sustained damage for this type of buildings. This might cause a deficiency in a probable future earthquake and preventive measures should be taken urgently.

5. The magnitude of maximum inter-storey drift ratios and the distribution of this ratio over the height of the all structures are very similar since the effects of higher modes are negligible and the response is primarily governed by the fundamental mode.
6. Recalling that these template structures were designed and constructed according to force based design approximations at the date of construction, such kind of deformation based deficiency (reduction of wall thickness and its effects on performance) may not be captured by means of force based evaluation procedures. On the other hand, performance based seismic assessment procedures are useful tools to correctly predict the deficiencies in this type of masonry constructions as in the case of framed structures.
7. In this study, two types of damage limits state definitions suggested by Calvi (1999) and Lagomarsino and Penna (2003) for the performance assessment of buildings have been taken into consideration. In the first approach, inter-storey drift ratios are used as damage limit states, whereas second one is useful if the assessment procedure is limited for global response prediction. For the studied buildings, while the second approach can give useful information for the global response of the buildings, it is incapable of representing the effects of change in wall thickness and its effects on seismic performance. Considering that all the template designs have wall thickness reduction from second to third story, first method seems more convenient for capturing such kind of deficiencies.
8. The observed building damages for URM structures during the past earthquakes support the analytical results obtained in this study; the reports and studies (Jagadish et al., 2003; Kaplan et al., 2010; Klingner, 2006; Tomazevic, 1999; Yoshimura and Kuroki, 2001) from past earthquakes pointed out masonry facades with numerous spandrels and between those spandrels failed due to shear. Stress concentrations due to shear in spandrels observed in pushover analyses for these constructions are clear indicators of such failures and potential risk in existing URM buildings for future earthquakes.
9. Shear failure of masonry piers seems the most frequent failure mechanism of URM buildings in the past earthquakes and the pushover analyses results support this fact. However, with regard to the ductility and energy dissipation capacity this mechanism is not favourable. Non-ductile behaviour of weak piers could be improved by means of adequately distributed bed joint reinforcements.
10. According to performance evaluation, template designs, except the TD 72/1 building, are far from satisfying the expected performance levels, suggesting that urgent planning and response need to be in action.

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