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Effects of structural irregularities on low and mid-rise RC building response

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ABSTRACT

During the recent earthquakes, it has been observed that structural irregularities are one of the main reasons of the building damage. Irregularities are weak points in a building which may cause failure of one element or total collapse of the building during an earthquake. Since Albania is a country with moderate seismicity which has been hit by earthquakes of different magnitudes many times establishes the need to study the effect of irregularities is well-founded. The main structural irregularities encountered in Albanian construction practice consist of short column, large and heavy overhangs and soft story. In this study, these types of irregularities are considered in two different types of buildings, low and mid-rise reinforced concrete frame buildings represented by 3- and 6- story respectively. Pushover analyses are deployed to get the effect of structural irregularities on RC building response. A building set is chosen to represent the existing construction practice in the region; regular framed building and buildings with irregularities such as soft stories, short columns, heavy overhangs and the presence of soft story with heavy overhangs. The analyses have been conducted by using ETABS and Seismosoft software. Pushover curves of building set are determined by nonlinear static analysis in two orthogonal directions. Comparative performance evaluations are done by considering EC8 and Albanian Seismic codes (KTP-N2-89). From the obtained results, it is observed that low and mid-rise structures with soft story- two sided overhangs and short column are more vulnerable during earthquakes.

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1. Introduction

The inadequate performance and the huge number of collapsed buildings during past earthquakes due to different structural irregularities determines the idea to analyze the buildings with dissimilar irregularities in order to understand the effect of irregularities on reinforced concrete (RC) buildings under seismic effects (Varadharajan, 2014; Altuntop, 2007; Dolsek and Fajfar, 2000; Inel and Ozmen, 2008; Uruci and Bilgin, 2016; Sattar and Liel, 2010; Apostolska et al., 2010; Sonmez, 2013; Vahidi and Malekabadi, 2009; Tena-Colunga, 2004). Structural irregularities have a significant effect on the response of RC buildings during an earthquake. In order to prevent possible damages caused by structural irregularities, seismic demand must be determined accurately. Several researchers and academicians (Varadharajan, 2014; Altuntop, 2007), have studied altered vertical and horizontal irregularities with different methods of analysis such: nonlinear static pushover analysis and time history analysis, etc. and realized which type of irregularities are riskier during an earthquake and what should be taken in consideration during the design process (Bachmann, 2002; Semnani et al., 2014). Albanian building stock is mostly composed of RC and masonry buildings. Most of these buildings are designed according to earlier versions of Albanian Seismic Codes (KTP-

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N2-89, 1989) and some of them were constructed without a definite construction project. Considering these facts and the observations done in Albanian construction industry, presence of structural irregularities is very common in these buildings. The aim of this study is to evaluate the seismic performance of 3- and 6- story RC buildings representing the low and mid-rise building stock of Albanian construction practice, in which soft story, short column and heavy overhangs irregularities are imposed, in order to evaluate the effects of structural irregularities on RC frame structure response. Effect of structural irregularities and performance of the considered frames are assessed by using capacity curves of the frames.

2. Case Study

2.1. Description of the building set

Two different 3- and 6- story RC structures are considered to represent reference low and mid-rise buildings in the region in this study. The selected buildings are typical RC frame buildings with no shear walls. Selected buildings have the same plan view, 20m by 16 m in plan. Both have 5 bays by 4m along x direction and 4 bays by 4m along y direction as shown in Figs. 1-2. Typical floor height for both frames is 3m. The location of masonry infill walls in plan is shown by the hatch of beams for both structures, Figs. 1-2.

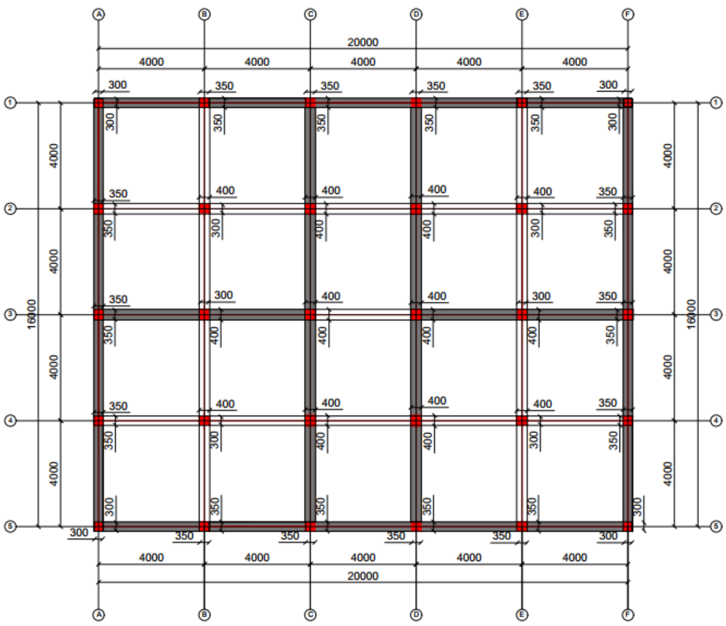

Fig. 1. Structural plan view of the 3-story frame (units in mm).

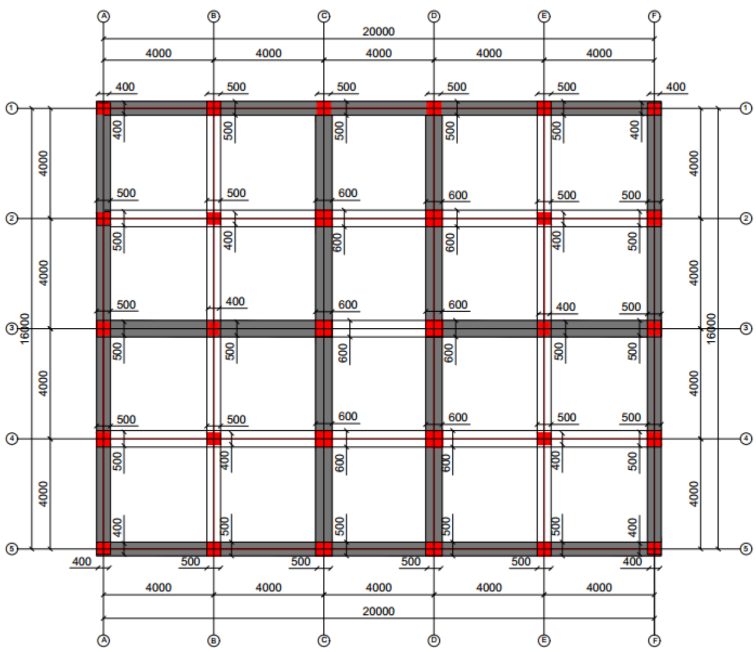

Fig. 2. Structural plan view of the 6-story frame (units in mm).

Beam and column dimensions of the reference buildings represent the most common frame elements for low and mid-rise frames in Albanian construction practice. The 3- story frame consists of 300mm x 300mm and 350mm x 350mm outside columns, identified as C1 and C2 respectively, 300mm x 400mm, 400mm x 300mm and 400mm x 400 mm inside columns, identified as C5, C4 and C3 respectively, as shown in Figure 3. Beams have all the same section for the 3- story frame which consists of 300mm x 400 mm (Fig. 3). The 6- story frame consists of 400mm x 400mm and 500mm x 500mm outside columns, and 400mm x 500mm, 500mm x 400mm and 400mm x 600 mm inside columns, identified as C9, C8 and C7 respectively, as shown in Figure 4. All the beams of the 6-story frame have the same cross section of 300mm x 500mm Fig. 4. The transverse reinforcement is represented 100mm by spacing in order to reflect the ductile detailing. The selected reference buildings do not have any vertical or horizontal irregularity (short columns, soft story, overhangs, etc.)

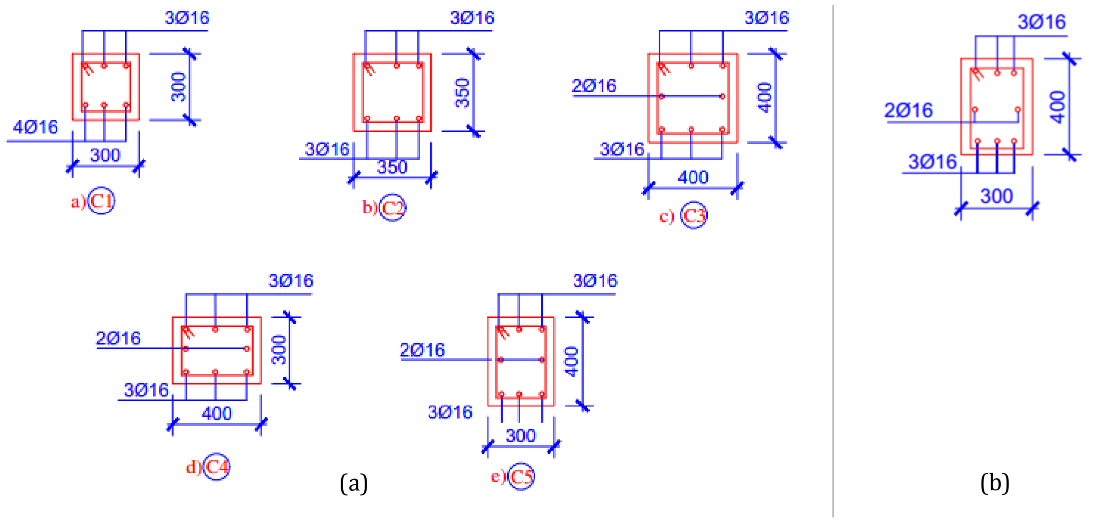

Fig. 3. a) Column and, b) Beam reinforcement details for 3-story frame (units in mm).

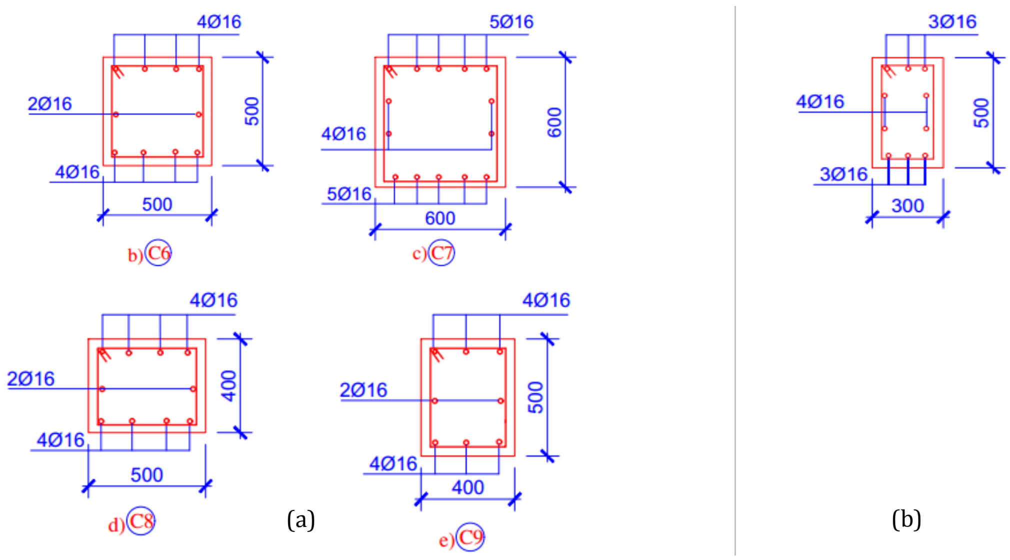

Fig. 4. (a) Column and, (b) Beam reinforcement details for 6-story frame (units in mm).

Soft story in most of the cases happens because of the lower stiffness of the first story of the buildings which comes as a result of fewer amounts of masonry infill walls or because the first story may have greater height compared to the other ones because of commercial reasons (Altuntop, 2007; Dolsek and Fajfar, 2000; Inel and Ozmen, 2008; Uruci and Bilgin, 2016; Sattar and Liel, 2010; Apostolska et al., 2010; Sonmez, 2013). In this study both cases are taken in consideration for the two types of structures, low and mid-rise buildings. In the first story of the selected frames the masonry infill walls are removed, and the story height is done 4.5m instead of 3m normal height, Fig. 5(b). Short column may be formed because of different situations like band windows, mid story beams at the stairway shafts in buildings, semi-infilled frames, etc. (Vahidi and Malekabadi, 2009). In this study, short column is created by semi-infilled frames. As seen in the Fig. 5(d), because of two semi-infilled bays four columns have become short. Heavy overhangs shift the buildings mass centre upwards and take it away from centre of rigidity. Thus, it has negative effects on seismic behaviour. Past earthquakes revealed that buildings with heavy overhangs are more vulnerable to damage (Tena-Colunga, 2004). In this study are modelled overhangs at two cross sides of a building. For this purpose, 1.5m overhangs are attached to the regular building sides Fig. 5(c). The wall loadings are shuffled on the beams nearby the overhang portion. All the building set with different configurations are shown in Table 1.

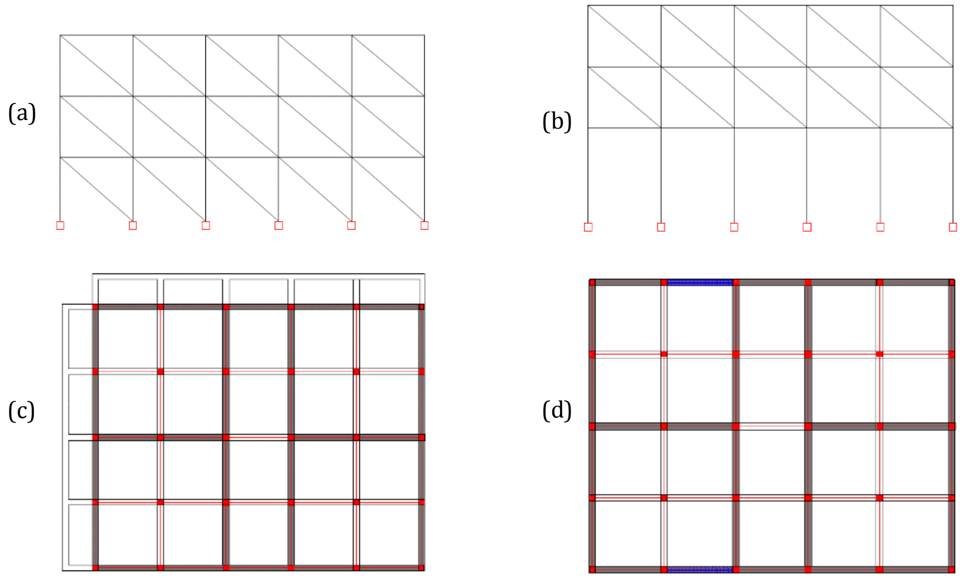

Fig. 5. (a) Reference frame; (b) Soft story; (c) Two-sided overhang; (d) Short column.

Table 1. Building set (3- and 6- story frames).

Ref	Reference Building (without any irregularity), Figure 5(a).
TSO	Two-sided overhang building, Fig. 5(c).
SSH	Soft story due to 4.5 m ground story height (instead of 3 m), Fig. 5(b).
SSW	Soft story due to absence of masonry infill wall at ground story, Fig. 5(b).
SS-H-W	Soft story due to both height and infill effect, Fig. 5(b).
SS-H-W-TSO	Soft story due to both height and infill, and two-sided overhang, Fig. 5(c).
SHC	Short column due to semi-infilled bays at ground story, Fig. 5(d).

* These models are considered for both 3- and 6- story buildings.

2.2. Material properties

Material properties are based on most common materials used in Albanian construction practice; it is assumed 20 MPa for the concrete compressive strength and 355 MPa for the yield strength of reinforcement. Then in order to get the effect of structural irregularities in reinforced concrete structures the selected 3 and 6 story buildings, Fig. 5(a), are modified to have one or more of the above-mentioned structural deficiencies: soft story, short column, overhangs observed in last earthquakes.

2.3. Modeling

Modelling of the considered frames in ETABS is done in similar way for all of them with small changes while implementing the considered structural irregularities. In the below section a step by step analysis is explained for the Ref 3 story frame.

Firstly material and frame sections are defined in accordance with the properties defined in section 2.2. Then the selected frames are modelled by using ETABS software, in Fig. 6 below is shown elevation view of the 'Ref' 3-story reinforced concrete frame.

The model in Fig. 6 is formed by beam and columns whose joints connected to the ground story are made fixed supports in order to be restrained in all directions.

Masonry infill walls include partially of fully panels within the plane of concrete frames, which are bounded by columns and beams. Masonry infill walls are modelled as diagonal strut elements with:

Modulus of elasticity = 1000 MPa
Compressive strength =1 MPa
Shear strength = 0.15 MPa

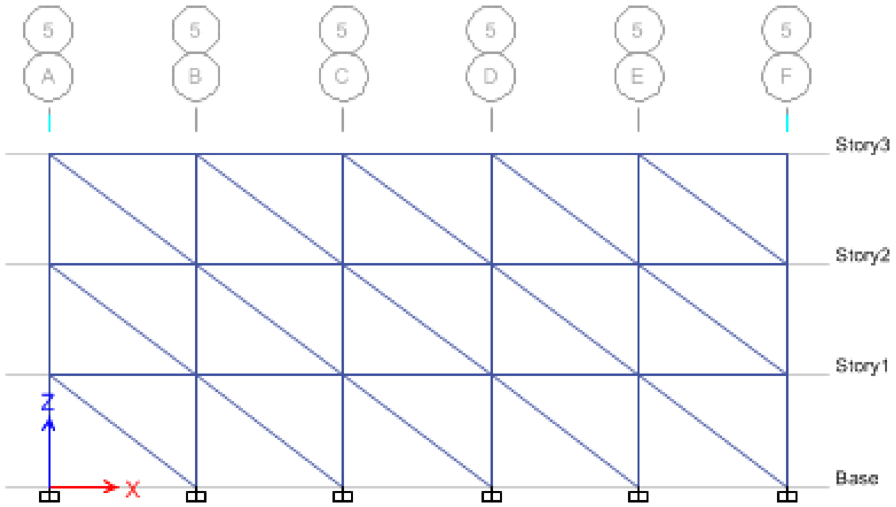

Fig. 6. Elevation view of the Ref 3-story frame.

The elastic stiffness of a masonry infill panel is represented by an equivalent diagonal compression strut with width "a" as in Eq. (1) below. The strut have the same thickness and modulus of elasticity as the infill panel it represents:

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{inf} \,, \tag{1}$$

where,

$$\lambda_1 = \left[\frac{E_{me}\, t_{inf} \sin 2\theta}{4E_{fe}\, I_{col}\, h_{inf}}\right]^{\frac{1}{4}} , \tag{2}$$

h_{inf} = Height of infill panel,
E_{fe} = Expected modulus of elasticity of frame material,
E_{me} = Expected modulus of elasticity of infill material,
I_{col} = Moment of inertia of column,
L_{inf} = Length of infill panel,
r_{inf} = Diagonal length of infill panel,
t_{inf} = Thickness of infill panel and equivalent strut,
θ = Angle whose tangent is the infill height to-length aspect ratio, radians,
λ_1 = Coefficient used to determine equivalent width of infill strut.

In case of non-composite infill panels only the panels in direct contact with frame elements should be taken in consideration while determining the in plane stiffness. In plane lateral stiffness is not the sum of the panel and infill stiffness because of the interaction of the infill with the frame. From the tests it is seen that during an earthquake the infill tends to separate from the frame making possible for the compressive stresses to be created. So masonry infill panels could be represented by a single equivalent strut as shown in Fig. 7, for which if thickness and modulus of elasticity are assumed the same as those of the masonry just width is needed to be determined.

Transfer of forces from one story to another in a masonry infilled frame incorporated with concrete or steel should be considered a deformation-controlled action. Expected shear strength of the in-plane panels should be determined with the Eq. (3) as follows:

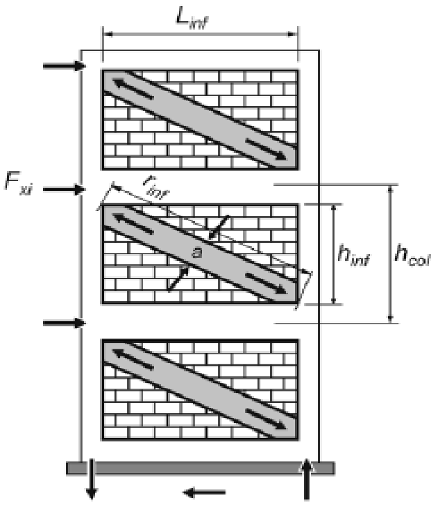

Fig. 7. Masonry infill walls as single equivalent diagonal strut.

$$Q_{CE} = V_{ine} = A_{ni}\, f_{vie} \,, \tag{3}$$

where,
A_{ni} = Area of net mortared/grouted section across infill panel,
f_{vie} = Expected shear strength of masonry infill.

In Eq. (3), expected shear strength should not exceed the expected masonry bed-joint shear strength.

After modelling the masonry infill walls, load patterns are defined:

- Dead load
- Live load
- Dead load from slabs
- Dead loads of infill walls

Self-weight multiplier of dead loads from infill walls and slabs are taken as 0. Since after the linear analysis the nonlinear analysis is performed and the slabs are not considered in this case for simplicity in calculations, loads are directly assigned as a uniformly distributed loads on beam. After assigning all the considered loads, their combination is done. Two load combinations are considered: Load combinations in this linear analysis will be two:

- 1.4 DD (dead loads) + 1.6 LL (live loads)
- DL (dead loads) + 0.3 LL (live loads)

After defining the load combinations the linear analysis is carried out. By choosing Modal case, dynamic characteristics of the buildings are obtained for the first 9 first modes.

Nonlinear static pushover analysis is a type of analysis which is performed by subjecting a monotonically increasing pattern of lateral loads in the structure which represents the forces that the structure may experience during an earthquake. Under incrementally increasing loads various structural elements may yield sequentially. Consequently, at each event, the structure experiences a loss in stiffness. Using a pushover analysis, a characteristic non-linear force displacement relationship can be determined. Guidelines like FEMA 356 (2000) have mentioned the modelling procedures, acceptance criteria and analysis procedures for the pushover analysis.

Nonlinear properties of the frame elements are assigned as plastic hinges:

- Beam: Plastic hinges are assigned to the start, 0, and end, 1, point of the beam as specified in FEMA 356. Beams will be released in rotational moment M3.
- Columns: Plastic hinges for columns with have different degree of freedom, axial force and rotational moment in both directions, P-M2-M3.

After assigning the plastic hinges the load cases for the nonlinear static pushover analysis is defined: Firstly the "Push Combo Case" is defined as nonlinear case in which are included the dead loads, dead loads from slabs, infill walls and 30% of live loads. Pushover Analysis is performed in two orthogonal directions. P-Delta effect is taken in consideration. Results are obtained in multiples steps. For the displacement control the maximum displacement at the top of the building is considered. After defining all load cases for the nonlinear analysis, the rigid diaphragms are assigned to each story in order to concentrate the story weight in center of mass.

2.4. Modelling in Seismosoft

Seismosoft is a finite element software used for structural analysis, being able to predict large displacement behaviour of space frames under static or dynamic loadings. The Ref frame is also analysed by Sesimosoft software in order to see the difference between two programs and compare the behaviour of considered reinforced concrete frames.

Firstly material properties and frame sections are defined as shown in Fig. 8 below.

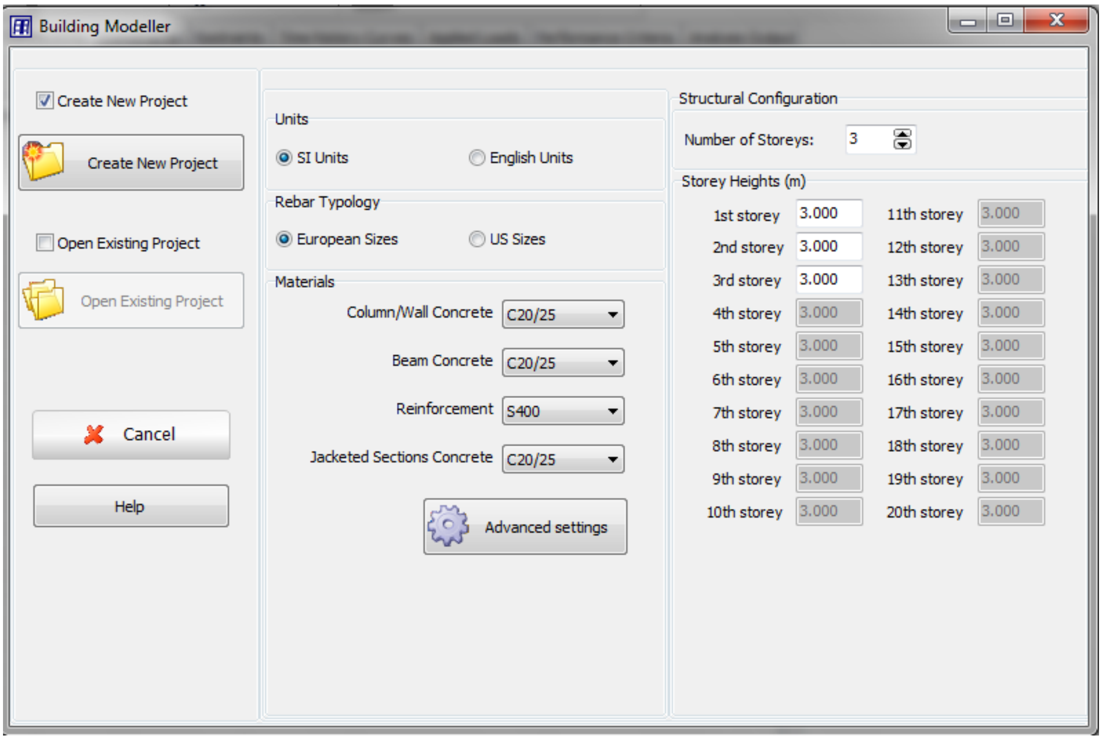

Fig. 8. Defining material properties and frame sections.

After defining the materials, analysis and loading type are chosen, in this case static pushover analysis and uniform distribution respectively. Frame elements are modelled as inelastic plastic hinge force based frame elements. Loading combination factors is 1 for dead loads and 0.3 for live loads. Then the modelling of the Ref frame for the 3 story case is done as shown in Fig. 9.

After modelling the frame elements, masonry infill walls are modelled as diagonal strut elements, in accordance with the structural plan of the 3 story frame where location of masonry infill walls is shown. After modelling of all elements and specifying the loads and pushover analysis parameters the analysis is run out and the results are generated.

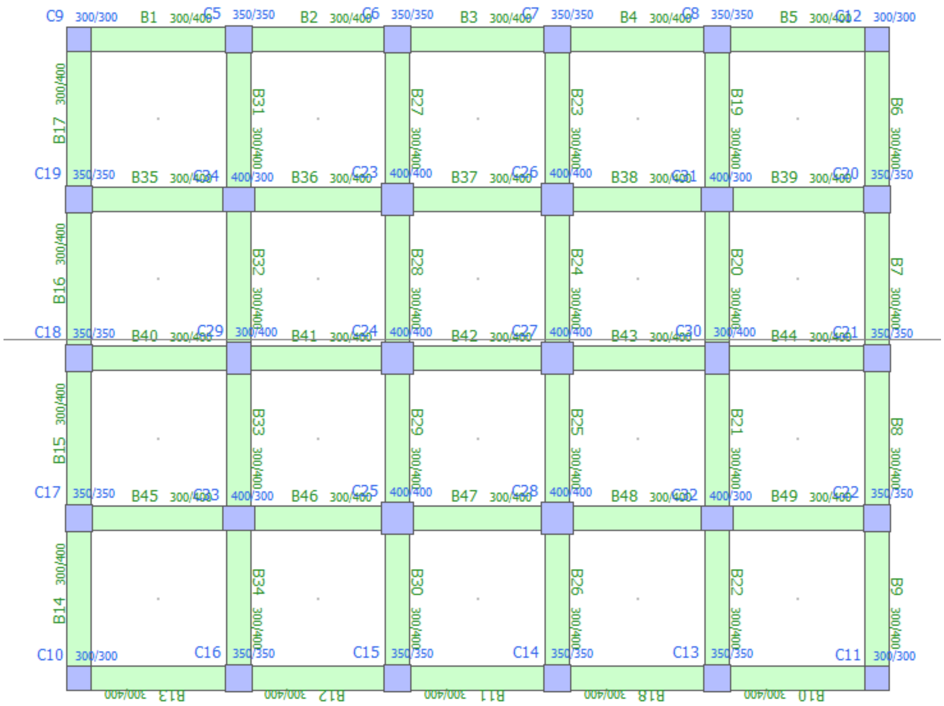

Fig. 9. Plan view of 3 story Ref frame.

3. Structural Analysis

Nonlinear static pushover analysis is a type of analysis which is performed by subjecting a monotonically increasing lateral load patterns in the structure which represents the forces that the structure may experience during an earthquake. Under incrementally increasing loads various structural elements may yield sequentially. Consequently, at each event, the structure experiences a loss in stiffness. Using a pushover analysis, a characteristic non-linear force displacement relationship can be determined. Guidelines like FEMA 356 have mentioned the modelling procedures, acceptance criteria and analysis procedures for the pushover analysis (FEMA-356, 2000). This guideline defines the force-deformation criteria for possible locations of lumped inelastic behavior defined as plastic hinges in the pushover analysis.

In this study, pushover analysis has been conducted for the 14 building models (Table 1) for both type of structures, 3- and 6-story buildings. The material nonlinearities are assigned as plastic hinges; release in rotational moment M3 for flexural hinges for beams and axial force, rotational moment in both directions P-M2-M3 flexural hinges for columns. Infill panels are modelled by one nonlinear strut elements, which only has compressive strength. Then each lateral load pattern is applied and static pushover analyses results of the case study buildings are generated. Behaviour of the structure is represented by capacity curves that represents the base shear force and displacement of the roof. Figs. 10(a-d) illustrates capacity curves obtained from the pushover analysis of the 3 and 6-story frames. In *x*- axis is shown the roof drift ratio that is roof displacement normalized by the building height and in *y*- axis is shown the shear strength coefficient that is the base shear force normalized by the seismic weight.

In the below graphs, Figs. 10(a-d) are shown the normalized graphs of the Ref 3- and 6- story frames, analysed with both software's ETABS and Seismosoft.

From the graphs, it is seen that the programs have generated almost the same capacity curves for both reference frames, 3- and 6- stories. For the rest of the analyses of the other frames are done by using just ETABS software.

In the below graphs, Figs. 11(a-d), are shown the capacity curves of the considered 3- and 6- story regular frames and frames with structural irregularities. From the normalized graphs, presence of structural irregularities effects the seismic performance of the frame, it both weakens and softens the system.

Soft story due to absence of masonry infill walls at the ground story is found to be more damaging than the soft story due to greater height of the ground story in both cases low and mid-rise buildings, 3-and 6-story respectively. Soft story due to absence of infill has shown approximately 33% lower stiffness, 25% lower strength than soft story due to higher story height and 54% lower stiffness, 30% lower strength than the Ref building, for the 3- story frame in x direction. Soft story due to lack of masonry infill walls for the 6- story frame in *x* direction has shown approximately 19% higher stiffness, 25% lower strength than soft story due to higher story height and 3% lower stiffness, 25% lower strength than the Ref building. But the most unfavourable case is soft story due to both absence of infill walls and higher height of the ground story. The capacity curve of 6-story SS-H-W building has shown approximately 81% lower strength and 19% lower stiffness than Ref 6 story building, and capacity curve of 3 story SS-H-W building has shown 61.3% lower strength and 62.5% lower stiffness than capacity curve of Ref 3 story building in *y* direction.

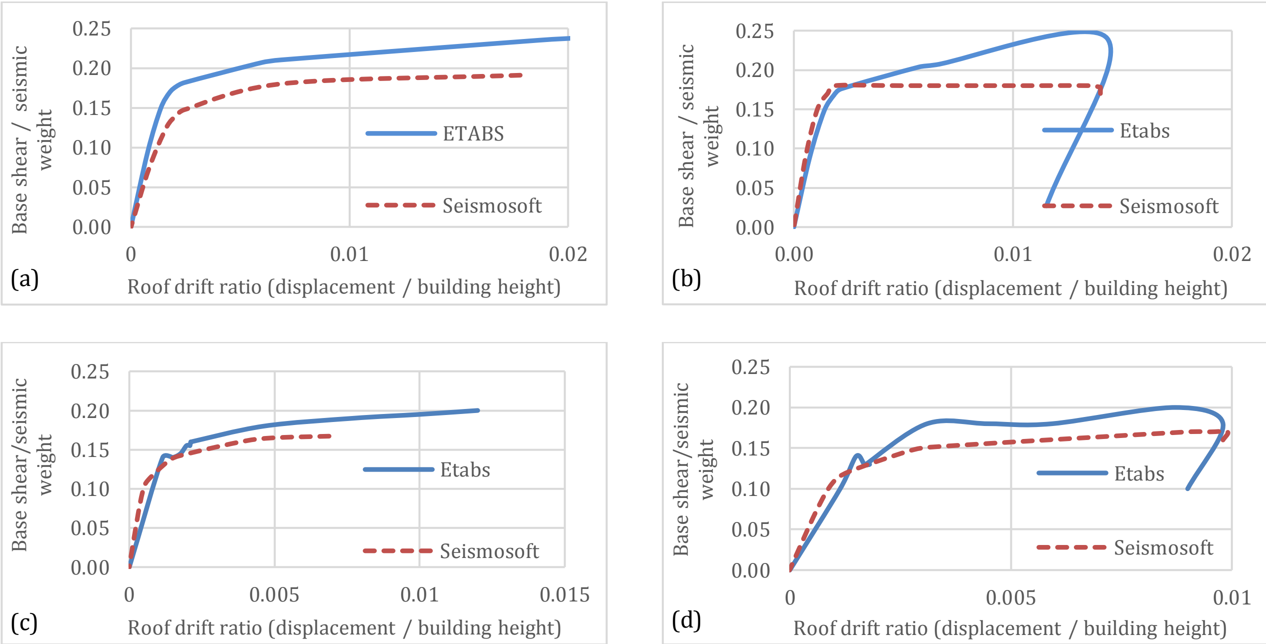

Fig. 10. Comparison between Etabs and Seismosoft analysis results:
(a) Ref 3-story, *x*-direction; (b) Ref 3-story; *y*-direction; (c) Ref 6-story, *x*-direction; (d) Ref 6-story, *y*-direction.

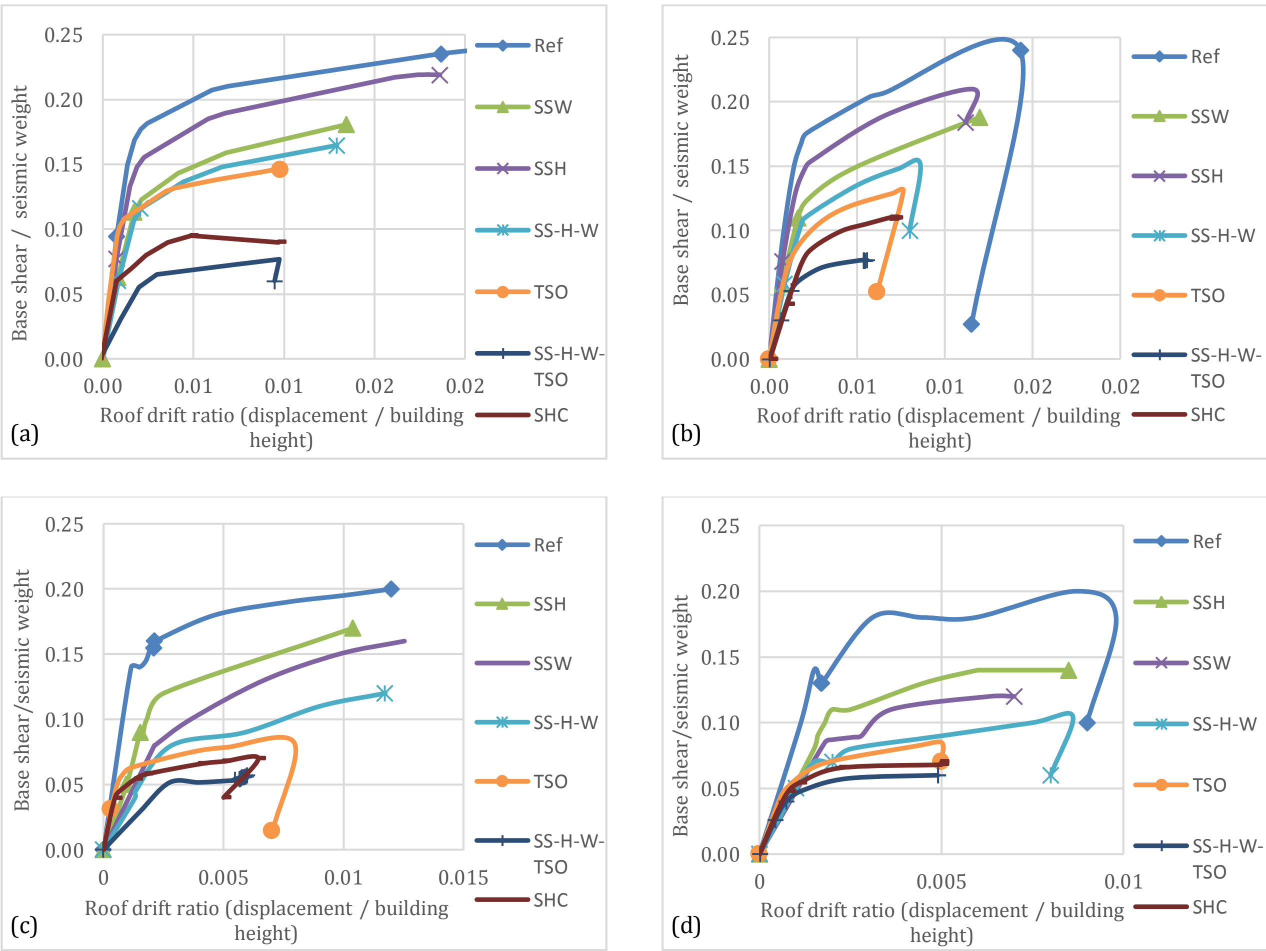

Fig. 11. Capacity curves of: (a) 3-story frames *x*-direction; (b) 3-story frames *y*-direction;
(c) 6-story frames *y*-direction; (d)6- story frames, *y*-direction.

Two sided overhang irregularity is found to be more damaging than soft story irregularity, it lowers more the performance of the building. Capacity curve of the 3 story TSO frame in *x* direction shows 60% lower strength and 100% lower stiffness that the Ref frame in *x* direction. For the 6 story TSO frame it shows 78.5% lower stiffness and 150% lower strength in comparison with Ref 6 story frame in *x* direction.

Soft story with two sided overhang irregularity is found to be more damaging than soft story, short column and two sided overhang irregularities. Capacity curve of the 3 story SS-H-W-TSO in *x* direction shows 182.4% lower strength and 100% lower stiffness than the Ref frame in x direction. The 3 story SS-H-W-TSO in y direction has shown 212.5% lower strength and 116.7% lower stiffness than the Ref frame in Y direction. For the 6 story case it shows 92% lower stiffness, 300% lower strength and 90% lower stiffness and 233% lower strength than 6 story Ref frame, for *x* and *y* direction respectively.

Short column irregularity both softens and weakness the system as shown in the comparison graphs above, Figs. 11(a-d). Capacity curve of the 3 story SHC frame shows 100% lower stiffness, 152.6% lower strength and 62.5% lower stiffness and 127% lower strength than 3 story Ref frame, for x and y direction respectively. Capacity curve of the 6 story SHC frame shows 92.3% lower stiffness, 207% lower strength and 90% lower stiffness and 185% lower strength than 6 story Ref frame, for *x* and *y* directions respectively. From the results it is observed that SS-H-W-TSO and SHC frames are more vulnerable during earthquakes similar to other studies (Inel et al., 2007).

In Table 2 below is shown the summary of the results, stiffness and strength of each one of the considered frames compared to Ref 3 and 6 story frames respectively. From the results it is observed that SS-H-W-TSO and SHC frames are more vulnerable during earthquakes.

Table 2. Comparison of stiffness and strength capacities with Ref frame.

	3-Story frames in x direction	Stiffness in comparison with Ref frame	Strength in comparison with Ref frame
3-Story frames in *x* direction	Ref	=%	=%
	TSO	<100%	<60%
	SSH	<11.1%	<9.1%
	SSW	<53.8%	<29.7%
	SS-H-W	<53.8%	<45.5%
	SS-H-W-TSO	**<100%**	**<182.4%**
	SHC	<100%	<152.6%
3-Story frames in *y* direction	Ref	=%	=%
	TSO	<73.3%	<85.2%
	SSH	<8.3%	<19.0%
	SSW	<8.3%	<31.5%
	SS-H-W	<62.5%	<61.3%
	SS-H-W-TSO	**<116.7%**	**<212.5%**
	SHC	<62.5%	<127.0%
6-Story frames in *x* direction	Ref	=%	=%
	TSO	<78.5%	<150%
	SSH	>11%	<21%
	SSW	>3%	<25%
	SS-H-W	=%	<60%
	SS-H-W-TSO	**<92.3%**	**<300%**
	SHC	<92.3%	<207%
6-Story frames in *y* direction	Ref	=%	=%
	TSO	<90%	<135.3%
	SSH	<11.8%	<35.1%
	SSW	<35.7%	<60%
	SS-H-W	<18.8%	<81%
	SS-H-W-TSO	**<90%**	**<233%**
	SHC	<90%	<185%

3.1. Capacity spectrum method

In order to get the performance point of each of the considered frames, the capacity spectrum method is used. Capacity curve is expressed in terms of spectral displacement and spectral acceleration, positioned in *x*- and *y*- axis, respectively. Then performance point is the intersection of the capacity curve with a modified response spectrum (ATC-40, 1996). Performance point on a capacity curve can also be determined by the ETABS program for a specified elastic spectrum.

In order to get the performance point of the considered frames two response spectrums are used considering two different Codes, Eurocode 8 (2004) and Seismic Albanian Codes (KTP-N2-89, 1989). Parameters of the considered Response spectrum consist of:

Acceleration - 0.25g; soil type – C; behaviour factor (q) = 4, spectrum type -1

In Fig. 12 it is seen that for the considered parameters two different response spectrums are generated based on Eurocode 8 (2004) and KTP Codes (KTP-N2-89, 1989). Demand spectrum is based on elastic response which is divided with damping reduction factors C_a and C_v factors which are achieved during constant acceleration and velocity respectively. In order to achieve the reduced response spectrum the below Table 3, is considered in which are given the correspondences between selected response spectrums parameters and reduction factors.

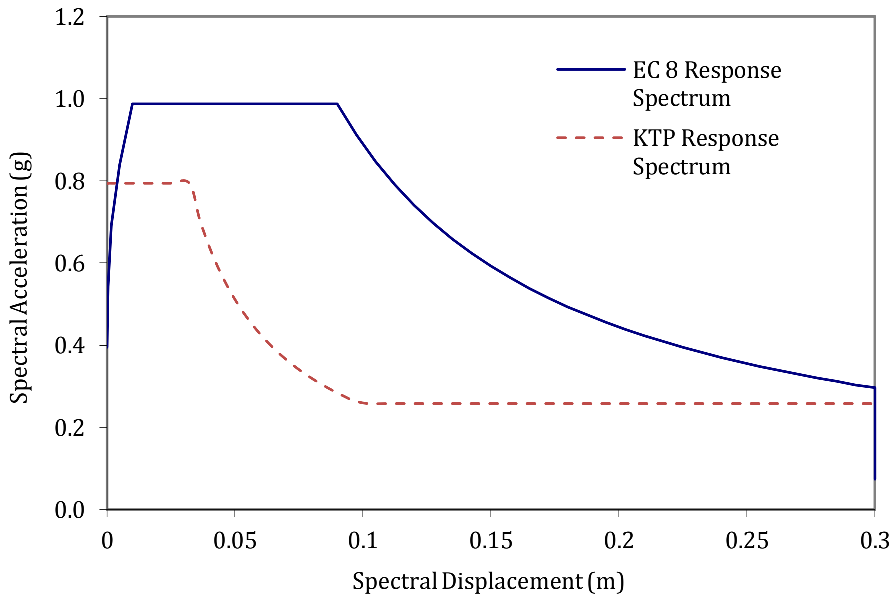

Fig. 12. Considered response spectrums for EC 8 and KTP.

Table 3. Seismic value correspondences (ATC-40, 1996).

Cases	C_a	C_v		A_0	S(T)
a	0.4	0.3	Correspond to	0.40	For soil type 1, T_b=0.3
b	0.4	0.4		0.40	For soil type 2, T_b=0.4
c	0.4	0.6		0.40	For soil type 3, T_b=0.6

*A_0- Seismic zone coefficient

In the study “c” case is considered with reduction factors C_a=0.4 and C_v=0.6 which correspond to soil type 3 and T_b=0.6.

In the below graphs Figs. 13-16 are shown the performance points of the Ref 3 and 6-story frame considering the spectrum from both Codes (EC 8 and KTP), intersection between capacity curve and demand spectrum.

For the 3-story Ref frame in *x* direction, using EC 8, performance point is found at S_a=0.23 and S_d=0.14.

For the 3-story Ref frame in *x* direction, using KTP response spectrum, performance point is found at S_a=0.18 and S_d=0.08.

For the 6-story Ref frame in *x* direction, using EC 8, performance point is found at S_a=0.19 and S_d=0.165. For the 6-story Ref frame in *x* direction, using KTP response spectrum, performance point is found at S_a=0.18 and S_d=0.12.

4. Conclusions

This study assesses the seismic performance of low and mid-rise frames represented by 3- and 6- story frames, respectively. These frames represent the major building stock in Albanian construction industry. Seismic performance of the considered frames is assessed by considering nonlinear behaviour of reinforced concrete components and masonry infill walls.

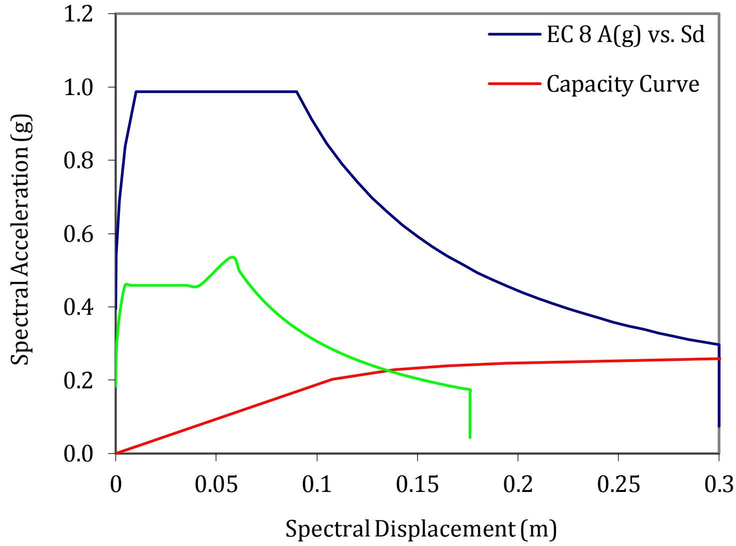

Fig. 13. Performance point of Ref 3 story *x*-direction frame EC 8.

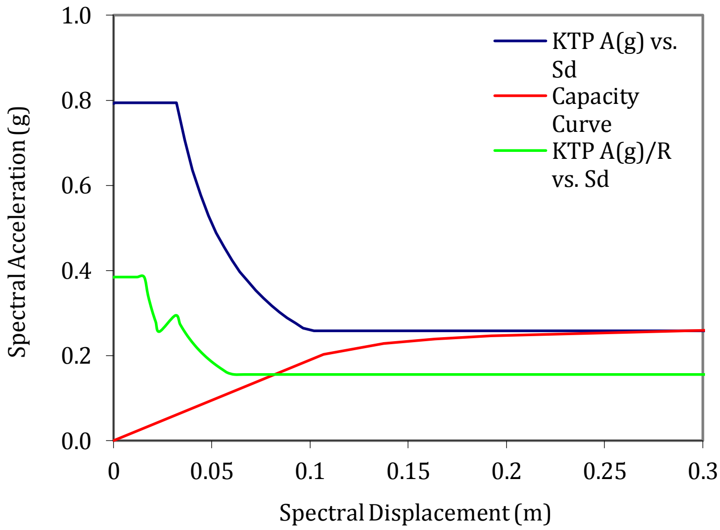

Fig. 14. Performance point of Ref 3 story *x*-direction frame KTP.

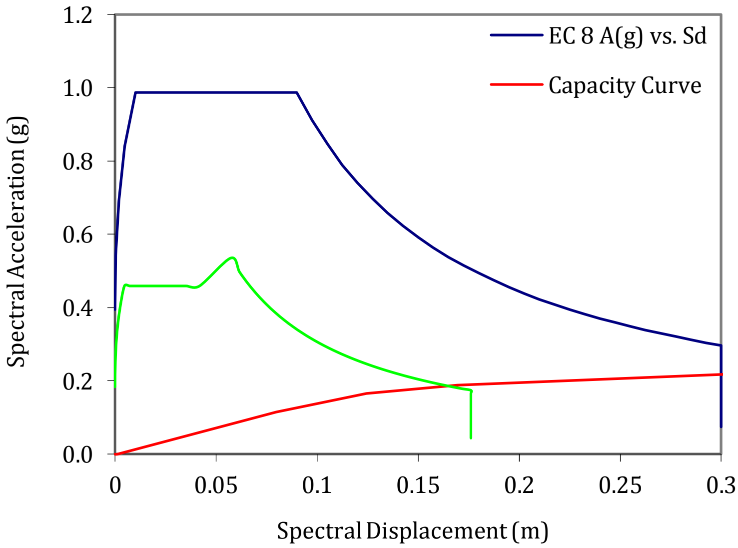

Fig. 15. Performance point of Ref 6 story *x*-direction frame EC 8.

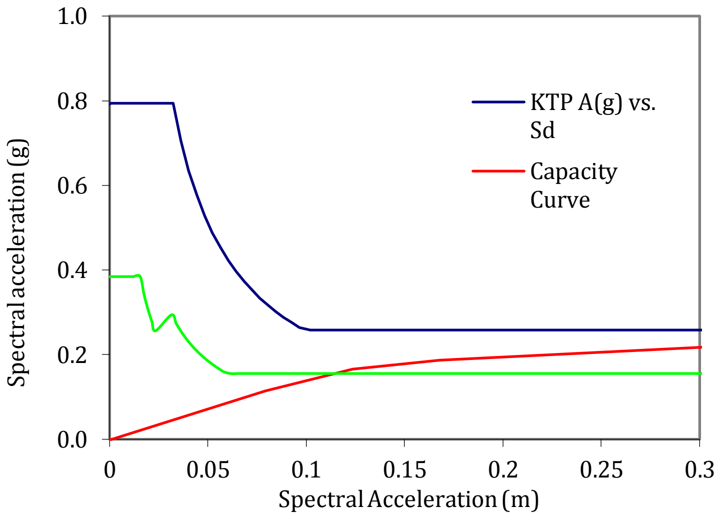

Fig. 16. Performance point of Ref 6 story *x*-direction frame KTP.

In this study masonry infill walls were modelled as diagonal strut elements in accordance with FEMA-356 (2000) guidelines. Regular and irregular frames are considered. Irregular frames are obtained as a result of different structural irregularities, implemented to the regular frames. Structural irregularities taken in consideration for this study are: soft story, heavy overhangs, short column. Effect of structural irregularities and performance of the considered frames are achieved by using capacity curves of the frames and performance point by considering two different response spectrums, from Eurocode 8 (2004) and KTP codes (KTP-N2-89, 1989). Calibration of results is checked by using two different programs ETABS and Seismosoft to analyse the reference frames, 3 and 6 story. The observations and findings of the current study are briefly summarized in the following:

Presence of structural irregularities in reinforced concrete buildings decreases the performance of the frame by lowering its lateral load bearing and displacement capacity. Structural irregularities have almost the same effect in both low and mid-rise frames represented by 3- and 6- story frames.

Soft story with two sided overhangs and short column are the most detrimental irregularities for both low and mid-rise buildings.

Soft story due to lack of masonry infill walls at the ground story is found to be more damaging than soft story because of higher ground story height.

From the achieved performance points it was observed that demands of Eurocode 8 are higher than KTP codes, representing higher values for both spectral acceleration and spectral displacement.

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