EFFECTS OF STRUCTURAL IRREGULARITIES ON LOW AND MID-RISE RC BUILDING RESPONSE

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Approval sheet of the Thesis

This is to certify that we have read this thesis entitled "**Effects of Structural Irregularities on Low and Mid-Rise RC Building Response**" and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

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ABSTRACT

EFFECTS OF STRUCTURAL IRREGULARITIES ON LOW AND MID-RISE RC BUILDING RESPONSE

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Albanian building stock is composed of reinforced concrete and masonry buildings. Most of these buildings are designed according to earlier versions of Albanian Seismic Codes (KTP Codes) 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. Irregularities are weak points in the building which may cause failure of one element or total collapse of the building during an earthquake. Irregularities encountered in Albanian construction practice consist of short column, large and heavy overhangs and soft story. Since Albania is country with moderate seismicity which has been hit many times from earthquakes of different magnitudes establishes the need to study the effect of irregularities. All these irregularities are taken in consideration in this study under lateral loads on low and mid-rise buildings of Albanian construction practice. In order to get the effect of structural irregularities on RC buildings response several numbers of Nonlinear Static (Pushover) analyses are done. The considered frames consist of regular frames, frames with soft story because of higher height and lack of masonry infill walls in ground story or because of the presence of both cases, presence of short column because of semi-infilled bays, overhangs because of presence of balconies, presence of soft story and heavy-overhangs. These irregularities are considered for two types of structures, 3 and 6-story frames representing low and mid-rise buildings respectively. The analyses has been performed by using ETABS and Seismosoft software. 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 EC 8 and KTP codes. The results of the analyses indicate that low and mid-rise structures with soft story- two sided overhangs and short column are more vulnerable during earthquakes. Frames with soft story and two sided overhangs showed 266% and 300% lower strength, 120% and 92% lower stiffness than 3 and 6 story reference frames respectively; frames with short column showed 100% and 92% lower stiffness and 140% and 122% lower strength than 3 and 6 story reference frames respectively.

Keywords: Low and Mid-Rise RC buildings; Structural Irregularities; Nonlinear Static Pushover Analysis;

ABSTRAKT

EFEKTET E PARREGULLSIVE STRUKTURORE NË SJELLJEN E GODINAVE B/A TE ULETA DHE TE MESME

Uruci, Rezarta
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Udhëheqësi: Prof. Assoc. Dr. Huseyin Bilgin

Ndërtesat në Shqipëri përbëhen nga ndërtesa betoni dhe murature. Shumica e këtyre ndërtesave janë të projektuara me versionet e hershme të Kodeve Sizmike Shqiptare (Kodet KTP) dhe disa prej tyre janë ndërtuar pa ndonjë projekt konstruktiv. Duke patur parasysh këto fakte dhe vërejtjet e bëra në industrinë e ndërtimit në Shqipëri, prania e parregullsive strukturore është shumë e zakonshme në këto ndërtesa. Parregullsitë janë pika të dobëta në ndërtesë të cilat mund të shkaktojnë dështimin e një elementi ose kolapsin total të ndërtesës gjatë një tërmeti. Parregullsitë e hasura në praktikën e ndërtimit të Shqipërisë përbëhen nga kollonë e shkurtër, konsola të mëdha dhe të rënda dhe kati i butë. Fakti qe Shqipëria është një vend me sizmicitet të lartë dhe që është goditur shumë herë nga tërmetet e madhësive të ndryshme përcakton nevojën për të studiuar efektin e parregullsive strukturore. Të gjitha këto parregullsi janë marrë në konsideratë per këtë studim nën ngarkesa sizmike në ndërtesa të ulëta dhe të mesme bazuar ne praktikën e ndërtesave në Shqipëri. Në mënyrë që të kuptojmë efektin e parregullsive strukturore në sjelljen e ndërtesave betonarme është perdorur analiza jolineare (Pushover). Ndërtesat e marra në konsideratë konstatojnë në ndërtesa të rregullta, ndërtesa me kate të butë për shkak të mungeses së mureve të tullës në katin zero ose për shkak të lartësisë më të madhe të katit zero në krahasim me katet e tjera ose për shkak te prezencës së katit të butë për të dyja arsyet në të njëjtën kohë, prezenca e kollonës së shkurtër për shkak të hapësirave gjysëm të mbushura me muraturë, prezenca e konsolave të rënda dhe të mëdha të shkaktura nga ballkonet, prania e konsolave dhe katit të butë në të njëjtën kohë. Këto parregullsi janë studiuar në të dyja llojet e ndërtesave, 3 dhe 6-katëshe të cilat përfaqësojne ndërtesat e ulëta dhe të mesme respektivisht. Analizat janë bërë me dy programe të ndryshme, ETABS dhe Seismosoft. Efekti i parregullsive dhe sjellja e ndërtesave është bërë duke marrë në konsideratë kurbën e kapaciteteve dhe pikën e performancës nga analiza e dy spektrave të ndryshëm reagimi, EUROCODE 8 dhe KTP. Rezultatet e analizave tregojnë se ndërtesat e ulëta dhe të mesme ku prania e parregullsive strukturore si variet e rënda dhe kati i butë, kollona e shkurter janë te pranishme, janë më të prekshme gjate tërmeteve. Ndërtesat ku kati i butë dhe konsolat janë të pranishme kanë treguar fortësi 266 % dhe 300 %, ngurtësi 120 % dhe 92 % më të ulët se ndërtesat bazë me 3 dhe 6 kate respektivisht; ndërtesat ku kollona e shkurtër është e pranishme kanë treguar fortësi 100 % dhe 92%, ngurtësi 140 % dhe 122 % më të ulët se ndërtesat bazë me 3 dhe 6 kate respektivisht.

Fjalët kyçe: Ndërtesa betonarme të ulëta dhe të mesme; Parregullsi Strukturore;

Analizë Jolineare;

Dedicated to my parents and husband

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LIST OF ABBREVIATIONS

S_i, S_{i+1},S_{i+2}	Stiffness of ith, i +1th and i + 2th storey
SB_i	Setback irregularity limits
l_s	Radius of gyration
r_x,r_y	Torsional radius in x and y direction
R_i	Re- entrant corner projection limit
$e_{ox}e_{oy}$	Eccentricity in x and y direction
Ref	Reference Building (without any irregularity)
TSO	Two sided overhang building
SSH	Soft story due to 4.5 m ground story height
SSW	Soft story due to absence of masonry infill walls at ground story
SS-H-W	Soft story due to both height and infill effect
SS-H-W-TSO	Soft story due to both height, infill and two sided overhang
SHC	Short column due to sem-infilled bays at ground story
h_{inf}	Height of infill panel
E_{fe}	Expected modulus of elasticity of frame materials
E_{me}	Expected modulus of elasticity of infill materials
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
A_{ni}	Area of net mortared/grouted section across infill panel
f_{vie}	Expected shear strength of masonry infill

CHAPTER 1

INTRODUCTION

1.1 General

The inadequate performance and the high level of structural damages in the building stock of our country during the last earthquakes [Ormeni *et. al.,* 2013], establishes the need to determine and asses the damages of the structures. Reinforced concrete buildings which present the low and mid-rise building stock of Albanian construction practice are composed of three and six story frame structures respectively in most of the cases.

During the last earthquakes worldwide most of the building collapse because of the presence of structural irregularities in frame construction [Varadharajan, 2014]. The widespread presence of structural irregularities in reinforced concrete buildings highlights the need to extend the level of knowledge about their effect on RC frame structures.

Structural irregularities in reinforced concrete structures can be broadly classified as plan and vertical irregularities. A reinforced concrete structure can be classified as vertically irregular if it contains non-uniform distribution of mass, stiffness and strength along the building height; Structure is horizontally irregular when it has asymmetrical plan shapes, re-entrant corners and non-uniform distribution of stiffness, mass, and strength along the plan also [Varadharajan, 2014].

1.2 Problem Statement

Structural irregularities have a significant effect on reinforced concrete buildings during an earthquake. In order to prevent a collapse mechanism caused because of structural irregularities, seismic demand must be determined accurately.

Aim of this study is to carry out the nonlinear static analysis of three and six 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 conclude structural irregularities effect on RC frame structures.

This study will assist in identification of structural irregularities effect on RC frame structures response.

1.3 Objective of the study

The main objectives of this study are:

- To determine structural irregularities that are created in reinforced concrete buildings.
- To determine the types of irregularities existing in Albanian construction practice.
- Determination of nonlinear behaviour of building structures with short column, soft story and heavy overhangs by utilizing nonlinear static pushover analysis.
- To determine and compare the effect of each irregularity (soft story, short column, overhangs) on RC frame structures under the seismic loads effects.

1.4 Outline of the thesis

Chapter 1 presents an introduction to the topic, defines the problem of the thesis, states the main objectives and a general summarize of the thesis outline.

Chapter 2 presents structural irregularities that are studied in this thesis, soft story, heavy overhangs and short column according to EUROCODE 8 [Eurocode 8, 2004]. Their effect is developed in two types of structures low and mid-rise buildings, representing the building stock in Albania construction practice, under the seismic actions. A literature review of the previous studies published in the field of the effect of structural irregularities in reinforced concrete frame structures is introduced in the second part of the chapter.

Chapter 3 presents the modelling of the two different types of buildings low and mid-rise, represented by three and six story respectively, which are studied in this thesis, in two different software's ETABS [Etabs, 2013] and Seismosoft [Seismosoft, 2014]. The first part of this chapter covers a description the case study buildings, structural plans and reinforcement details. Firstly the RC frame structures, three and six story, are modelled in their regular form and then the structural irregularities: short column, soft story, heavy overhangs are imposed to them. Soft story irregularity is created in two ways: by removing the masonry infill walls and by increasing the height of the ground story level. Short column is created by semi-infilled bays and heavy overhangs by large balconies present in the structure. So all these cases generated from different irregularities are analysed in both software's.

Chapter 4 includes the analysis of the case study buildings. In the first part the general information about the type of analysis used in this thesis is done. Then the analysis results are shown for each irregularity considered in this study for the two different types of building. Analysis of different irregularities achieved with different software's are compared in order to show which irregularity has the highest effect in a reinforced concrete frame structure under the seismic actions. Results gained from both software’s, ETABS and Seismosoft, are also compared.

Chapter 5 highlights the most important results achieved in the study. Firstly summarize of the main objectives reached in this thesis is done and then the main results gained from analysis will be summarized in order to conclude the importance of them.

CHAPTER 2

LITERATURE REVIEW

2.1 General

The inadequate performance and the huge number of collapsed buildings during past earthquake because of diverse structural irregularities determines the idea to analyse the buildings with dissimilar irregularities in order to understand the effect of irregularities in RC buildings under the seismic effect. Different researchers and academics [Varadharajan, 2004; Altuntop, 2007], have studied altered vertical and horizontal irregularities with different methods of analysis such: nonlinear static pushover analysis, dynamic analysis, and time history analysis, etc. and realized which type of irregularities are more risky during an earthquake and what should be taken in consideration from the designers during the design process.

2.2 Irregularities in Reinforced Concrete Buildings

Structural irregularities in reinforced concrete buildings may be classified as vertical and horizontal irregularities. Sudden changes in mass, stiffness or strength in either direction of the buildings, horizontal or vertical, can result in different distribution of lateral loads and deformations from the ones expected for uniform structures. Even though the effects of structural irregularities may be taken in consideration in design process the uncertainties associated with the effects of irregularities are such that they are better avoided if possible. Classifications and limits of structural irregularities in RC buildings according to EUROCODE 8 are represented in *Figure 1* and *Table 1* below:

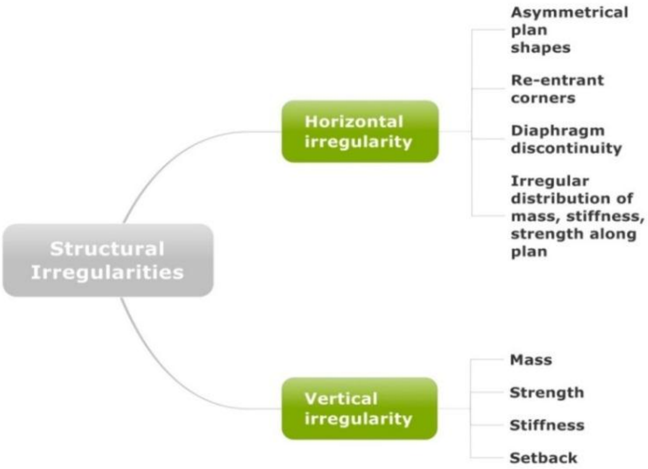


Figure 1. Classification of Structural Irregularities [Eurocode 8, 2004]

Irregularities that are studied in this thesis consist of: soft story, overhangs and short column.

2.2.1 Soft Story

Soft story irregularity is created because of the lower stiffness of one story compared to the other stories. A soft story irregularity exist if the stiffness of one story is less than 70% of the story above or less than 80% of the average of the three stories above [FEMA 368, 2001]. Soft story irregularity may be created as a result of:

- Non uniform distribution of infill walls
- High level of one floor compared to the other floors

Most of the buildings have parking and commercial areas in the first stories having more open areas with less infill walls and higher floor level creating a soft story in the ground floor of the building.

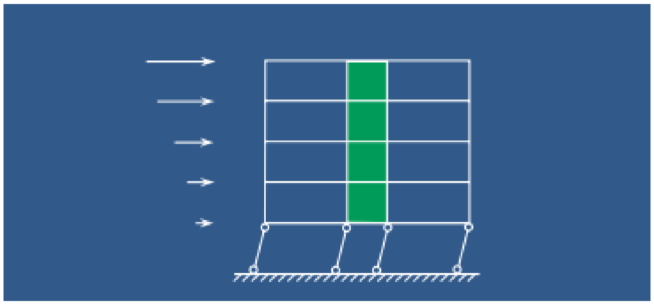


Figure 2. Soft Story Ground Floor [Bachmann, 2002]

Table 1. Irregularity Limits according to EUROCODE 8 [Eurocode 8, 2004]

Type of Irregularity	**EUROCODE 8 2004**
Horizontal	
Re-entrant Corners	$R_i \leq 5\%$
Slenderness	$\lambda = L_{max}/L_{min} < 4$
In-plan stiffness	In-plan stiffness of the floors shall be sufficiently large in comparison with the lateral stiffness of the vertical structural elements C, H, I, and X plan shapes should be carefully examined
Torsional irregularity	$0.3\ r_x \geq e_{ox}$
	$0.3\ r_y \geq e_{oy}$
	r_x and $r_y \geq l_s$
Diaphragm Discontinuity	$r_x^2 > l_s^2 + e_{ox}^2$
	$r_y^2 > l_s^2 + e_{oy}^2$
Vertical	
Mass	Should not reduce abruptly
Stiffness	$S_i < 0.7S_i+1$ or $S_i < 0.8\ (S_{i+1} + S_{i+2} + S_{i+3})$
Setback irregularity	$SB_i < 0.3$. plan dimension of the first storey

where:

- S_i, S_{i+1},S_{i+2}- Stiffness of i^{th}, i +1^{th} and i + 2^{th} storey,
- SB_i -Setback irregularity limits,
- l_s- Radius of gyration,
- r_x,r_y - Torsional radius in x and y direction,
- R_i - Re-entrant corner projection limit,
- e_{ox} e_{oy}- eccentricity in x and y direction

Soft story in ground floor makes that the columns of that floor to be damaged because of cyclic displacements between the upper part of the building and the moving soil. From the plastic deformations (hinges) that are created at the top and bottom of the columns, *Figure 2,* a story mechanism may be formed which may lead to large deformation at column ends from which in most of the cases an inevitable collapse of the building happens [Bachmann, 2002]. In this type of stories the lateral load resisting system is lower than the other ones. Because of this irregularity during an earthquake the inter story drifts between adjacent stories and lateral forces are not well distributed along the height of the building. So the lateral forces are concreted in the stories having larger displacement, *Figure 3.*

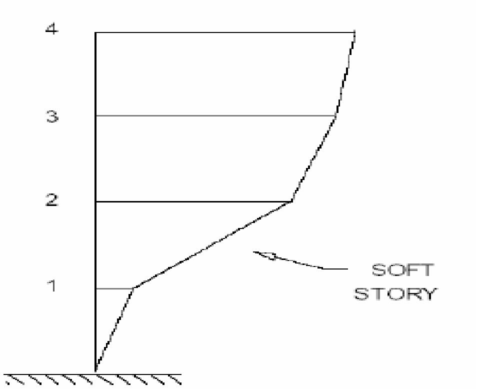


Figure 3. Soft story behavior of a building structure under lateral loading [Altuntop, 2007]

If during design phase the local ductility demands are not considered for that soft story and the inter story drifts are not limited a failure mechanism of the story may be created which may lead to the total collapse of the building. In the *Figures 4-5* below is shown the failure mechanism created because of the soft story irregularity, under gravity and earthquake loads:

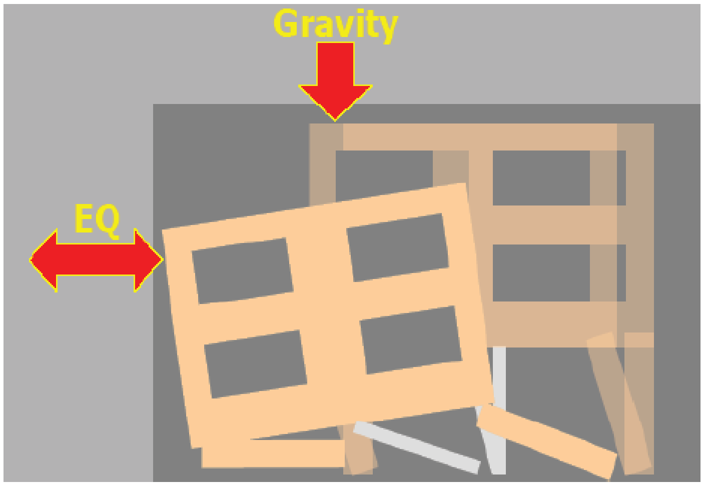


Figure 4. Failure mechanism created because of the soft story [Altuntop, 2007]

Figure 5. Sway mechanisms are often inevitable with soft story ground floors [Bachmann, 2002]

But soft story is not created in ground floors only; it may also be created in cases when an upper story is soft in comparison with other ones or if horizontal resistance is strongly reduced above a certain floor, *Figures 6-7*.

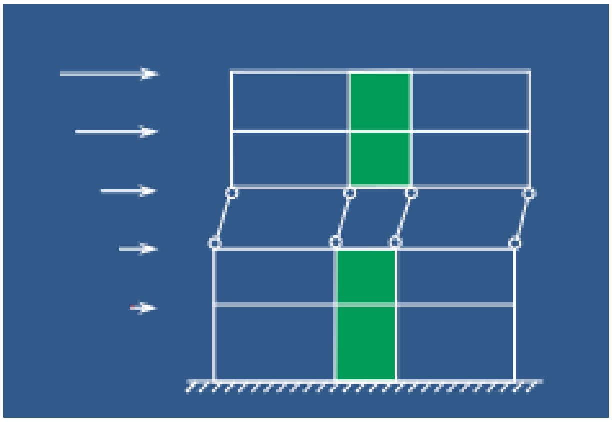


Figure 6. Soft Story in Upper Floors [Bachmann, 2002]

Impacts may also be creation of a failure mechanism in the building, *Figure 7*.

Figure 7. Failure an upper storey [Bachmann, 2002]

EUROCODE 8 states that the soft story mechanism must be prevented in order for the local ductility demands in the soft story columns not to be exceeded. According to the code the below provision must be for main columns and beams of the building:

$$\sum MRc \geq 1.3 \sum MRb \tag{3.1}$$

- where $\sum MRc$ is the sum of designed moments of the columns that are connected to the considered joint and $\sum MRb$ is the sum of the designed moments of the beams connected to the columns. In the code is also specified that while calculating the sum of designed moments for the columns the minimum column moment values in the variety of column axial forces created by seismic design technique should be used.

In the *Figures 8-10* below are shown some cases of building failures because of soft story irregularity, in past earthquakes:

Figure 8. Building failures because of soft stories in Turkey during Bingol earthquake 2003 [Varadharajan, 2014]

a) b) c) d)

Figure 9. Building failures because of: a) Sway mechanism with soft story ground floors, b) Soft first story collapsed, c) Collapse of soft middle story, d) Vertical Split between two blocks [Semnani, 2014]

a) *b)*

Figure 10. Soft-story collapse of infill buildings from: a) the 2009 Padang, Indonesia earthquake, b) 2009 L'Aquila, Italy earthquake [Semnani, 2014]

2.2.2 Overhangs

Heavy overhangs are a type of irregularity which makes that the mass centre and centre of rigidity not to be in the same location creating so the torsional effect in the building.

Heavy overhangs like balconies shift the mass centre upwards increasing the lateral forces and overturning moments during earthquakes. Buildings having balconies with large overhanging cantilever spans enclosed with heavy concrete parapets sustained heavier damages during the recent earthquakes in the world compared to regular buildings in elevation. Since this building feature can easily be observed during a walk-down survey, it is included in the parameter set.

In the *Figure 11* below are shown some cases of building failures during earthquakes because of overhangs.

Figure 11. Failure due to overhangs [Varadharajan, 2014]

2.2.3 Short Column

During last earthquakes it has been observed that in reinforced concrete buildings in which columns with different height were present, columns that were shorter were more damaged. During an earthquake short column and tall column will move both horizontally even though the short column is stiffer and will attract more earthquake forces, the larger the stiffness the larger is the force required to deform it. But if the shorter column is not adequately designed to resist such forces it will suffer significant damage during an earthquake. This behaviour is known as short column effect *Figure 12*, [Vahidi, 2009].

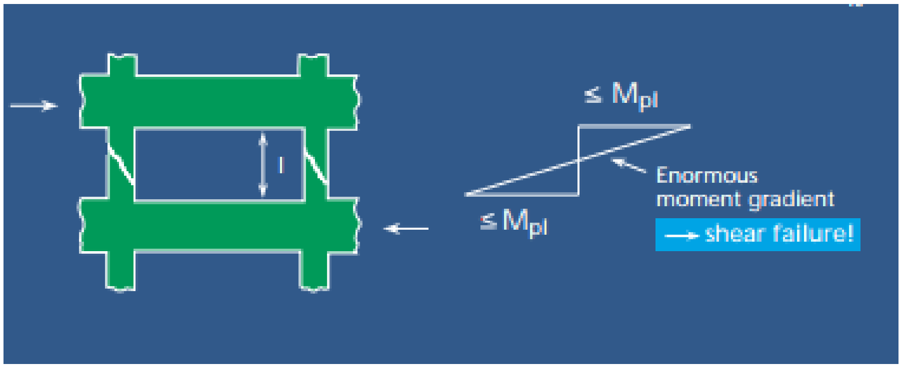


Figure 12. Short column effect [Bachmann, 2002]

Short column effect may be created from different situations in a reinforced concrete building:

- When the building is rested in a sloped ground and during the earthquake column with different height in a story will move horizontally by the same amount. But short columns will attract more forces and will get damaged more than tall columns, *Figure 13a*.
- Short column effect may be also created in cases when a columns support mezzanine floors or loft slabs that are added between two floors, *Figure 13b*.

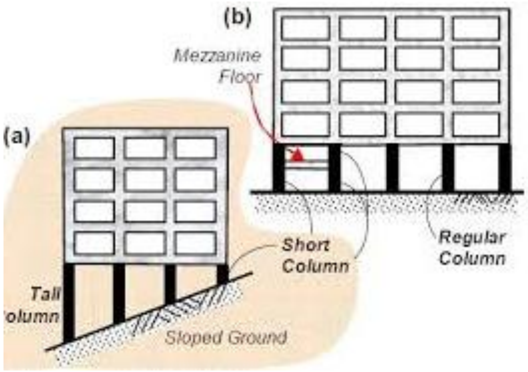


Figure 13. Short column effect: a) building is rested in a sloped ground b) mezzanine floors [Vahidi, 2009]

- Another situation is when a masonry or reinforced concrete partial wall is built to fit the window over the remaining height. In this case the two adjacent columns will behave as short columns while the other columns in the story are of regular height. During the earthquake normal columns will get deformed along all their height but the short columns are restricted by the walls in the lower portion so all the deformation will be carried out by the shorter height where the window is located, *Figure 14.*
- Short column effect may be also created in cases when difference in level exists in a reinforced concrete building, *Figure 15*.

Source: Wakabayashi,M., Design of Earthquake-Resistant Buildings, McGraw Hill Book Company, New York,

Figure 14. Short column effect created by band window [Vahidi, 2009]

Figure 15. Short colun effect because of difference in level in a reinforced concrete building, [Vahidi, 2009]

Behaviour of columns in a RC building during an earthquake is very important since failure of them may lead to the total collapse of the structure. Columns which have unequal length may lead to different load distribution and so failure will occur.

In the *Figures 16-17* below are shown some cases of building failures because of short column irregularity, in past earthquakes

Figure 16. Building failures because of short column effect, 2009 Padang, Indonesia earthquake [Semnani, 2014]

Figure 17. Building failure because of short column effect, during the 1988 Udaypur, Nepal earthquake [Semnani, 2014]

2.2.4 Transverse Reinforcement

Transverse reinforcement in reinforced concrete buildings are specified to the codes for beams and columns to serve the functions below:

- To resist shear forces and avoid shear failure
- To prevent buckling of longitudinal reinforcing bars
- To confine concrete core in order to provide sufficient ductility

In order for the reinforced concrete building to have a ductile behaviour its member's should yield in flexure and shear failure should be avoided. Shear failure is brittle, especially in columns, which may cause the loss of its lateral strength and axial load carrying capacity. So in order to avoid failure the flexural yielding regions should be identified and designed for code required moment strengths and then to calculate the design shears assuming that the yielding regions to develop probable moment strength [Moehle *et. al*., 2008].

In the *Figure 18* below are shown two cases in which the confinement of columns is almost missing and the failure of the column has led to the total story collapse.

Figure 18. Failure of columns with inadequate ties, almost no confinement [Moehle *et. al*., 2008]

2.3 Past Studies

Dolsek and Fajfar on their paper studied the effect of soft story in uniformly infilled reinforced concrete frames. From the pushover analysis vertical load distribution was triangular for the bare frame and uniform for the infilled frame. Beam hinge mechanism was formed in the third story columns in case of the bare frame. For the infill frame stiffness and strength was grater but with the increase of deformation the infill panels failed. In the first story of the infill frame a plastic mechanism was formed in large deformations. Presence of the uniform distribution of infill decreases the top displacement and story drift compared to the structure with irregular distribution of the infill, soft story created. From the test it was concluded that the infill in the first and second story totally collapses, in the upper stories it remained almost intact, creating so the soft story mechanism in the ground stories. So the soft story effect is created also in structures with uniform distribution of infill walls if: ground motion is stronger than the design strength, global ductility and ductility of the structural elements are small, and the infill walls are relatively weak and brittle [Dolsek *et. al*., 2000].

Arturo Tena- Colunga on his paper studied the seismic response of two irregular moment-resisting frame buildings with setback and slender irregularity by using dynamic analysis [Tena- Colunga, 2004]. From the nonlinear dynamic analysis it was realized that structures with one bay frame designed with drift ratio very close to the limit are more vulnerable to earthquakes because they create weak story effect which may result in total building collapse. So increasing the redundancy of the frame structure improves also the behavior of the building under the seismic effect. But from the analysis of the buildings with setback irregularity there was no high stress concentration or high yielding demand changed, but this result could be different in other situations so it needs more checks in order to conclude this result properly [Tena- Colunga, 2004].

Altuntop studied and analysed the buildings with soft story irregularity since it has been a serious problem during the past earthquakes in Turkey [Altuntop, 2007]. In order to realize the effect of this irregularity during an earthquake the nonlinear static pushover and time history analysis were used. Structures with different number of stories have been analysed. All the models were designed according to the Turkish Code-2007 in order to meet the high level of ductility demands. From the analysis it was seen that for structures with soft story irregularity the ultimate deformation capacity is considerably lower compared to the ones without any irregularity. Ultimate deformation capacity is increased if the number of spans is increased and the height of the first story is decreased. Structures with fewer stories, where the soft story irregularity is present are more vulnerable to earthquakes. As a conclusion can be said that it was observed that all the buildings, especially the ones with soft stories, which have only the frames as lateral load resisting system are more vulnerable during the earthquake.

Apostolska *et. al* on their paper studied the influence of the infill walls on seismic performance of RC frame buildings [Apostolska *et. al*., 2010]. Seismic effect on the structure has been defined according to the lateral force method. In order to get the effect of the infill under the seismic effect the nonlinear static pushover analysis was carried out and the most probable mechanism of damage distribution was elaborated. Masonry infill was modelled as equivalent diagonal strut in which nonlinearity was concentrated and with increase of the level of input horizontal plastic hinges entry even in the collapse prevention zone. As a conclusion the effect of the infill in the structure is significant increasing initial stiffness, strength and decreasing fundamental period which results in large elastic stiffness and small deformability, immediately after appearance of first cracks nonlinearity increase rapidly up to failure of infill [Apostolska e*t. al*., 2010].

Sattar and Liel on their paper studied the effect of infill walls on RC buildings under the seismic effect [Sattar *et. al*., 2010]. Their case study structures were designed and detailed according to 1975 California Code. Nonlinear static pushover analysis was carried out on 2 dimensional 3-bay frames which were implemented in Open Sees software. Infill walls were represented by equivalent struts in compression only; each of the diagonal struts represented the initial stiffness, peak strength, and post peak behaviour of the masonry infill walls sufficient to predict the failure of the wall. From the results of pushover analysis it was seen an increase of initial stiffness, strength and energy dissipation of the in filled frames compared to the bare frame. From the dynamic analysis results was concluded that bare frames are more vulnerable during earthquakes compared to infill frames. These results have been observed for the two types of structures used, 4 and 8 stories [Sattar *et. al*., 2010].

Sonmez studied the effect of infill walls on the behaviour of reinforced concrete frames under the earthquake demands [Sonmez, 2013]. In order to get the effects of the infill walls during earthquake loading the nonlinear static pushover and dynamic analysis was performed. From the analysis the base shear capacity was increased in case of infill walls presence. Since the stiffness of the infill walls was higher during the induced ground motions they performed better, decrease in period decreased the displacement demand for these frames. From the dynamic analysis in the first story of the first infill frame a soft story mechanism was caused since all the drifts were concentrated in the lower stories. Since the distribution of stiffness in second infill frame was in a uniform way the drifts showed to be lower and well controlled. In case of pushover analysis the well distribution of stiffness in elevation revealed a better inter-story drift along the height of the building.

Varadhajaran on his Phd thesis studied irregular RC buildings under the seismic effect since most of the buildings collapse or suffers severe damage during earthquakes because of sudden changes in mass, stiffness and strength along vertical or horizontal plane [Varadharajan, 2014]. In this study analysis of different models with different location of irregularities were performed and then compared with the reference structure which was without any irregularity. From the nonlinear static pushover analysis and time history analysis, an irregularity index which overcomes the codes was evaluated. From the code and other research works was clearly understood that the location of irregularity was always neglected in quantification of irregularity. But these cannot be neglected because it has an important role in the presentation of irregularity. Irregularity index proposed in this thesis captures both of them: irregularity and its location being more efficient. Fundamental time period was classified as a critical parameter in seismic design and seismic vulnerability assessment of buildings [Varadharajan, 2014]. Two different period-height of the building relation are presented from the Code: Eigen Value Analysis and Inelastic Dynamic analysis. But these methods have different inherent disadvantages because two buildings with similar heights will lead to same period. So it was convenient to represent the period related with the index relation found in the previous section which was based on dynamic response. Then by having the dynamic analysis of the irregular buildings and a correction factor for the period a relation between the index and period was found, *Equation 2.3.1*:

$$T=\delta_{11}(0.075H^{0.75}) \tag{2.3.1}$$

Fundamental time period increases with the increase in height, strength, stiffness of the building and it is not depending in the change of mass irregularity. But an increase of irregularity presence in the structure increases marginally the fundamental time period. Deformation demands are the stresses which show the behaviour of the structure. In this section elastic and inelastic deformation demands were calculated and then the values were compared with the reality. After finding the inelastic and elastic values the mean ratio from inelastic to elastic deformation demands were expressed in terms of ʎ$_{irr.}$ Then the equations proposed were compared to the dynamic analysis according to EUROCODE 8. The comparison resulted in close agreement between the equations with high correlation factor which means that the methodology discovered can be used instead of EUROCODE 8 with the help of ʎirr (mean relation factor correlating elastic and inelastic seismic response) factor. As a conclusion can be said that effect of irregularity in the building cannot be neglected since it has a high effect to an RC frame structure during an earthquake.

Table 2. Literature Review Summary

No	Title	Author	Year	Objectives	Irregularity	Type of analysis	Main effect
1	Soft story effect in uniformly in filled reinforced concrete frames	Matjaz Dolsek, Peter Fajfar	2000	To determine the effect of soft story irregularity in uniformly and non-uniformly msonry infilled frames	Soft Story	Nonlinear Static Pushover Analysis	- Presence of the uniform distribution of infill decreases the top displacement & story drift compared to the structure with irregular distribution of the infill, soft story created. - Soft story effect is created also in structures with uniform distribution of infill walls if: ground motion is stronger than the design strength, global ductility and ductility of the structural elements are small, the infill walls are relatively weak and brittle
2	Evaluation of the seismic response of slender, setback, RC moment-resisting frames buildings designed according to the seismic guidelines of a modern building code	Arturo Tena-Colunga	2004	To determine the effect of irregularities(setback, slender) on RC buildings	Slender, Setback	Dynamic Analysis	- Structures with one bay frame designed with drift ratio very close to the limit are more vulnerable to earthquakes because they create weak story effect - In buildings with setback irregularity there was no high stress concentration or high yielding demand different
3	Analysis of building structures with soft story irregularity	Mehmet Alper Altuntop	2007	- Determination of the nonlinear behaviour of building structures with soft stories by utilizing nonlinear static pushover and time-history analyses for various deformation levels - Evaluation of the accuracy and efficiency of the nonlinear static pushover analysis by considering various lateral load patterns - Evaluation of the provisions that are defined in various earthquake codes for soft story irregularity	Soft Story	Nonlinear Static Pushover and Time History Analysis	-Lowers the ultimate deformation capacityof the RC buildings -Ultimate deformation capacity is increased if the number of spans is increased and the height of the first story is decreased

4	Effect of infill walls stiffness variations on the behavior of reinforced concrete frames under earthquake demands.	Egemen Sonmez	2013	To investigate the effects of changes in the stiffness of the infill walls on the seismic response of reinforced concrete frames.	Infill Walls	Nonlinear Static Pushover and Dynamic Analysis	- Existence of infill walls increased the stiffness and strength of the frame substantially - Base shear capacity was increased in case of infill walls presence - Since the stiffness of the infill walls was higher during the induced ground motions, decrease in period decreased the displacement demand
5	Influence of Masonry Infill on Seismic Performance of RC Frame buildings	R.Apostolska, G.Necevska-Cvetanovska, J.Cvetanovska, E.Gjorgjievska	2010	To get the influence of masonry infill walls on RC buildings by using nonlinear static pushover analysis	Infill Walls	Nonlinear Static Pushover Analysis	-Increases initial stiffness, strength and decreases fundamental period which results in large elastic stiffness and small deformability
6	Seismic performance of reinforced concrete frame structures with and without masonry infill walls	Siamak Sattar, Abbie B.Liel	2010	To determine the effect of masonry infill walls on RC frame structures	Infill Walls	Nonlinear Static Pushover and Dynamic Analysis	Increase the initial stiffness, strength and energy dissipation of the RC frame structure
7	Study of irregular RC buildings under the seismic effect	S.Varadhajaran	2014	To determine the effect of different structural irregularities on RC structures under the seismic effect	Irregular Rc buildings	Nonlinear Static Pushover and Time History Analysis	-Irregularity Index: Position of irregularity in the frame is important in defining the effect of it under the seismic effect - Fundamental time period increases with the increase in height, strength, stiffness of the building and it is not depending in the change of mass irregularity. But an increase of irregularity presence in the structure increases marginally the fundamental time period. - ʎirr (mean relation factor correlating elastic and inelastic seismic response) factor can be used to determine the deformation demands in the structure - Effect of irregularity in the building cannot be neglected since it has a high effect to an RC frame structure during an earthquake

CHAPTER 3

ANALYTICAL MODELLING OF THE CASE STUDY BUILDINGS

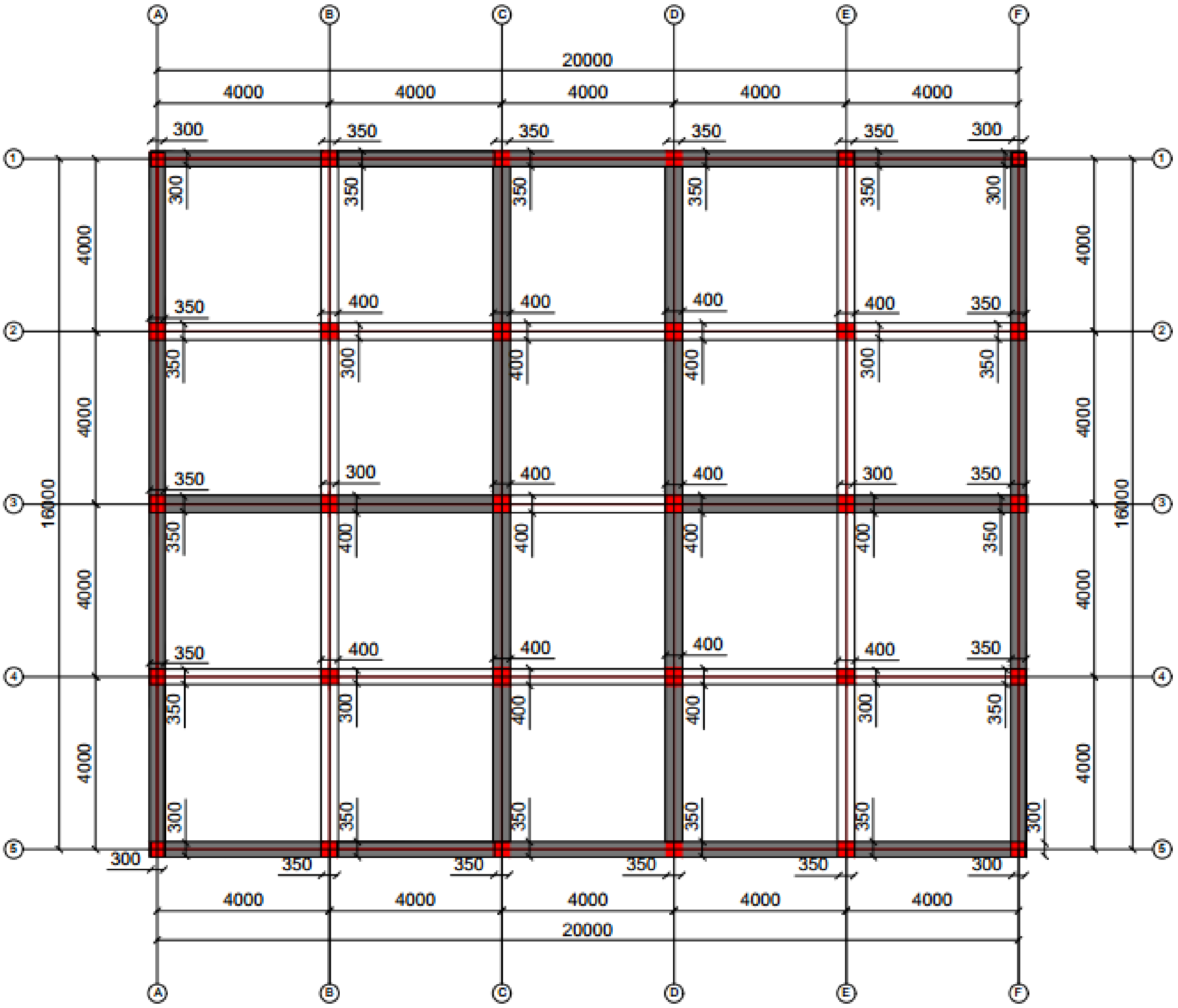
3.1. General

This study aims to assess the seismic performance of the major portion of building stock in Albanian construction represented by low and mid-rise buildings, three and six story respectively (Anex A). Seismic performance of the selected buildings will be done by considering reinforced concrete elements and as well as masonry infill walls.

Selected buildings are modified to have the structural deficiencies observed in damaged buildings during the last earthquakes in Albania and worldwide. Structural irregularities consist in: soft story, short columns, large and heavy overhangs and soft story with heavy overhangs.

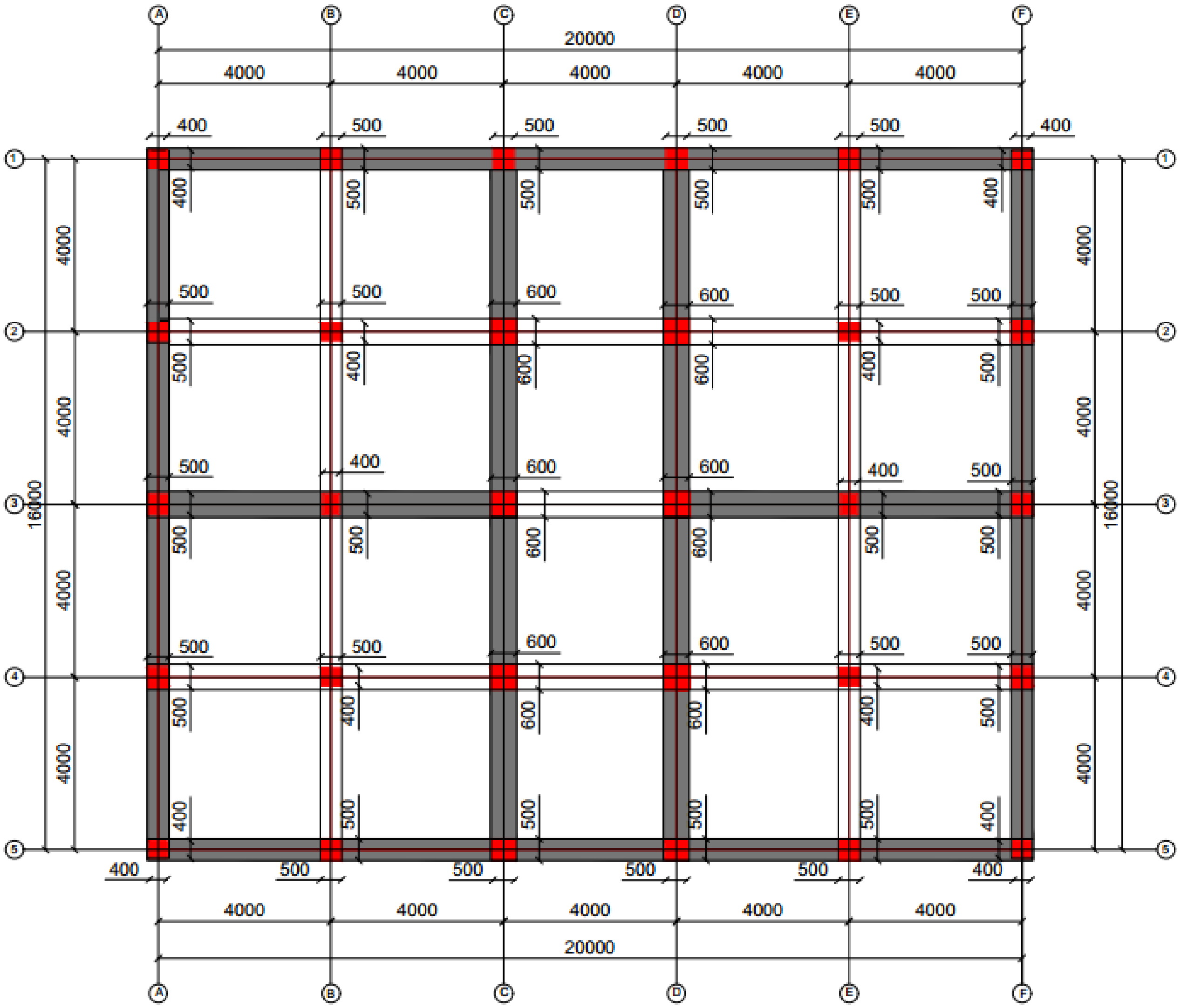
3.2. Description of the Case Study Buildings

In order to achieve the aim of this study two reinforced concrete buildings, 3- and 6-story, are selected to represent the reference low and mid-rise buildings. The certain buildings are typical RC frame buildings with no shear walls. The selected 3 and 6 story frame buildings are regular 20m by 16 m in plan. Both have 5 bays by 4m along X direction and 4 bays by 4m along Y direction *(Figure 19-21).* Typical floor height for both frames is 3m. The location of masonry infill walls in plan is shown by the hatch of beams for the both structures respectively *(Figure 19-21).* Both selected buildings have the same plan view as shown in *Figures 19-21* below:



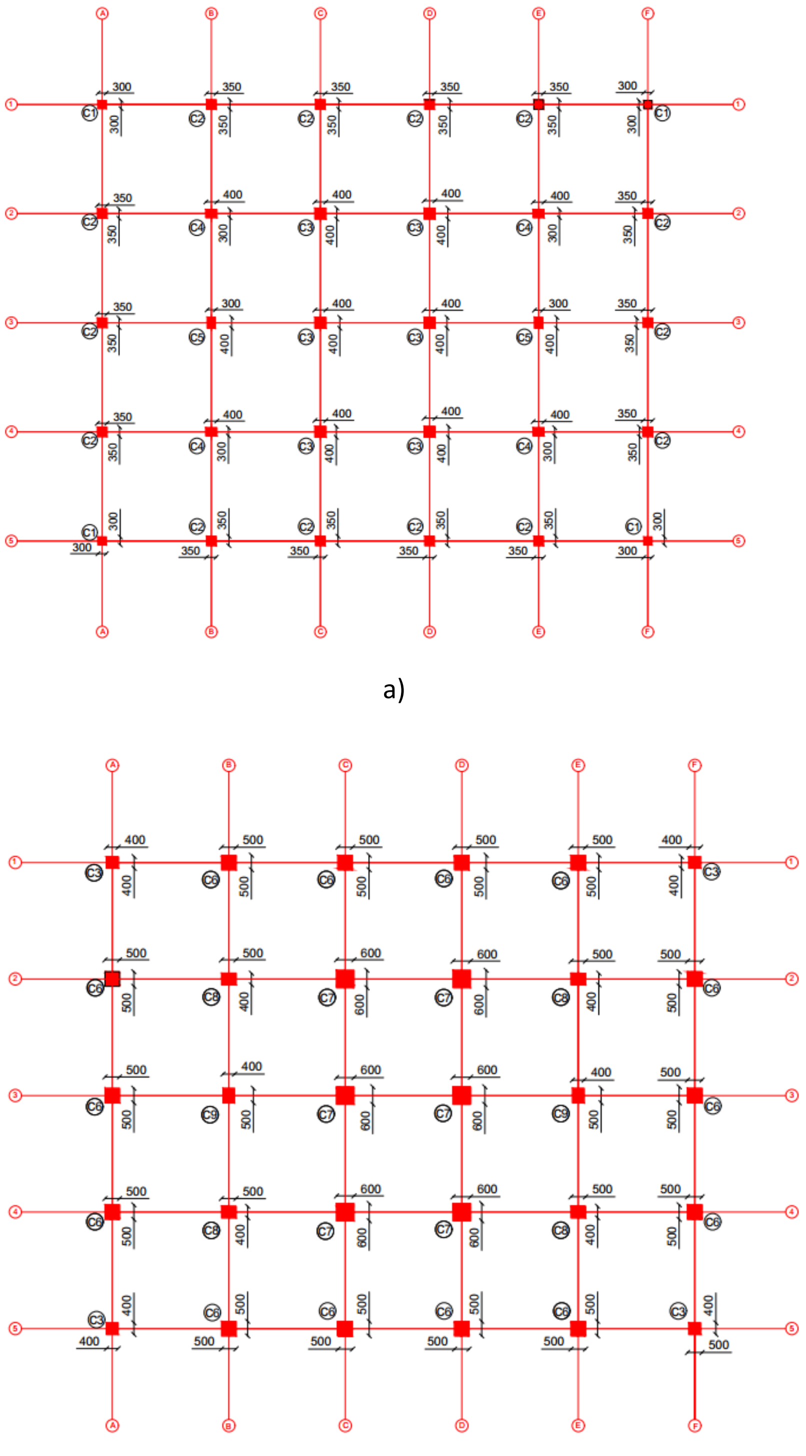
▬ Masonry Infill walls

Figure 19. Structural plan view of the 3-story frame, (units in mm)



▬ Masonry Infill walls

Figure 20. Structural plan view of the 6-story frame, (units in mm)

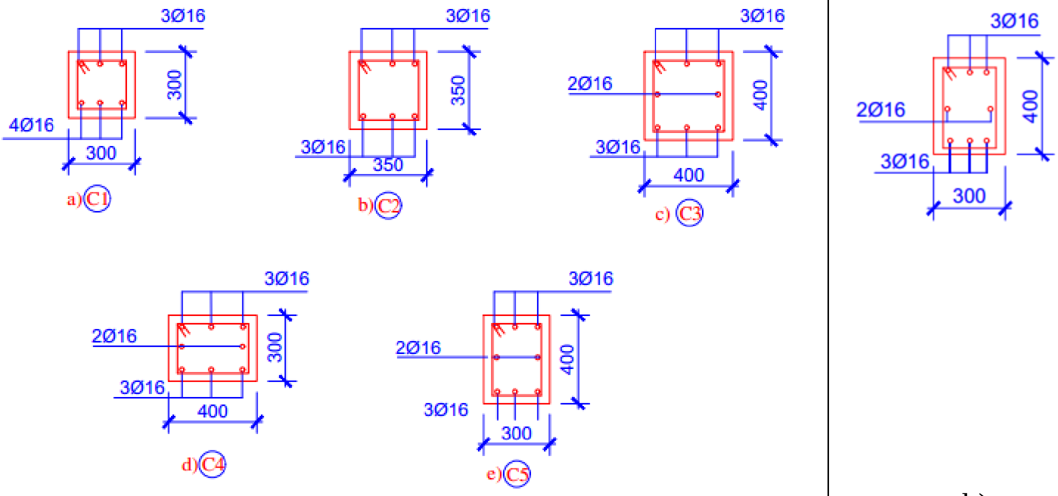


a)

b)

Figure 21. Column plan view of the: a) 3-story frame, b) 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 (Anex A). 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 22a*. Beams have all the same section for the 3- story frame which consists of 300mm x 400 mm *Figure 22b*. Typical column and beam sections and their reinforcement details for the 3 story frame are given in *Figure 22a-b* below:

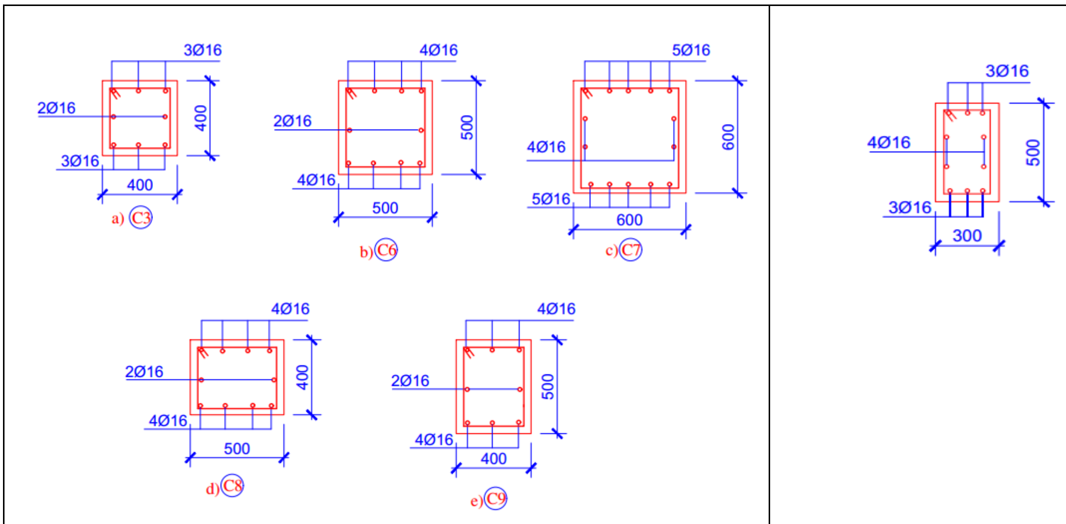


a)

b)

Figure 22. a) Column and, b) Beam reinforcement details for 3-story frame (units in mm)

The 6- story frame consists of 400mm x 400mm and 500mm x 500mm outside columns, identified as C3 and C6 respectively, 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 23a*. All the beams of the 6-story frame have the same section which consists of 300mm x 500 mm *Figure 23b*. Typical column and beam sections and their reinforcement details for the 6 story frame are given in *Figure 23a-b* below:



a) b)

Figure 23. a) Column and b) Beam reinforcement details for 6-story frame (units in mm)

During the analysis no hooks are taken in consideration as transverse reinforcement but bordering shear links. The transverse reinforcement is represented 100mm by spacing in order to reflect the ductile detailing. The selected buildings do not have any vertical or horizontal irregularity (short columns, soft story, overhangs, etc.) 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 are modified to have one or more of the above-mentioned structural deficiencies: soft story, short column, overhangs observed in last earthquakes.

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 motives. 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.

In *Figures 24a-b* below is shown the location of masonry infill walls in elevation and the formation of soft story irregularity because of taking out the masonry infill walls in the ground story:

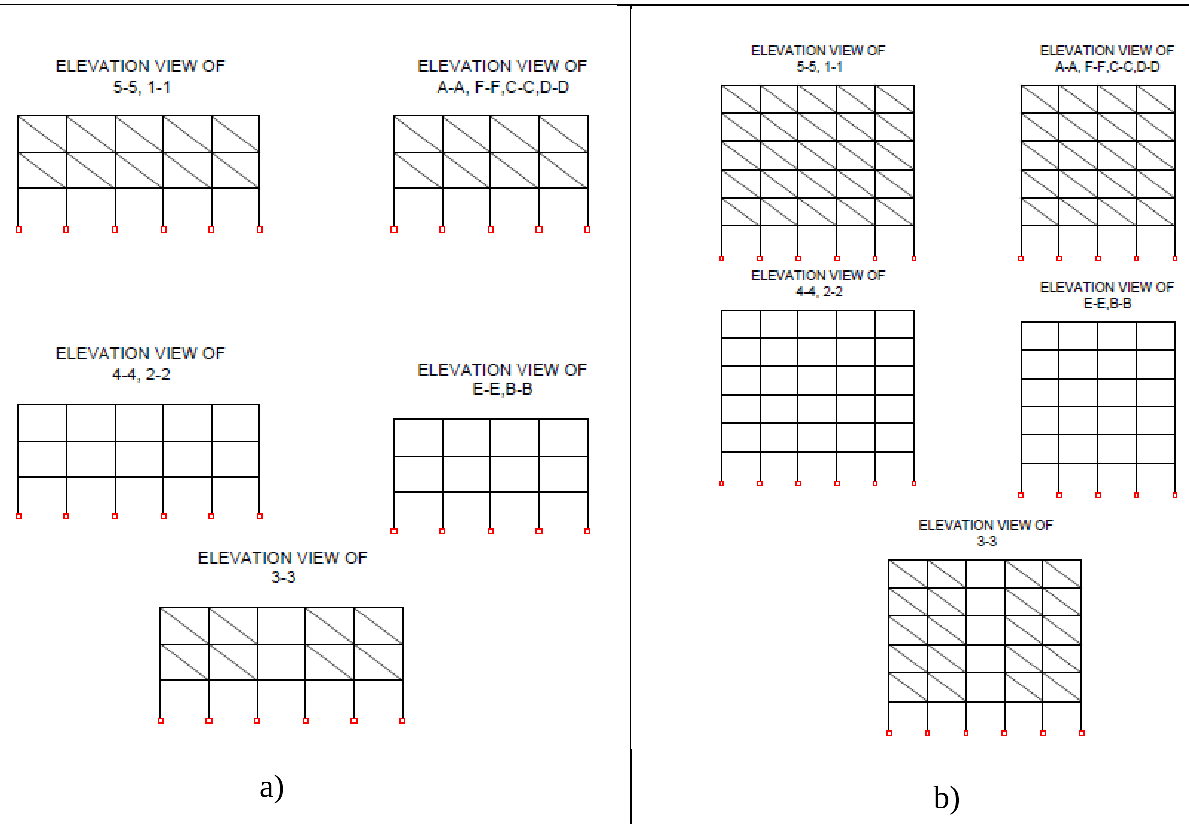


Figure 24. a) Location of masonry infill walls in elevation for the 3- story frame, b) Location of masonry infill walls in elevation for the 6- story frame

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. In this study short column is produced by semi-infilled frames. As seen in the *Figure 25a* below because of two seminfilled bays 4 columns have become short.

Heavy overhangs shift the buildings mass center upwards and take it away from center of rigidity. Thus it has negative effects on seismic behavior. Past earthquakes revealed that buildings with heavy overhangs are more susceptible to damage. In this study are modeled overhangs at two cross sides of a building. For this purpose 1.5 m overhangs are attached to the regular building sides *Figure 25b.* The wall loadings are shuffled on the beams nearby the overhang portion.

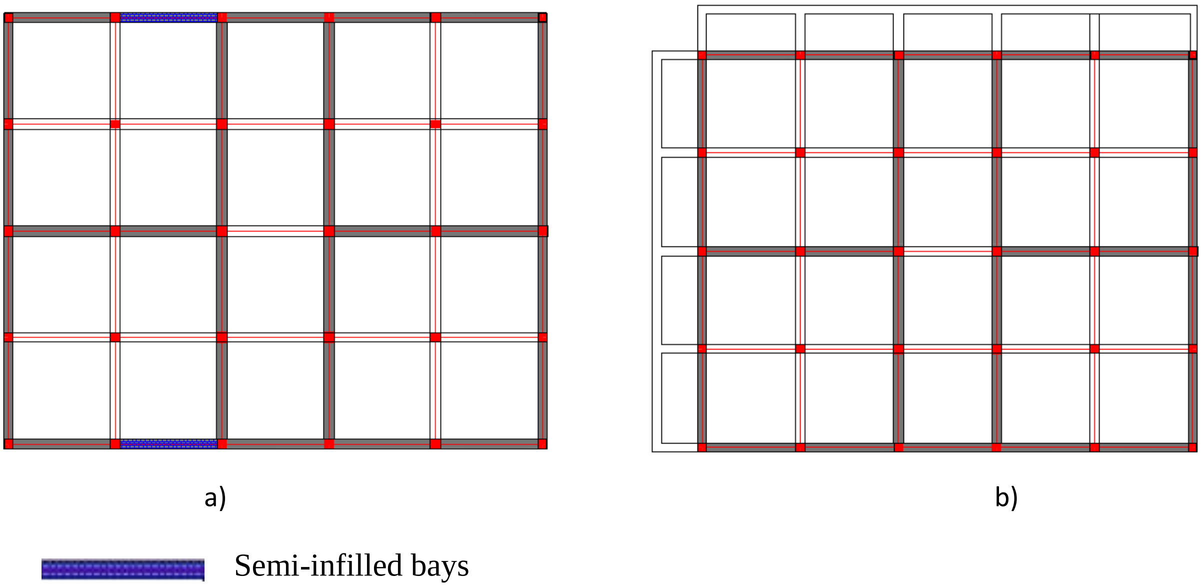


Figure 25. a) Semi-infilled frame (short column effect), b) Overhangs

In the Table 3 below are shown all cases that are analyzed in this study using both software's, ETABS and Seismosoft:

Table 3. Considered Frames

Ref	Reference Building (without any irregularity).
TSO	Two sided overhang building, *Figure 25b*.
SSH	Soft story due to 4.5 m ground story height (instead of 3 m).
SSW	Soft story due to absence of masonry infill wall at ground story, *Figure 24, 25*.
SS-H-W	Soft story due to both height and infill effect.
SS-H-W-TSO	Soft story due to both height and infill, and two sided overhang.
SHC	Short column due to sem-infilled bays at ground story, *Figure 25a.*

3.3. Modelling in ETABS

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 shown for the Ref 3 story frame.

3.3.1 Modeling of frame elements

Firstly material and frame sections are defined in accordance with the properties defined in section 3.2. Then the selected frames are modelled by using ETABS software, in *Figure 26* below is shown elevation view of the 'Ref' 3-story reinforced concrete frame:

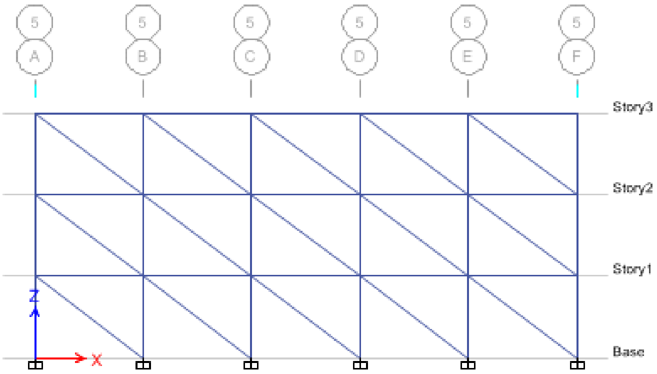


Figure 26. Elevation View of the Ref 3-story Frame

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

3.3.2 Modelling of masonry Infill walls (as diagonal strut elements)

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=1MPa
- Shear strength= 0.15MPa

- Stiffness: The elastic stiffness of a masonry infill panel is represented by an equivalent diagonal compression strut with width “a” as in *Equation 3.3.2.1-2* below. The strut have the same thickness and modulus of elasticity as the infill panel it represents [FEMA 356, 2000]*:*

$$a = 0.175(\lambda_1 h_{col})^{-0.4} r_{inf} \tag{3.3.2.1}$$

Where,

$$\lambda_1 = \left[\frac{E_{me}\, t_{inf} \sin 2\theta}{4E_{fe}\, I_{col}\, h_{inf}}\right]^{\frac{1}{4}} \tag{3.3.2.2}$$

h_{inf} = Height of infill panel

E_{fe} = Expected modulus of elasticity of frames 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 *Figure 27*, for which if thickness and modulus of elasticity are assumed the same as those of the masonry just width is needed to be determined.

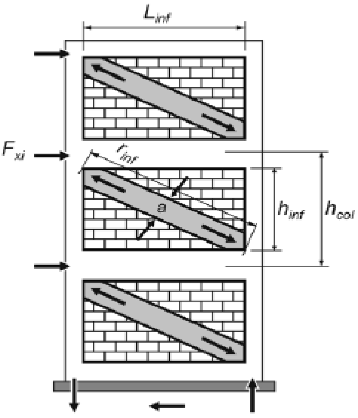


Figure 27. Masonry infill walls as single equivalent diagonal strut [FEMA 356, 2000]

- Strength: 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 *Equation 3.3.2.3* below [FEMA 356, 2000]:

$$Q_{CE} = V_{ine} = A_{ni}\, f_{vie} \qquad (3.3.2.3)$$

Where:

A_{ni} = Area of net mortared/grouted section across infill panel

f_{vie} = Expected shear strength of masonry infill

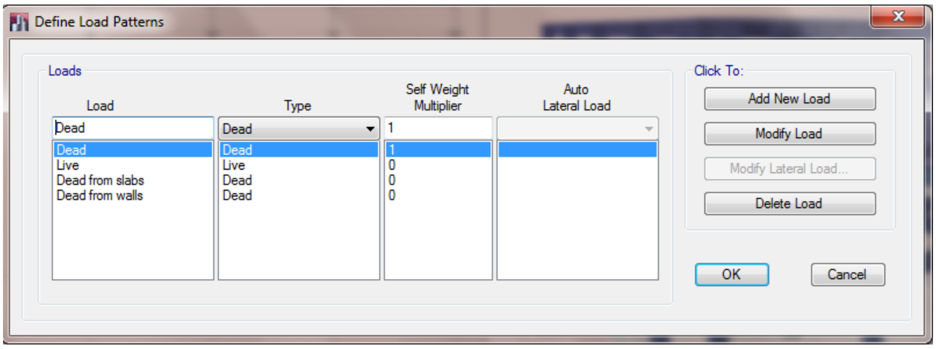
In *Equation 3.3.2.3* expected shear strength should not exceed the expected masonry bed-joint shear strength.

After modelling the masonry infill walls, load patterns are defined, *Figure 28-29*:

- Dead load
- Live load

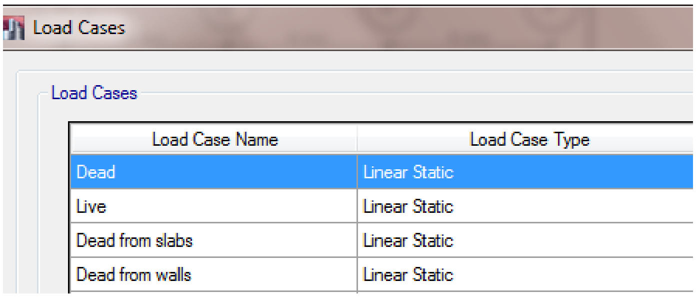
- Dead load from slabs
- Dead loads of infill walls

Self-weight multiplier of dead loads from infill walls and slabs will be 0.



Load	Type	Self Weight Multiplier	Auto Lateral Load
Dead	Dead	1	
Dead	Dead	1	
Live	Live	0	
Dead from slabs	Dead	0	
Dead from walls	Dead	0	

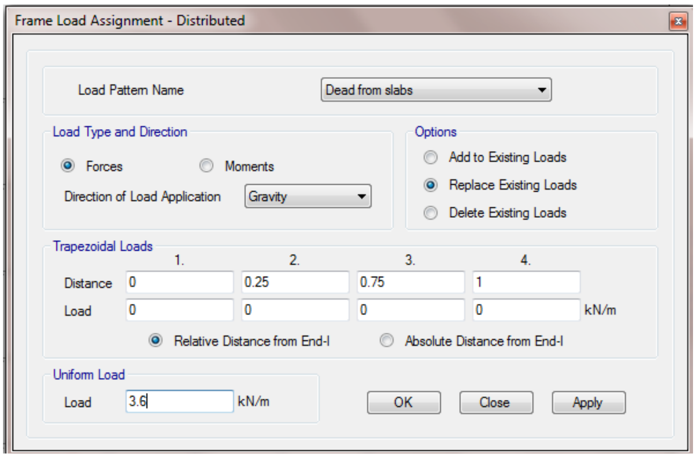
Figure 28. Defining Load Patterns



Load Case Name	Load Case Type
Dead	Linear Static
Live	Linear Static
Dead from slabs	Linear Static
Dead from walls	Linear Static

Figure 29. Defining Load Cases

Since after the linear analysis the nonlinear analysis will be 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, *Figure 30* below:



Load Pattern Name: Dead from slabs

● Forces ○ Moments

Direction of Load Application: Gravity

○ Add to Existing Loads
● Replace Existing Loads
○ Delete Existing Loads

	1.	2.	3.	4.	
Distance	0	0.25	0.75	1	
Load	0	0	0	0	kN/m

● Relative Distance from End-I ○ Absolute Distance from End-I

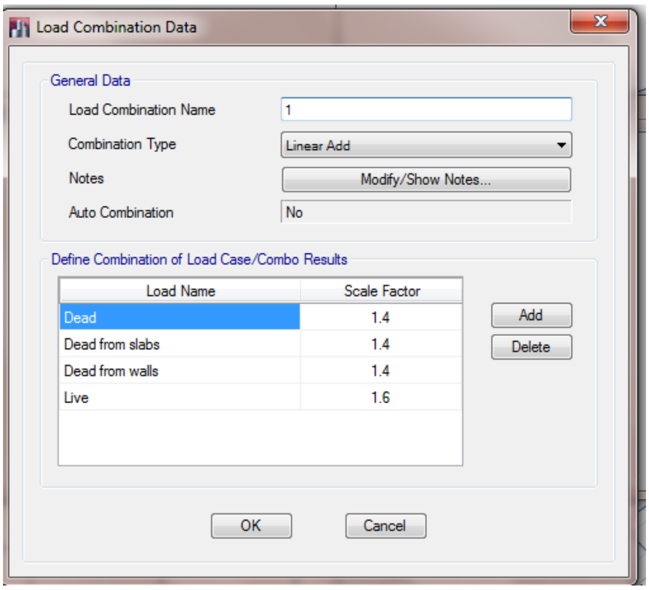
Load: 3.6 kN/m

Figure 30. Frame load assignment

After assigning all the considered loads, their combination is done.

Two load combinations are considered: Load combinations in this linear analysis will be two:

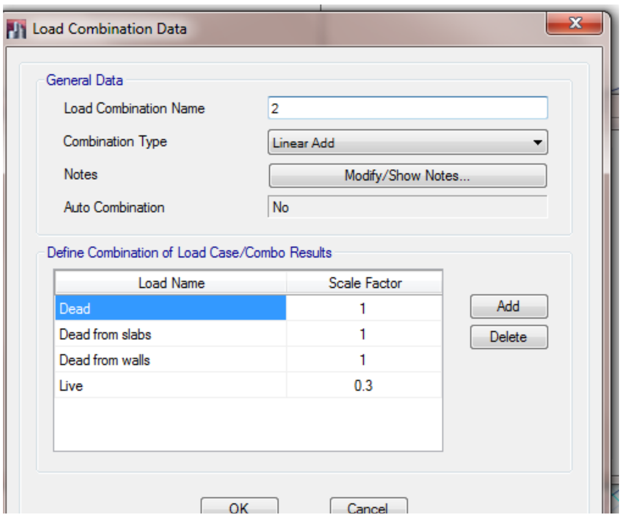
- 1.4 DD (dead loads) + 1.6 LL (live loads), *Figure 31*
- 1.0 DL (dead loads) + 0.3 LL (live loads), *Figure 32*



Load Combination Name: 1
Combination Type: Linear Add
Notes
Auto Combination: No

Load Name	Scale Factor
Dead	1.4
Dead from slabs	1.4
Dead from walls	1.4
Live	1.6

Figure 31. Load combinations 1

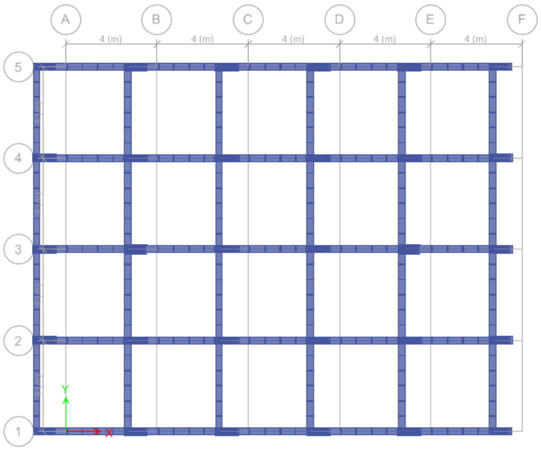


Load Combination Name: 2
Combination Type: Linear Add
Notes
Auto Combination: No

Load Name	Scale Factor
Dead	1
Dead from slabs	1
Dead from walls	1
Live	0.3

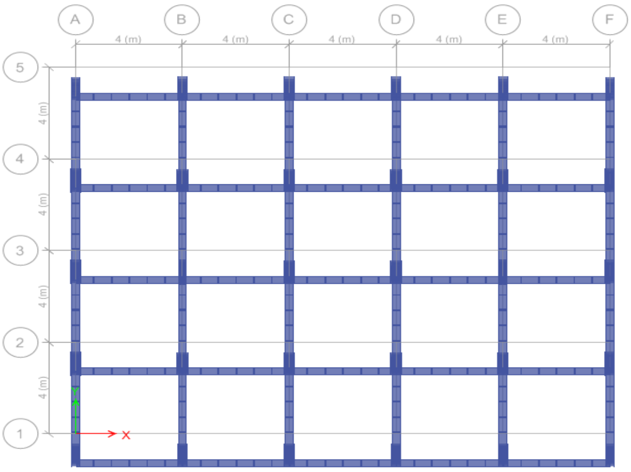
Figure 32. Load combinations 2

After defining the load combinations the linear analysis is carried out. By choosing Modal case we can obtain the periods for the 9 first modes. In the *Figures 33a-b* below are shown the deformations of frame in both X and Y-directions respectively:



a)

Deformation of the frame in X- direction, with period T_1=0.359. Deformation of the frame in Y- direction, with period T_2=0.346.



b)

Figure 33. a) Mode in X-direction, b) Mode in Y-direction

In the *Table 4* below are shown the period values for the first nine modes of the Ref Frame:

Table 4. Firts 9 modes of the Ref 3 story Frame

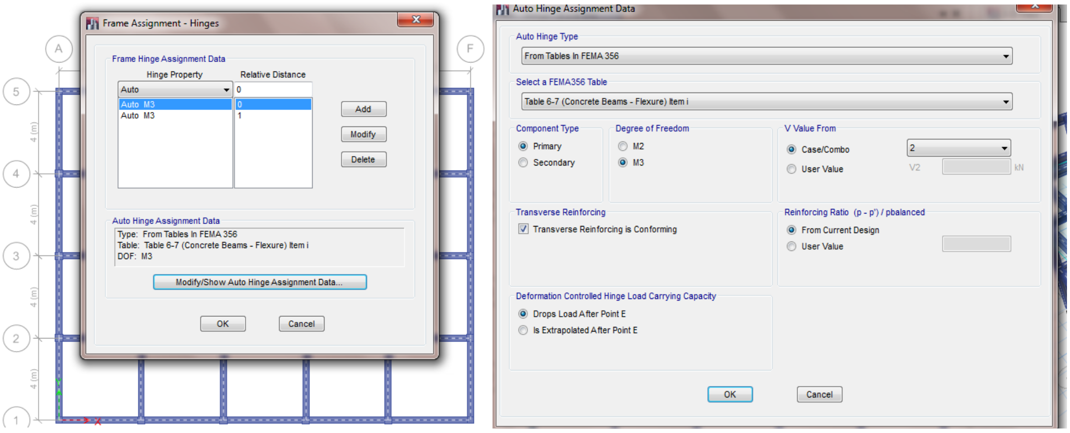
Mode	Period Sec	Frequency Cyc/sec	Circular frequency Rad/sec	Eigenvalue Rad²/sec²
1	0.359	2.782	17.4811	305.59
2	0.346	2.892	18.1679	330.07
3	0.315	3.175	19.9476	397.91
4	0.125	8.019	50.3828	2538.43
5	0.12	8.324	52.3027	2735.58
6	0.111	9.006	56.5835	3201.69
7	0.083	12.078	75.8866	5758.78
8	0.08	12.529	78.7249	6197.61
9	0.076	13.16	82.6836	6836.58

3.3.3 Nonlinear static pushover analysis:

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 have mentioned the modelling procedures, acceptance criteria and analysis procedures for the pushover analysis.

Nonlinear properties of the frame elements are assigned as 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, *Figure 34.*



Frame Assignment - Hinges

Frame Hinge Assignment Data

Hinge Property	Relative Distance
Auto	0
Auto M3	0
Auto M3	1

Auto Hinge Assignment Data

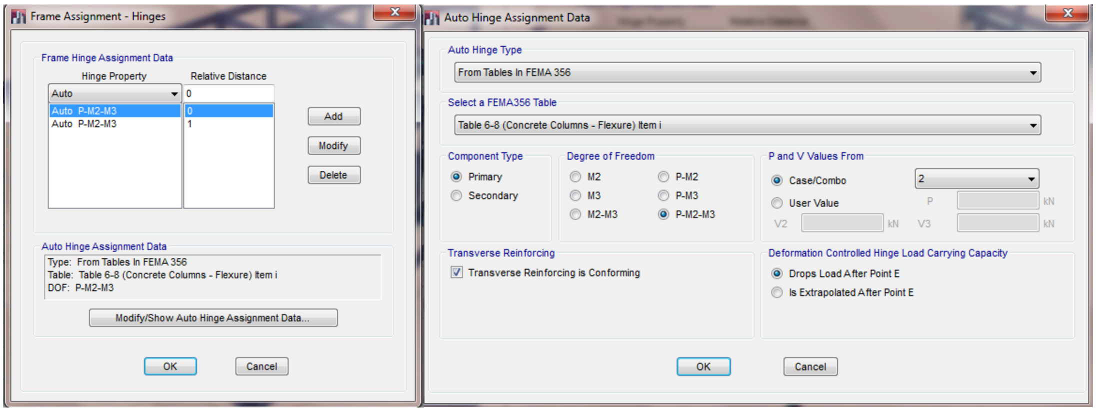
Type: From Tables In FEMA 356
Table: Table 6-7 (Concrete Beams - Flexure) Item i
DOF: M3

Auto Hinge Assignment Data

Auto Hinge Type: From Tables In FEMA 356
Select a FEMA356 Table: Table 6-7 (Concrete Beams - Flexure) Item i
Component Type: ● Primary ○ Secondary
Degree of Freedom: ○ M2 ● M3
V Value From: ● Case/Combo 2 ○ User Value V2 kN
Transverse Reinforcing: ☑ Transverse Reinforcing is Conforming
Reinforcing Ratio (p - p') / pbalanced: ● From Current Design ○ User Value
Deformation Controlled Hinge Load Carrying Capacity: ● Drops Load After Point E ○ Is Extrapolated After Point E

Figure 34. Beam assignment - Hinges

- Columns: Plastic hinges for columns with have different degree of freedom, axial force and rotational moment in both directions, P-M2-M3, *Figure 35*.



Frame Assignment - Hinges

Frame Hinge Assignment Data

Hinge Property	Relative Distance
Auto	0
Auto P-M2-M3	0
Auto P-M2-M3	1

Auto Hinge Assignment Data

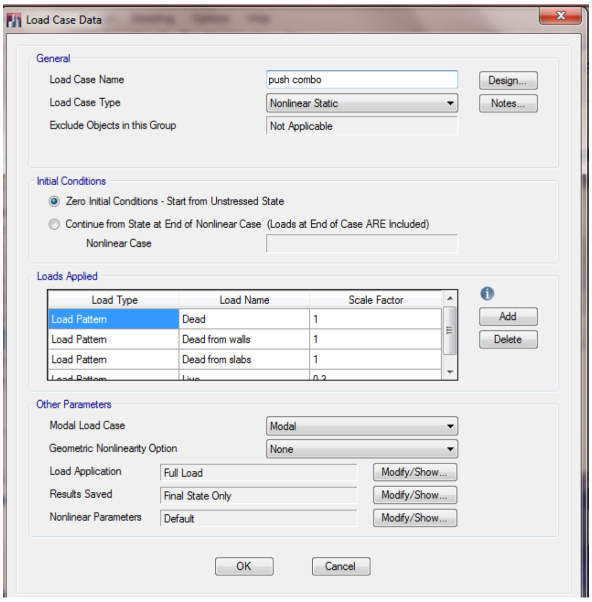
Type: From Tables In FEMA 356
Table: Table 6-8 (Concrete Columns - Flexure) Item i
DOF: P-M2-M3

Auto Hinge Assignment Data

Auto Hinge Type: From Tables In FEMA 356
Select a FEMA356 Table: Table 6-8 (Concrete Columns - Flexure) Item i
Component Type: ● Primary ○ Secondary
Degree of Freedom: ○ M2 ○ P-M2 ○ M3 ○ P-M3 ○ M2-M3 ● P-M2-M3
P and V Values From: ● Case/Combo 2 ○ User Value P kN V2 kN V3 kN
Transverse Reinforcing: ☑ Transverse Reinforcing is Conforming
Deformation Controlled Hinge Load Carrying Capacity: ● Drops Load After Point E ○ Is Extrapolated After Point E

Figure 35. Column assignment – Hinges

After assigning the plastic hinges the load cases for the nonlinear static pushover analysis will be 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, *Figure 36*.


Load Case Data

General

Load Case Name: push combo

Load Case Type: Nonlinear Static

Exclude Objects in this Group: Not Applicable

Initial Conditions

● Zero Initial Conditions - Start from Unstressed State

○ Continue from State at End of Nonlinear Case (Loads at End of Case ARE Included)

Nonlinear Case:

Loads Applied

Load Type	Load Name	Scale Factor
Load Pattern	Dead	1
Load Pattern	Dead from walls	1
Load Pattern	Dead from slabs	1

Other Parameters

Modal Load Case: Modal

Geometric Nonlinearity Option: None

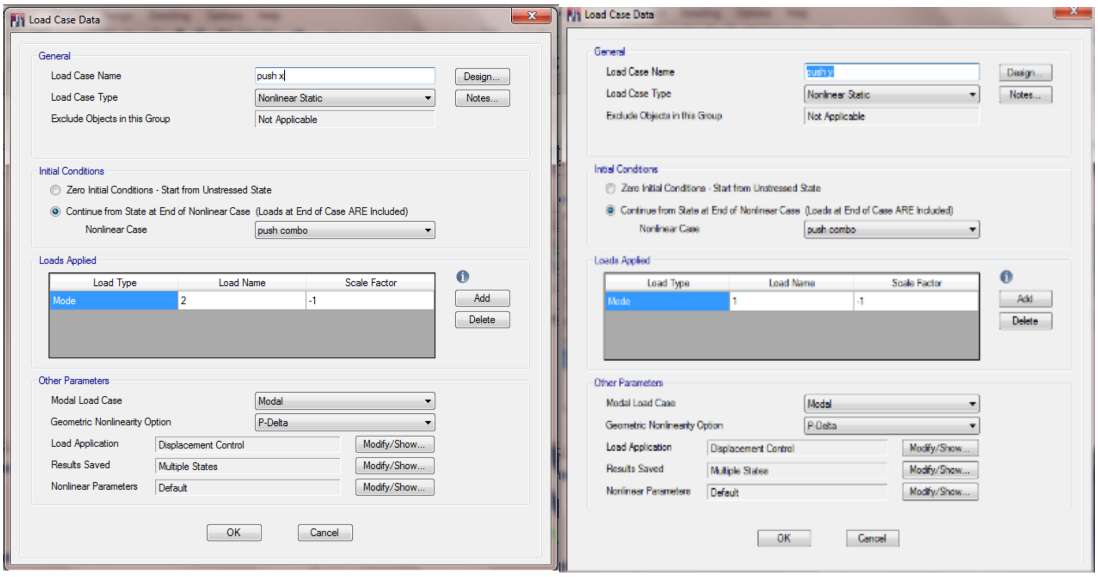
Load Application: Full Load

Results Saved: Final State Only

Nonlinear Parameters: Default

Figure 36. Load Case- Push Combo

Pushover Analysis is performed in two directions, X and Y directions, so two nonlinear case loads will be defined *Figure 37a-b*, push x and push y which will start from push combo under Mode load pattern. Geometric nonlinearity will be taken in consideration by choosing the P-Delta effect. Scale factor for X and Y-directions is -1 since the first and second mode displacement was in -x & -y-directions. Results are displayed in multiples steps. For the displacement control the maximum displacement at the top of the building is considered.


Load Case Data

General

Load Case Name: push x

Load Case Type: Nonlinear Static

Exclude Objects in this Group: Not Applicable

Initial Conditions

○ Zero Initial Conditions - Start from Unstressed State

● Continue from State at End of Nonlinear Case (Loads at End of Case ARE Included)

Nonlinear Case: push combo

Loads Applied

Load Type	Load Name	Scale Factor
Mode	2	-1

Other Parameters

Modal Load Case: Modal

Geometric Nonlinearity Option: P-Delta

Load Application: Displacement Control

Results Saved: Multiple States

Nonlinear Parameters: Default

Load Case Data

General

Load Case Name: push y

Load Case Type: Nonlinear Static

Exclude Objects in this Group: Not Applicable

Initial Conditions

○ Zero Initial Conditions - Start from Unstressed State

● Continue from State at End of Nonlinear Case (Loads at End of Case ARE Included)

Nonlinear Case: push combo

Loads Applied

Load Type	Load Name	Scale Factor
Mode	1	-1

Other Parameters

Modal Load Case: Modal

Geometric Nonlinearity Option: P-Delta

Load Application: Displacement Control

Results Saved: Multiple States

Nonlinear Parameters: Default

Figure 37. a) Load Case- Push in X-direction, b) Load Case- Push in Y-direction

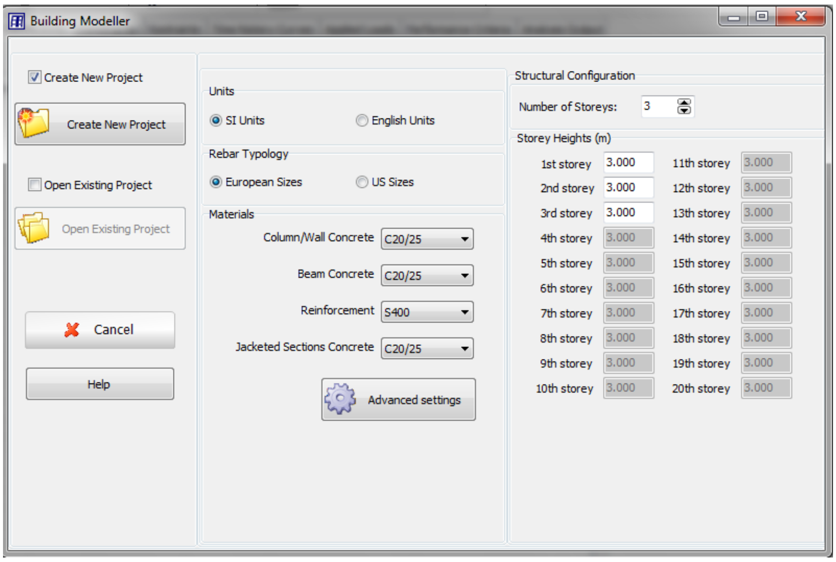
After defining all loads cases for the nonlinear analysis the rigid diaphragms is assigned to each story in order to concentrate the story weight in center of mass.

3.4. 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 analysed also by sesimosoft software in order to see the difference between two programs and compare the behaviour of considered reinforced concrete frames.

3.4.1 Modelling of frame elements

Firstly material properties and frame sections are defined as shown in *Figure 38* below:



☑ Create New Project

☐ Open Existing Project

Units

● SI Units ○ English Units

Rebar Typology

● European Sizes ○ US Sizes

Materials

Column/Wall Concrete C20/25

Beam Concrete C20/25

Reinforcement S400

Jacketed Sections Concrete C20/25

Structural Configuration

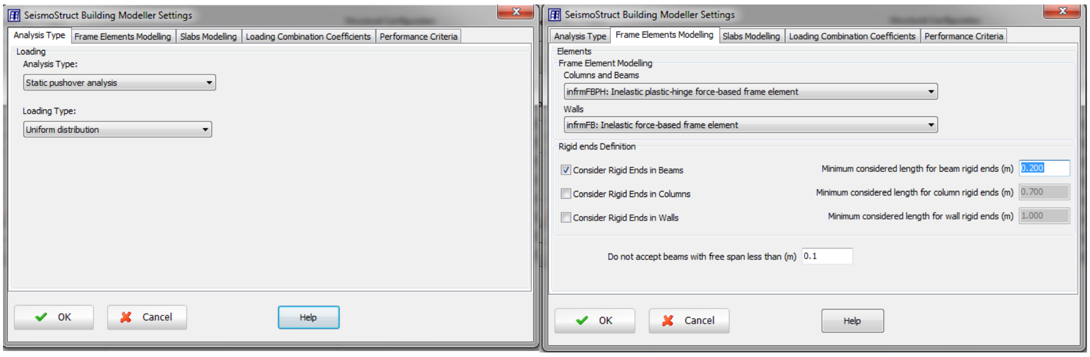
Number of Storeys: 3

Storey Heights (m)

1st storey	3.000	11th storey	3.000
2nd storey	3.000	12th storey	3.000
3rd storey	3.000	13th storey	3.000
4th storey	3.000	14th storey	3.000
5th storey	3.000	15th storey	3.000
6th storey	3.000	16th storey	3.000
7th storey	3.000	17th storey	3.000
8th storey	3.000	18th storey	3.000
9th storey	3.000	19th storey	3.000
10th storey	3.000	20th storey	3.000

Figure 38. Defining material properties and frame sections

After defining the materials, analysis and loading type are choosen *Figure 39a*, in this case static pushover analysis and uniform distribution respectively.



Loading

Analysis Type: Static pushover analysis

Loading Type: Uniform distribution

Elements

Frame Element Modelling

Columns and Beams: infrmFBPH: Inelastic plastic-hinge force-based frame element

Walls: infrmFB: Inelastic force-based frame element

Rigid ends Definition

☑ Consider Rigid Ends in Beams — Minimum considered length for beam rigid ends (m) 0.200

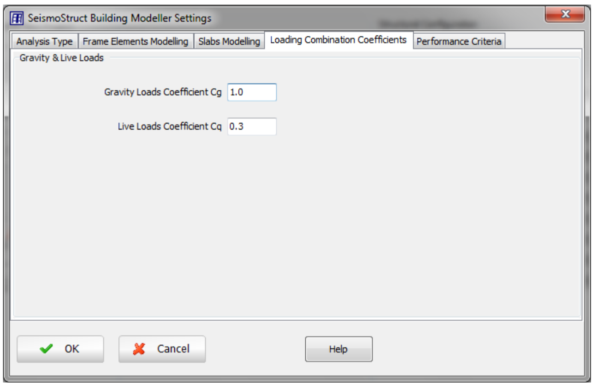
☐ Consider Rigid Ends in Columns — Minimum considered length for column rigid ends (m) 0.700

☐ Consider Rigid Ends in Walls — Minimum considered length for wall rigid ends (m) 1.000

Do not accept beams with free span less than (m) 0.1

a) b)

Figure 39. a) Defining analysis, loading type, b) frame element modelling



Gravity & Live Loads

Gravity Loads Coefficient Cg 1.0

Live Loads Coefficient Cq 0.3

Figure 40. Load combination factors

Frame elements are modelled as inelastic plastic hinge force based frame elements, *Figure 39b*. Loading combination factors is 1 for dead loads and 0.3 for live loads *Figure 40*. Then the modelling of the Ref frame for the 3 story case is done as shown in the *Figure 41* below:

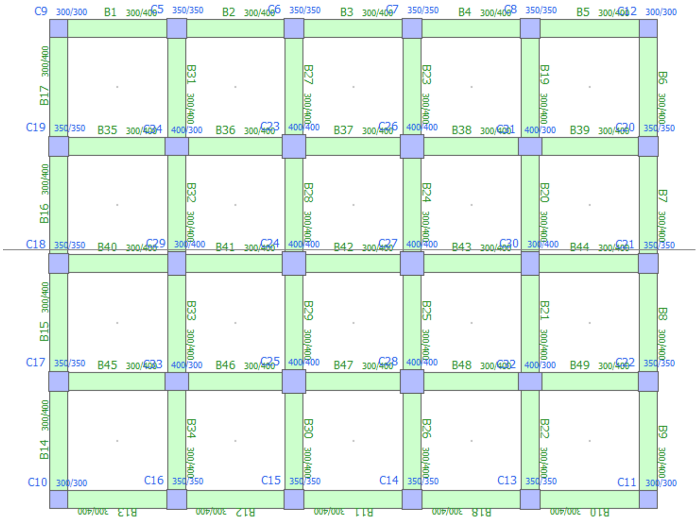
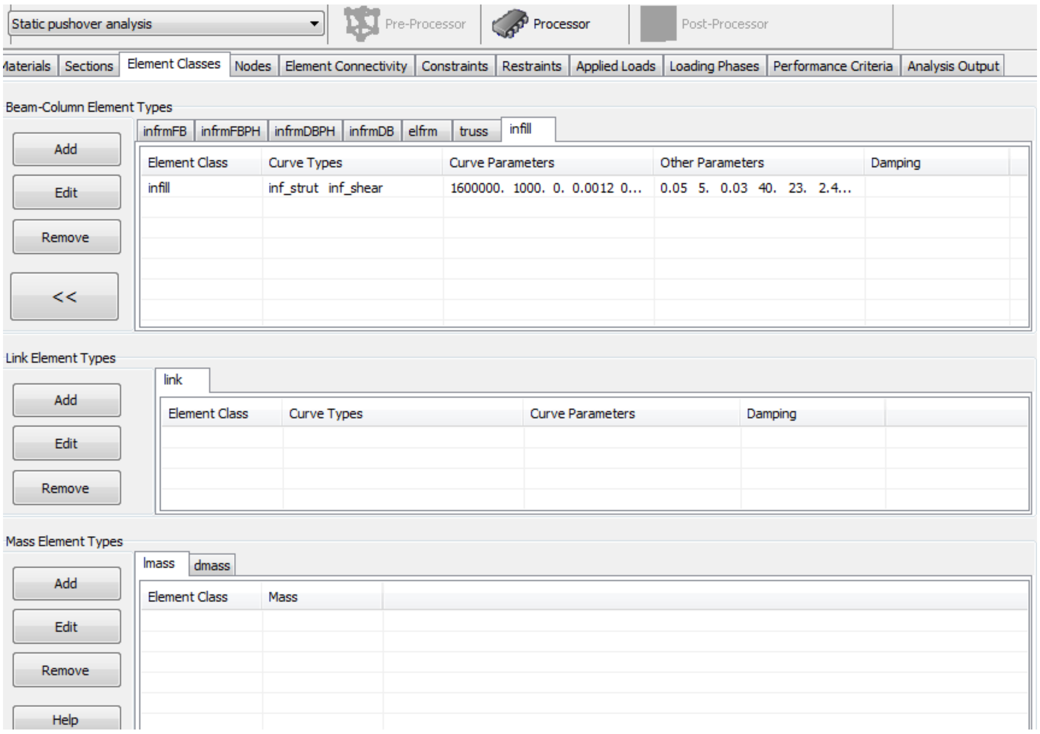


Figure 41. Plan view of 3 story Ref frame

After modelling the frame elements, masonry infill walls are modelled as diagonal strut elements *Figure 42*, in accordance with the structural plan of the 3 story frame where location of masonry infill walls is shown.



Static pushover analysis

Element Class	Curve Types	Curve Parameters	Other Parameters	Damping
infill	inf_strut inf_shear	1600000. 1000. 0. 0.0012 0...	0.05 5. 0.03 40. 23. 2.4...	

Figure 42. Modelling of Masonry infill wall

After modelling of all elements and specifying the loads and pushover analysis parameters the analysis is runned out and the results are generated as shown in *Figure 56-57.*

CHAPTER 4

ANALYSIS RESULTS

4.1 General

Pushover analysis has been conducted for the 14 building models for both type of structures, 3 and 6-story buildings. The material nonlinearities are assigned as 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. Behavior of the structure is represented by capacity curves that represents the base shear force and displacement of the roof. *Figures 43-58* 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.

4.2 Nonlinear Static (Pushover) Analysis

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 have mentioned the modelling procedures, acceptance criteria and analysis procedures for the pushover analysis [FEMA 356, 2000]. This code defines the force-deformation criteria for possible locations of lumped inelastic behaviour defined as plastic hinges in the pushover analysis. In *Figure 43* is shown the plastic hinge force-deformation behaviour by using five points labelled as A, B, C, D, and E and three point's labelled IO (Immediate Occupancy), LS (Life Safety) and CP (Collapse Prevention) are used to define the acceptance criteria for the hinge. In this study these three points are correspond to 10%, 60% and 90% use of plastic hinge deformation capacity.

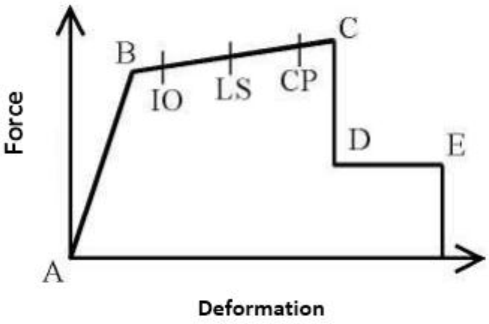


Figure 43. Deformation relationship for a typical hinge [FEMA 356, 2000].

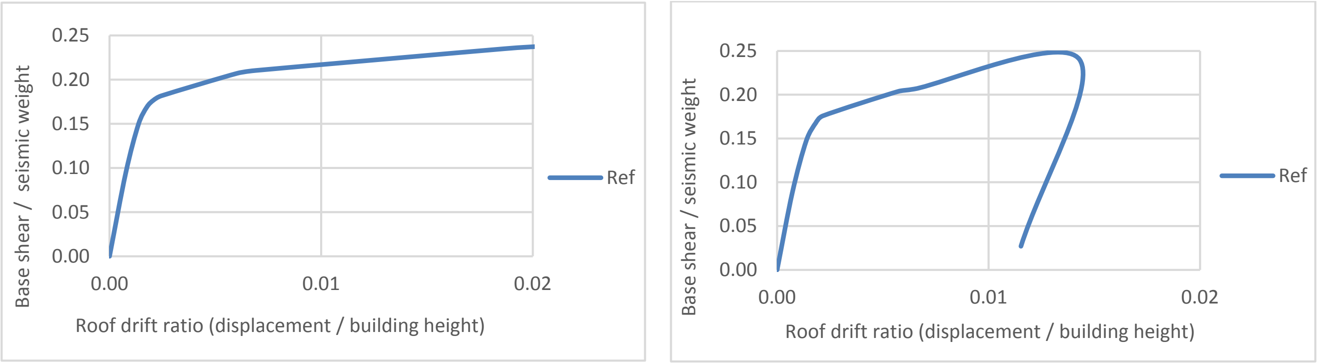
The values assigned to each of these points vary depending on the type of member as well as many other parameters, such as the expected type of failure, the level of stresses with respect to the strength, or code compliance.

4.3 Analysis Results

Nonlinear static (pushover) analysis is carried out by using both software's Etabs and Seismosoft. Performance of the considered frames is done by taking in consideration capacity graphs for two types of structures low and mid-rise buildings represented by 3 and 6-story respectively. Firstly the graphs are analysed separately in order to get the behaviour of each one of the considered frames. Then a comparison is done in order to see which one of the considered irregularities has the highest effect to the RC frames during an earthquake. Then a comparison between Etabs and Seismosoft results is done in order to see the differences between them.

4.2.1 ETABS

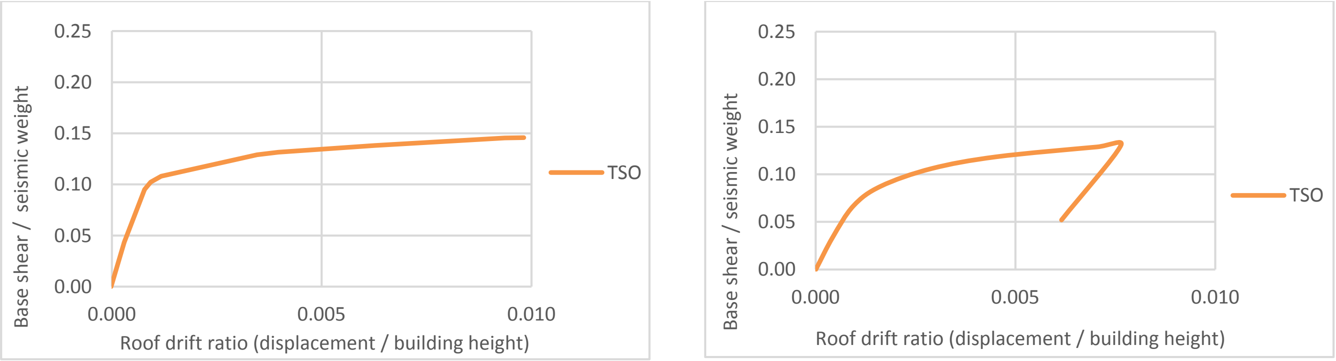
Fistly the normalized graphs of 3 story frame are represented:



a) b)

Figure 44. Capacity Curves of Ref 3-story frame: a) x-direction, b) y-direction

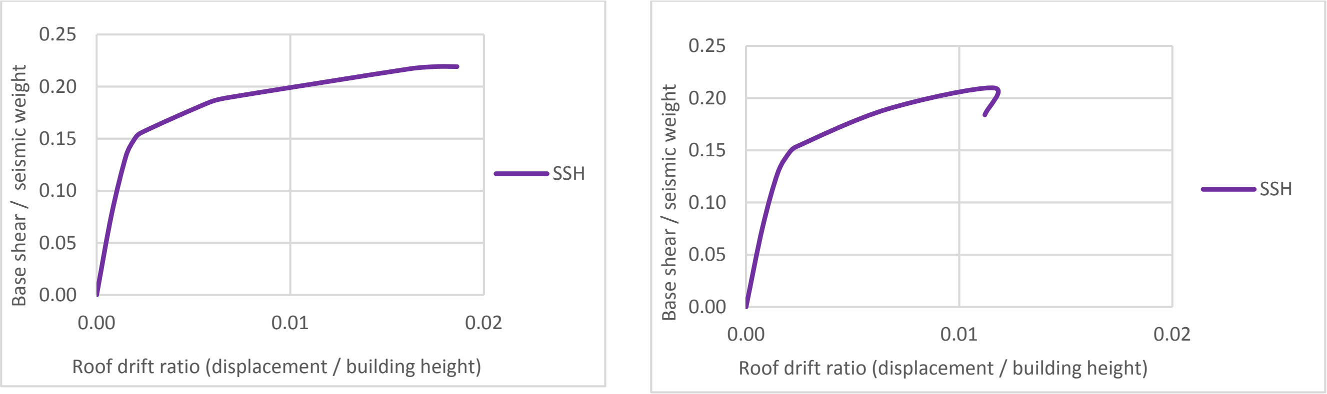
In *Figure 44a-b* are shown the capacity curves of Ref 3- story frame for both x and y directions. From the graphs it is seen that the drift ratios at significant strength loss are 2% for x direction and 1.3% for y direction respectively.



a) b)

Figure 45. Capacity Curves of TSO 3-story frame: a) x-direction, b) y-direction

In *Figure 45a-b* the capacity curves for both x and y direction of 3- Story TSO frame are shown. From the graphs it is seen that the drift ratios at significant strength loss are 0.95% and 0.75% for x and y direction respectively. Presence of overhangs has lowered the capacity with 60% in comparison with the Ref frame as seen from the graphs and drift ratios, *Figure 45a-b*.

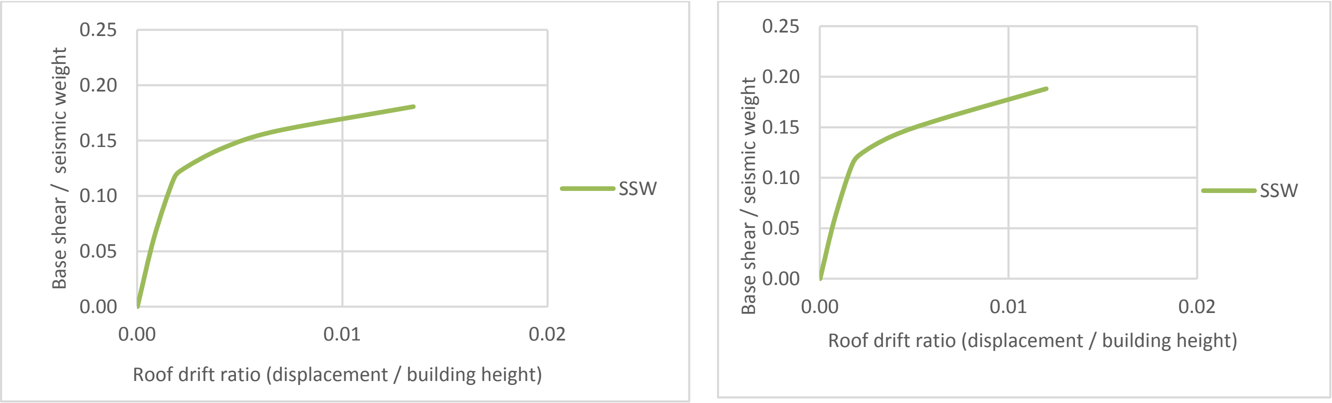


a)

b)

Figure 46. Capacity Curves of SSH 3-story frame: a) x-direction, b) y-direction

In the above graphs *Figure 46a-b* are shown the capacity curves for both x and y direction of 3 Story SSH frame. From the graphs it is seen that the presence of soft story irregularity both weakness and softens the frame. Drift ratios at significant strength loss are 1.9% and 1.25% for x and y direction respectively. Capacity of the frame is lowered in comparison with Ref frame, but it has better performance than TSO frame. So presence of heavy overhangs is more damaging than the presence of soft story irregularity.

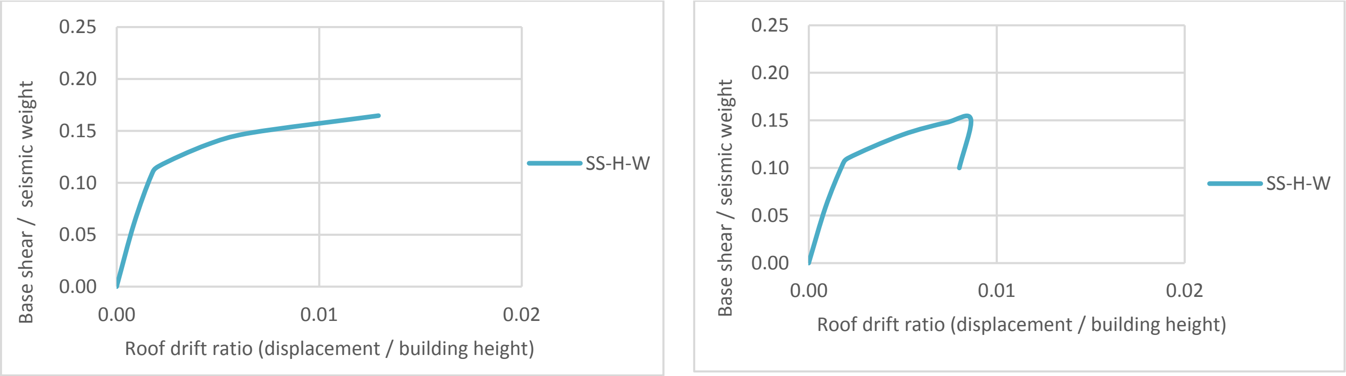


a)

b)

Figure 47. Capacity Curves of SSW 3-story frame: a) x-direction, b) y-direction

Then in *Figure 47a-b* are shown the capacity curves for both x and y direction of 3 Story SSW frame. From the graphs it is seen that the drift ratios at significant strength loss are 1.35% and 1.20% for x and y direction respectively. From the drift ratios presented in *Figure 47a-b* it is obviously seen that presence of soft story because of lack of masonry infill walls at ground story is more damaging than the presence of soft story because of higher ground story height. SSH frame has 40% higher stiffness and 22% higher strength for x direction and 4% higher stiffness and 13% higher strength for y direction than SSW frame.

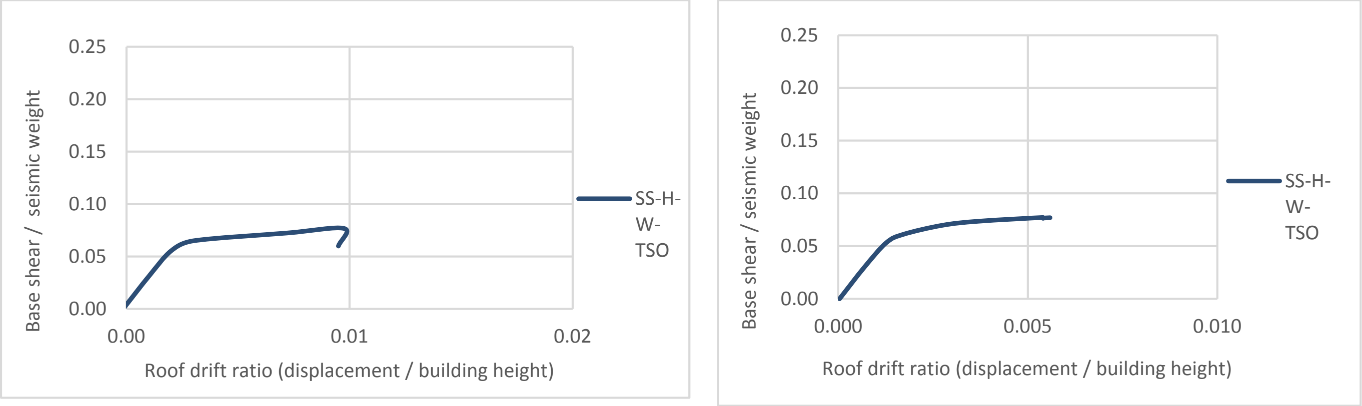


a) b)

Figure 48. Capacity Curves of SS-H-W 3-story frame: a) x-direction, b) y-direction

In *Figure 48a-b* are shown the capacity curves for both x and y direction of 3 Story SS-H-W frame. From the graphs it is seen that the drift ratios at significant strength loss are 1.25% and 0.85% for x and y direction respectively. So the presence of soft story irregularity because of both higher ground story height and lack of masonry infill walls at ground story is more destructive than presence of them separately. It has both lower strength and stiffness in comparison with SSH and SSW frames.

In the below graphs, *Figure 49a-b* are shown the normalized graps for both x and y direction of SS-H-W-TSO 3-story frame. In this type of frame both soft story and overhangs irregularities are present. Combining these irregularities in a single frame makes the frame to have lower capacity and performance during an earthquake.

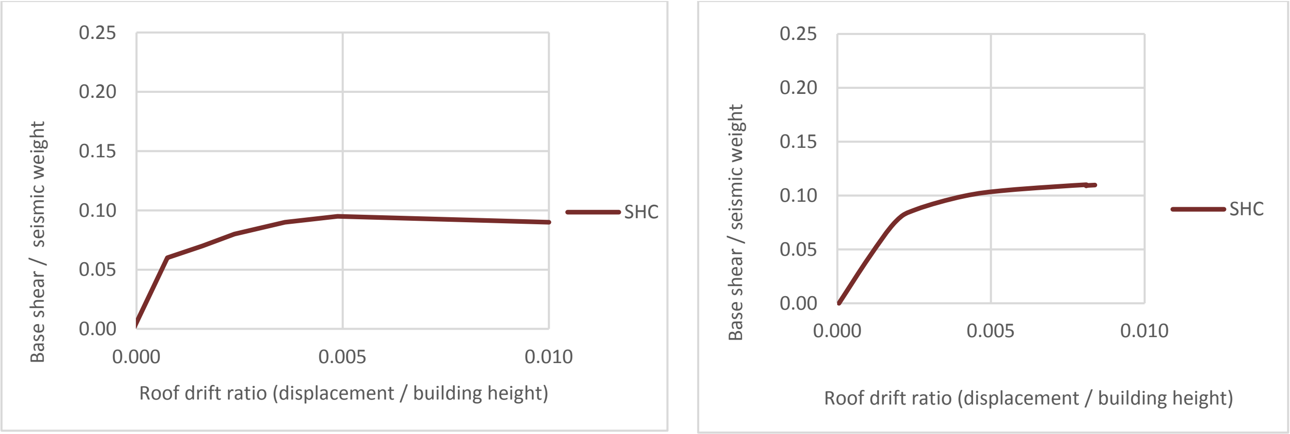


a) b)

Figure 49. Capacity Curves of SS-H-W-TSO 3-story frame: a) x-direction, b) y-direction

From the graphs *Figure 49a-b* it is seen that the drift ratios at significant strength loss are 0.9% and 0.6% for x and y direction respectively. It has 40% lower stiffness and 50% lower strength than SS-H-W for both x and y direction.

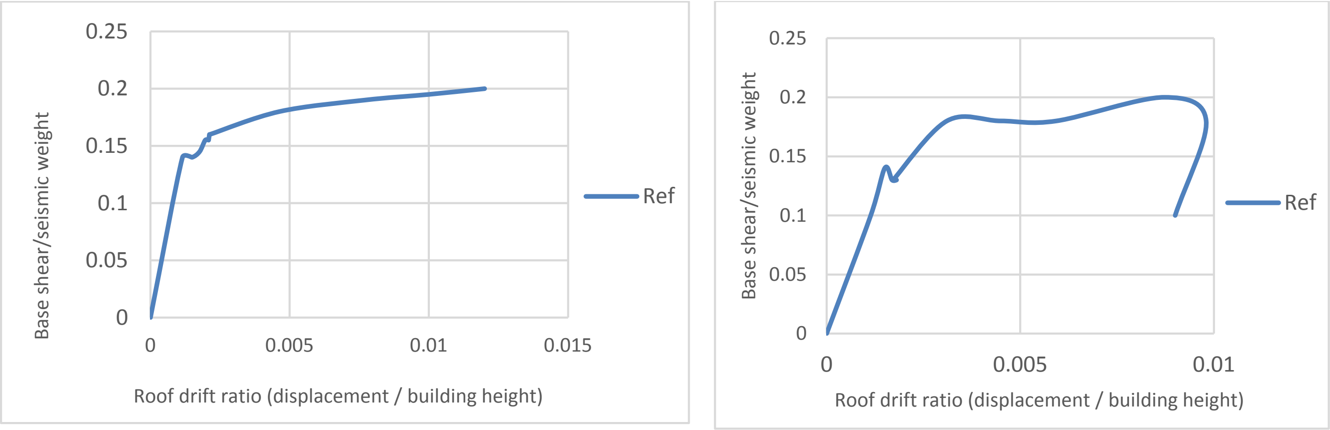
In *Figure 50a-b* the capacity curves for both x and y direction of 3 Story SHC frame are shown. From the graphs it is seen that the drift ratios at significant strength loss are 1% and 0.85% for x and y direction respectively. Comparing the values of drift ratios for all the considered 3 story frames it is observed that presence of soft story and overhangs at the same time, presence of short column has the highest effect to a RC frame under earthquake loads.



a) b)

Figure 50. Capacity Curves of SHC 6- story frame: a) x-direction, b) y-direction

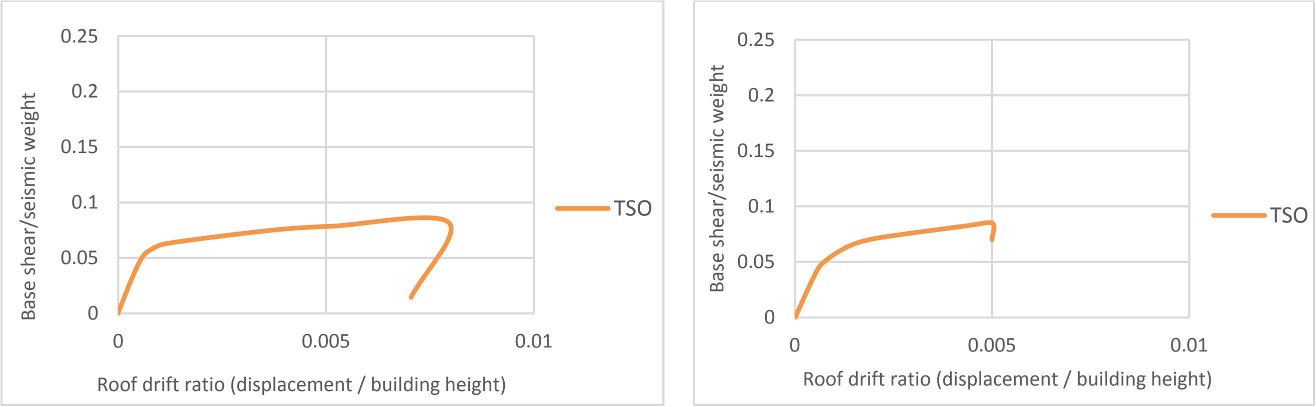
Then normalized graphs of the 6 story frames are shown:



a) b)

Figure 51. Capacity Curves of Ref 6-story frame: a) x-direction, b) y-direction

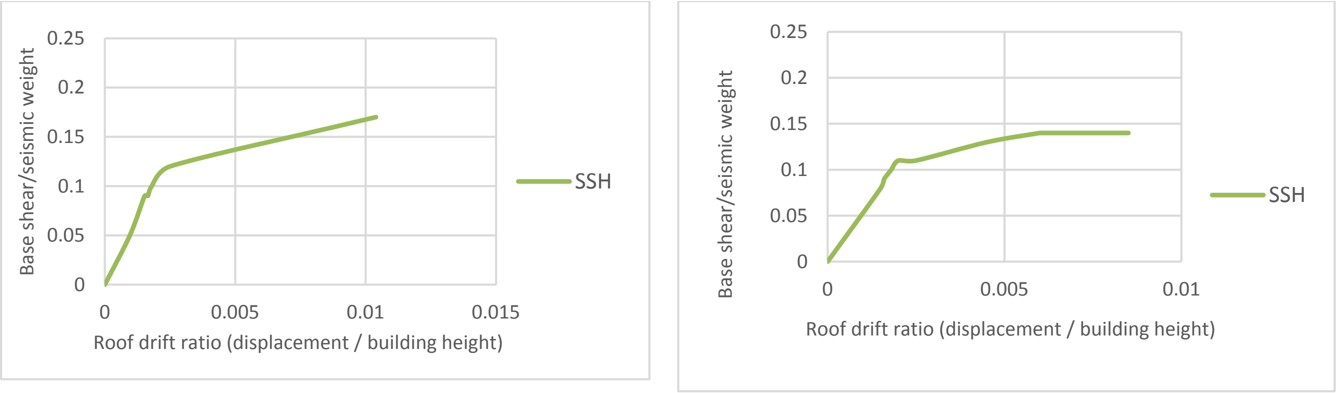
In *Figure 51a-b* are shown the capacity curves of Ref 6- story frame for both x and y directions. From the graphs it is seen that the drift ratios at significant strength loss are 1.25% for x direction and 0.9% for y direction respectively.



a) b)

Figure 52. Capacity Curves of TSO 6-story frame: a) x-direction, b) y-direction

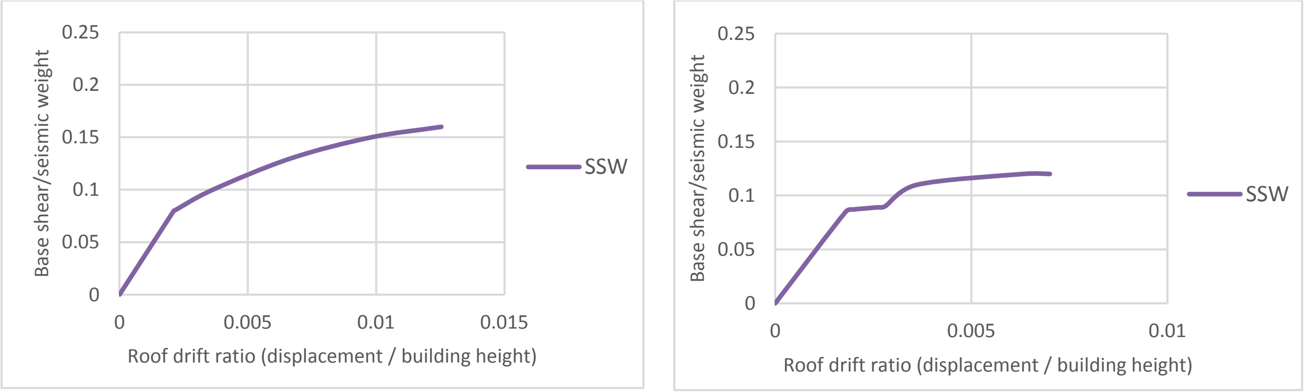
In *Figure 52a-b* the capacity curves for both x and y direction of 3 Story TSO frame are shown. From the graphs is seen that the drift ratios at significant strength loss are 0.8% and 0.5% for x and y direction respectively. Presence of overhangs has lowered the capacity of the frame as seen from the graphs and drift ratios, *Figure 51a-b*.



a) b)

Figure 53. Capacity Curves of SSH 6-story frame: a) x-direction, b) y-direction

In the above graphs *Figure 53a-b* are shown the capacity curves for both x and y direction of 3 Story SSH frame. From the graphs it is seen that the presence of soft story irregularity both weakness and softenes the frame. Drift ratios at significant strength loss are 1.05% and 0.80% for x and y direction respectively. Capacity of the frame is lowered in comparison with Ref frame, but it has better perfomnce than TSO frame. So presence of heavy overhangs is more damaganing tha presence of soft story irregularity in 6 story frame also.

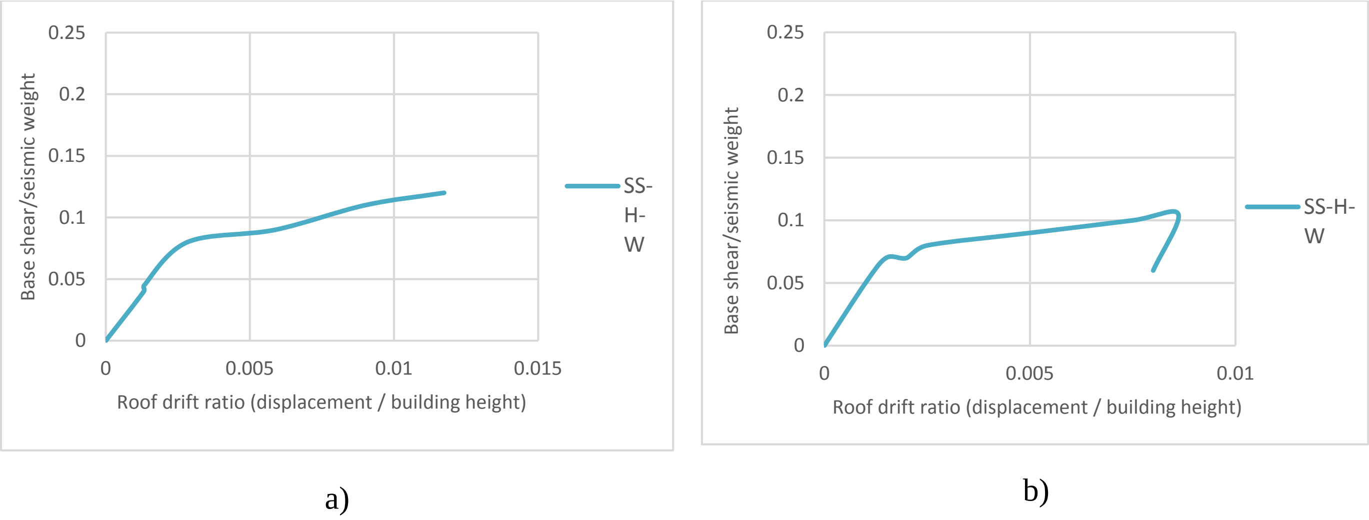


a) b)

Figure 54. Capacity Curves of SSW 6-story frame: a) x-direction, b) y-direction

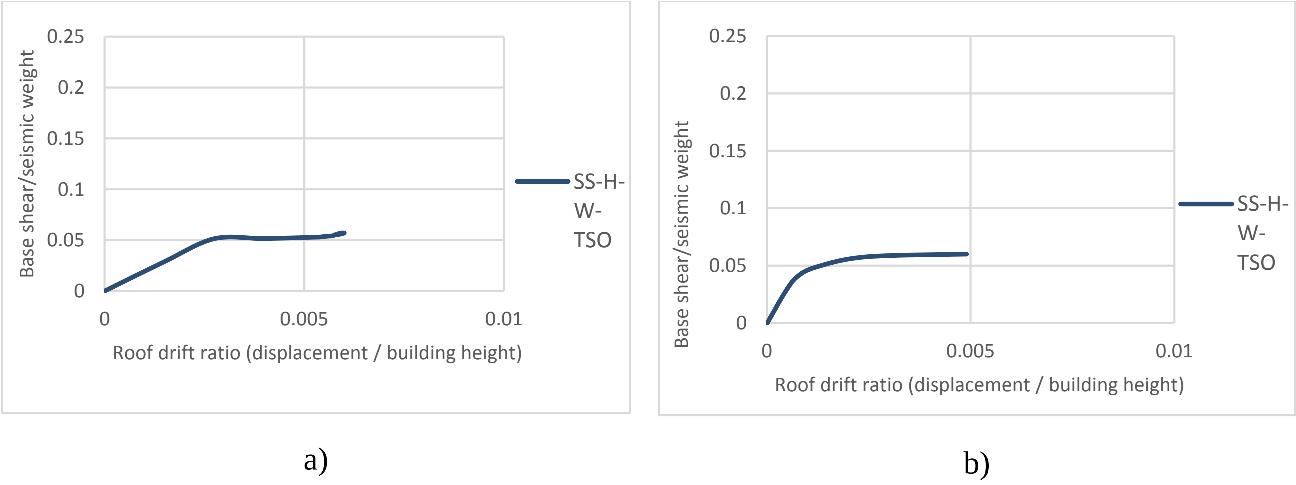
Then in *Figure 54a-b* are shown the capacity curves for both x and y direction of 6 Story SSW frame. From the graphs is seen that the drift ratios at significant strength loss are 1.25% and 0.75% for x and y direction respectively. From the drift ratios presented in *Figure 46a-b* it is obviously seen that presence of soft story because of lack of masonry infill walls at ground story is more damaging than the presence of soft story because of higher ground story height. SSH frame has 12% higher stiffness and 38% higher strength for x direction and 3% higher stiffness and 16%higher strength for y direction than SSW frame.

In *Figure 55a-b* the capacity curves for both x and y direction of 6 Story SS-H-W frame are shown.



a) b)

Figure 55. Capacity Curves of SS-H-W 6-story frame: a) x-direction, b) y-direction



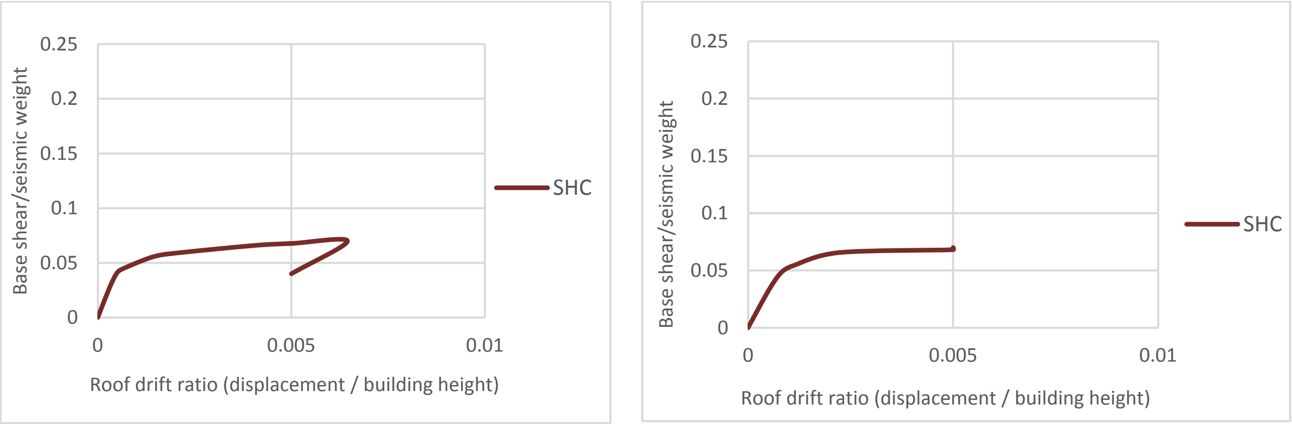
a) b)

Figure 56. Capacity Curves of SS-H-W-TSO 6-story frame: a) x-direction, b) y-direction

From the graphs *Figure 55a-b* is seen that the drift ratios at significant strength loss are 1.25% and 0.85% for x and y direction respectively. So the presence of soft story irregularity because of both higher ground story height and lack of masonry infill walls at ground story is more destructive than presence of them separately.

In the above graphs, *Figure 56a-b* are shown the normalized graphs for both x and y directions of SS-H-W-TSO frame. In this type of frame both soft story and overhangs irregularities are present. Combining these irregularities in a single frame makes the frame to have lower capacity and performance during an earthquake. From the graphs is seen that the drift ratios at significant strength loss are 0.65% and 0.48% for x and y direction respectively.

In *Figure 57a-b* the capacity curves for both x and y direction of 6 Story SHC frame are shown. From the graphs is seen that the drift ratios at significant strength loss are 0.68% and 0.5% for x and y direction respectively



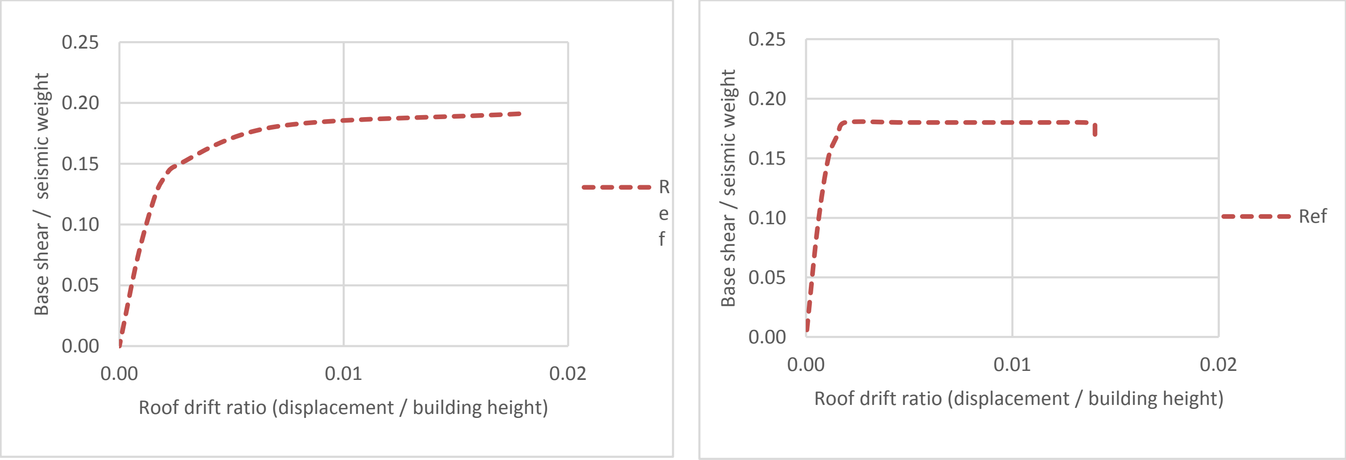
a) b)

Figure 57. Capacity Curves of SHC 6-story frame: a) x-direction, b) y-direction

Comparing the values of drift ratios for all the considered 6 story frames it is observed that presence of soft story and overhangs at the same time, presence of short column have the highest effect to a RC frame under earthquake loads.

4.2.2 Seismosoft

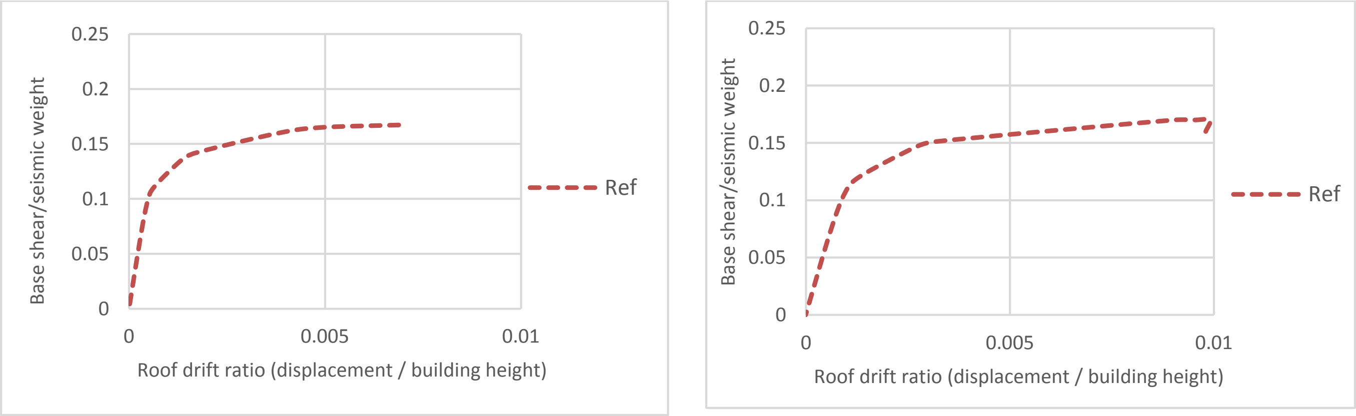
In *Figure 58a-b* and *Figure 59a-b* are shown the capacity curves of Ref 3 and 6- story frame for both x and y directions. The nonlinear static analysis is carried out by using Seismosft software.



a) b)

Figure 58. Capacity Curves of Ref 3- story frame: a) x-direction, b) y-direction

From the above graphs, *Figure 58a-b*, it is seen that the drift ratios at significant strength loss are 1.8% for x direction and 1.4% for y direction respectively.



a) b)

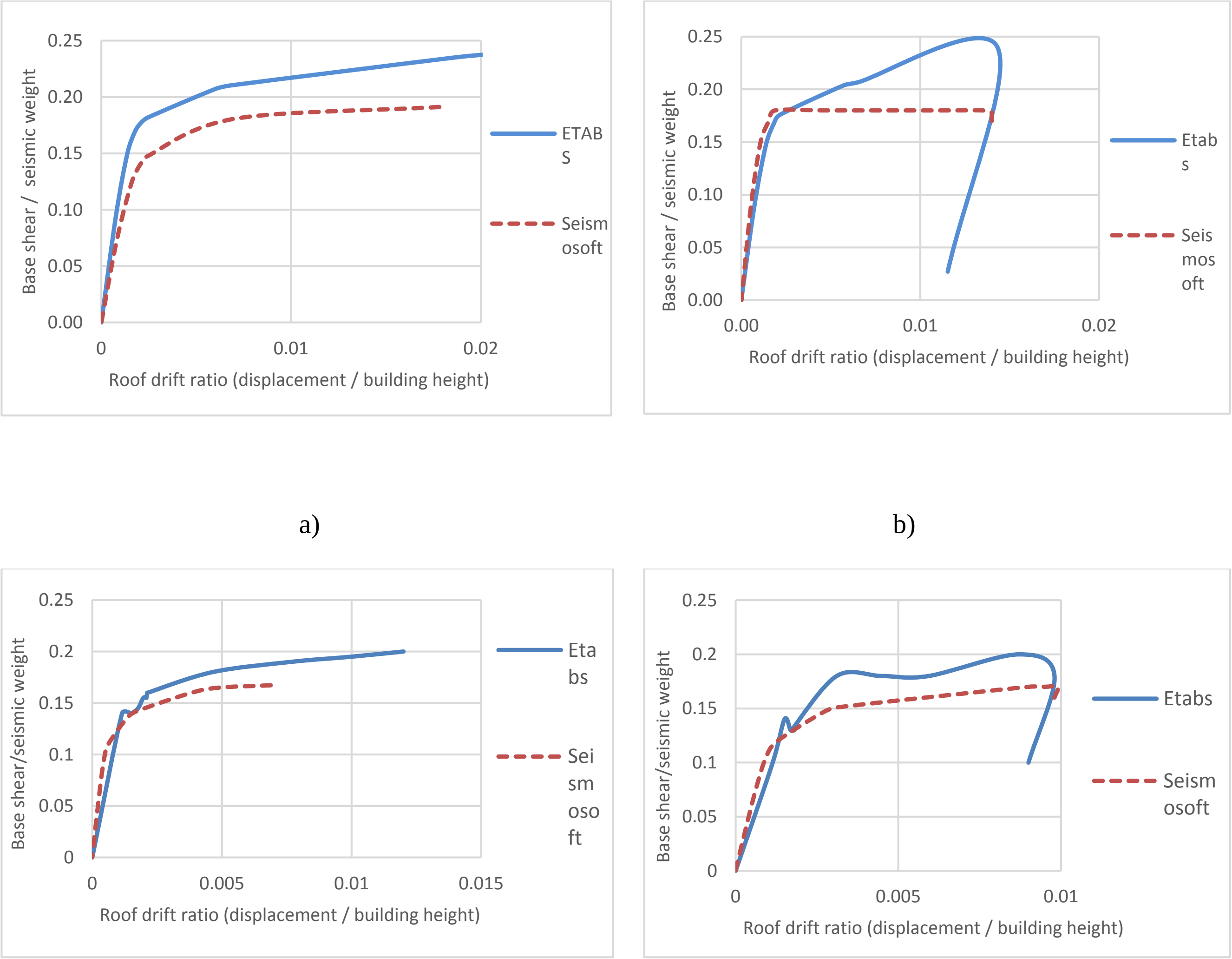
Figure 59. Capacity Curves of Ref 6- story frame: a) x-direction, b) y-direction

In *Figure* 59a-b are shown the capacity curves for the Ref 6 story frame in both x and y directions. Drift ratios at significant strength loss are 0.7% for x direction and 0.95% for y direction respectively.

4.3.3 Comparison of Analysis Results

4.3.3.1 Comparison between Etabs and Seismosoft

In the below graphs, *Figure 60a-d* are shown the normalized graphs of the Ref 3 and 6 story frames, analyzed with both software's ETABS and Seismosoft. Since in the study are not used any experimental data in order to calibrate the results two different programs are used to analyze the reference frames



a)

b)

c)

d)

Figure 60. 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

From the graphs it is seen that the programs have generated almost the same capacity curves for both reference frames, 3 and 6 story. So it can be said that the analyses is done in the right way since both programs show approximately the same performance and results. So the analyses of the other frames are done by using just ETABS software.

4.3.3.2 Comparison between different Irregularities

In the below graphs, *Figure 61-64*, 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 as shown in *Figures 61-64* below:

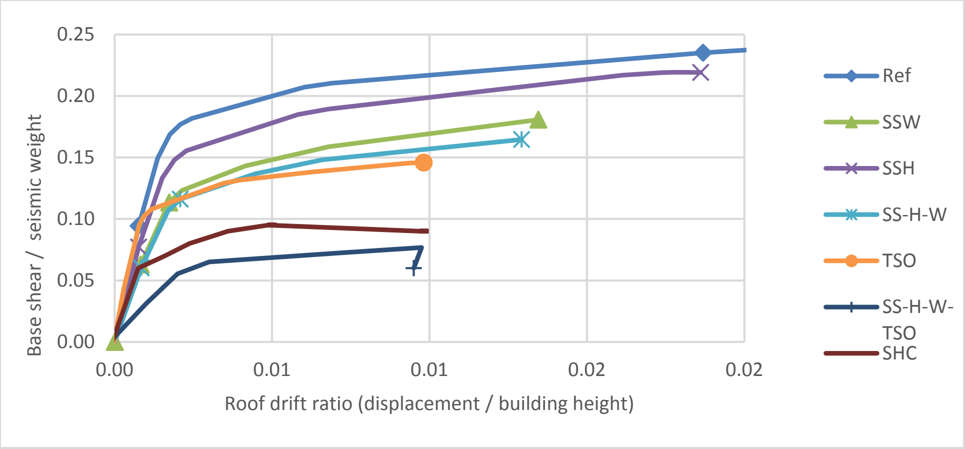


Figure 61. Capacity curves of 3-story frames, x-direction

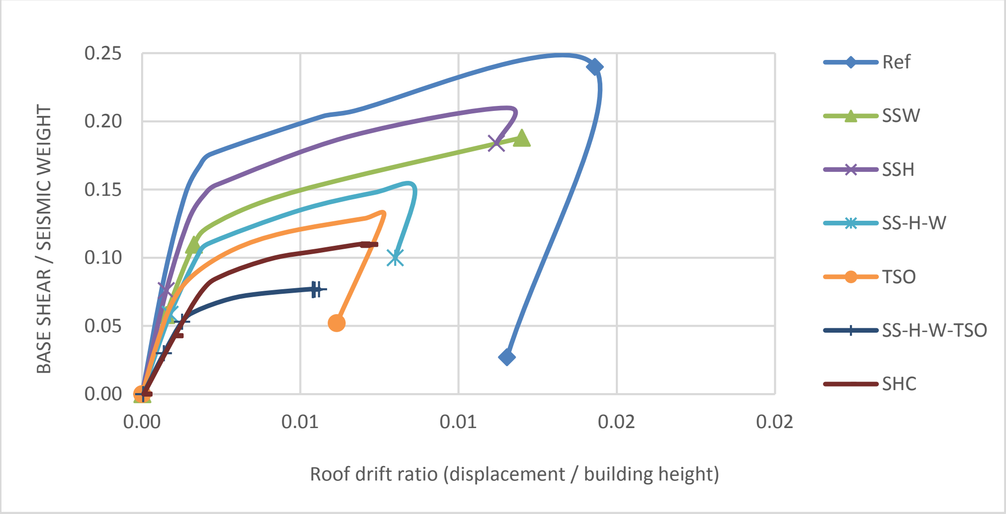


Figure 62. Capacity curves of 3-story frames, y-direction

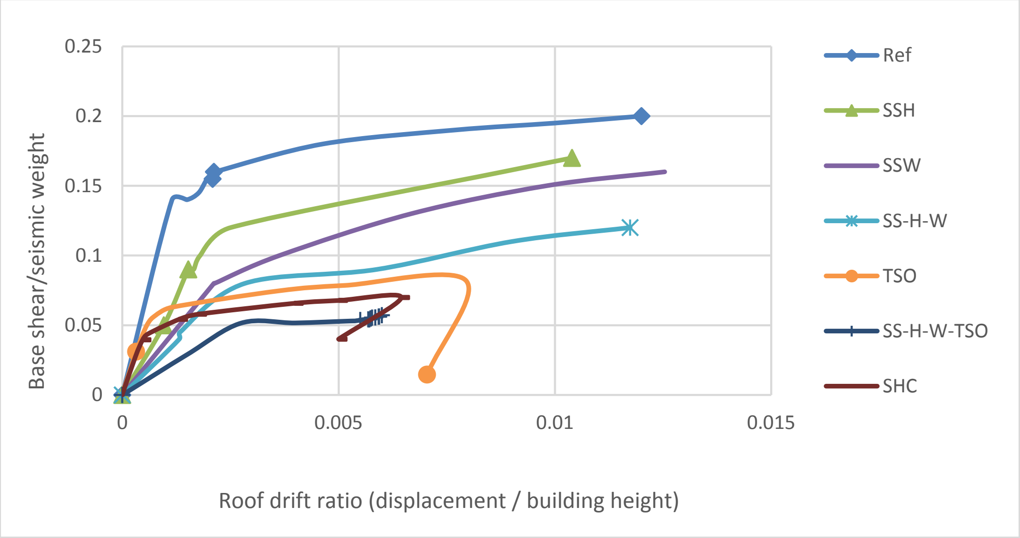


Figure 63. Capacity curves of 6-story frames, x-direction

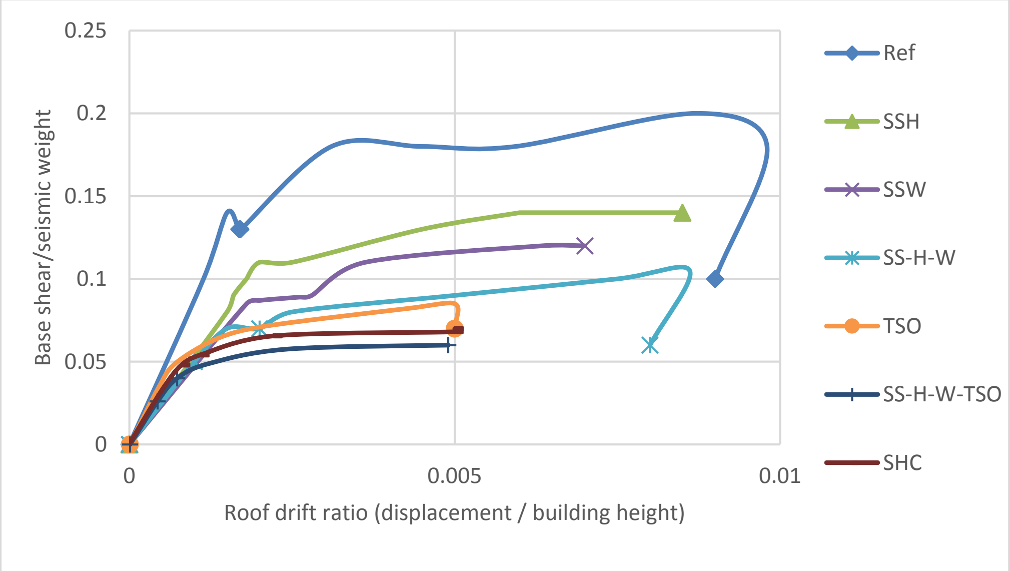


Figure 64. Capacity curves of 6- story frames, y-direction

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 stiffnes, 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 stiffnes, 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.

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, *Figure 61-64*. 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 direction respectively.In *Table 5* 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 similar to other studies [Inel *et. al.*, 2008].

Table 5. Comparison of stiffness and strength with Ref frames

	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%

In order to get the performance point of each of the considered frames, the capacity spectrum method is used in which the capacity curve is transformed in an equivalent capacity SDOF capacity curve. 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 and Seismic Albanian Codes (KTP). Parameters of the considered Response spectrum consist in:

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

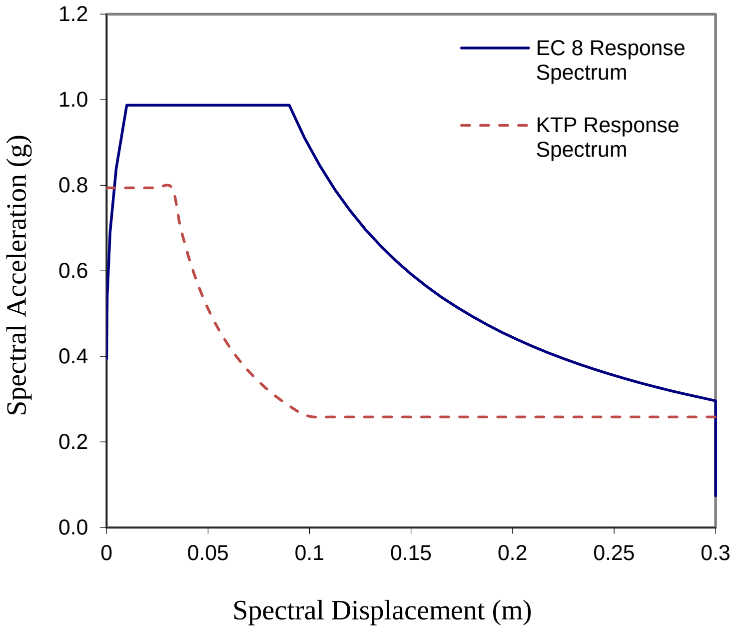


Figure 65. Considered Response Spectrums for EC 8 and KTP

In*Figure 65* it is seen that for the considered parameters two different response spectrums are generated based on EUROCODE 8 and KTP Codes. 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 6* [ATC 40, 1996], is considered in which are given the correspondences between selected response spectrums parameters and reduction factors.

Table 6. Seismic Value Correspondences [ATC 40, 1996].

Cases	Ca	Cv	Correspond to	A_0	S(T)
a	0.4	0.3		0.40	For soil type 1, Tb=0.3
b	0.4	0.4		0.40	For soil type 2, Tb=0.4
c	0.4	0.6		0.40	For soil type 3, Tb=0.6

*A_0- Seismic zone coefficient

In the study “c” case is considered with reduction factors Ca=0.4 and Cv=0.6 which correspond to soil type 3 and Tb=0.6.

In the below graphs *Figure 66-69* 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.

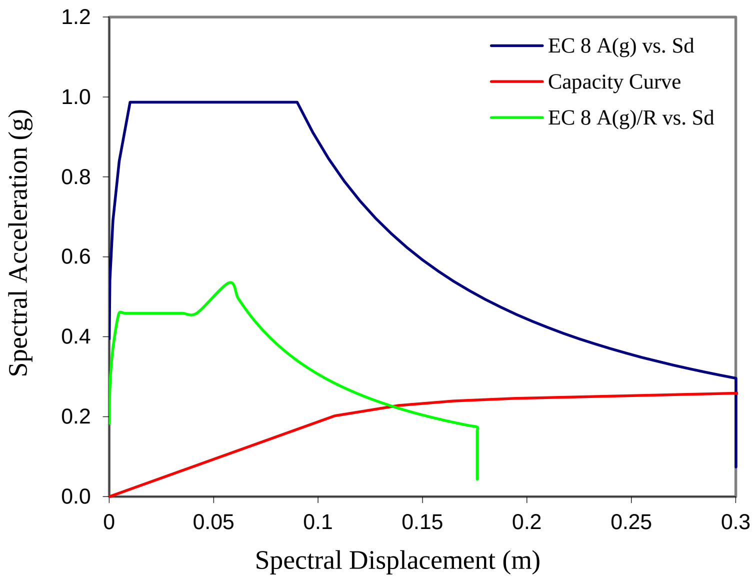


Figure 66. Performance point of Ref 3 story x-direction frame EC 8

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.

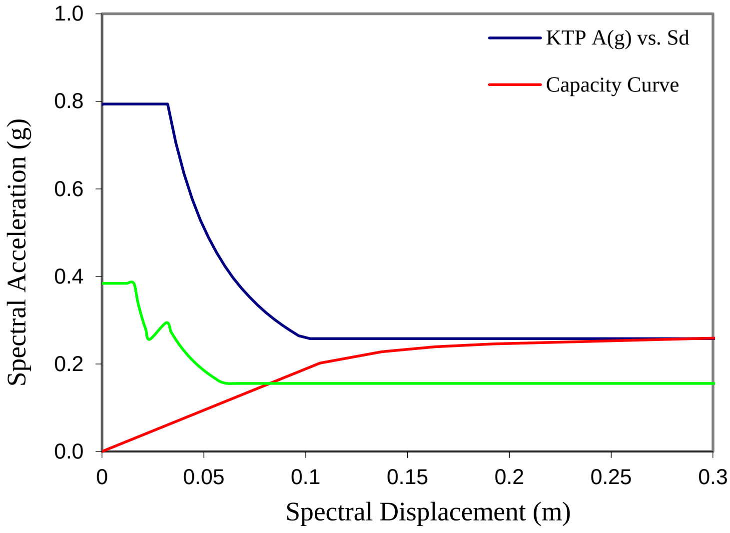


Figure 67. Performance point of Ref 3 story x-direction frame KTP

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.

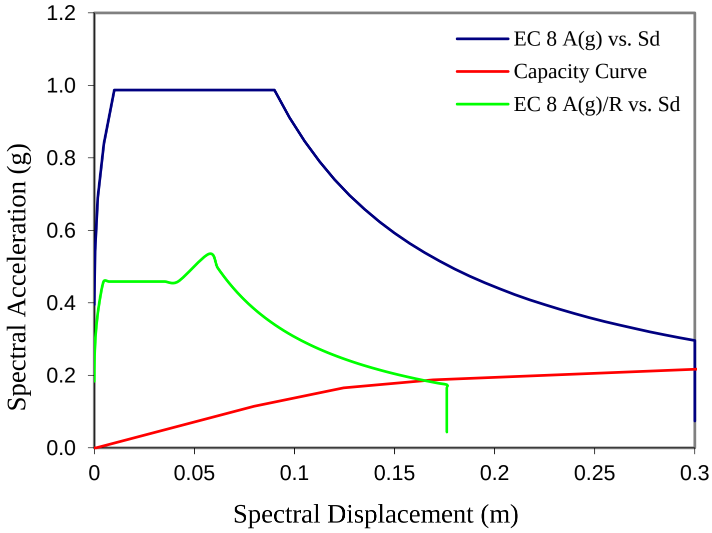


Figure 68. Performance point of Ref 6 story x-direction frame EC

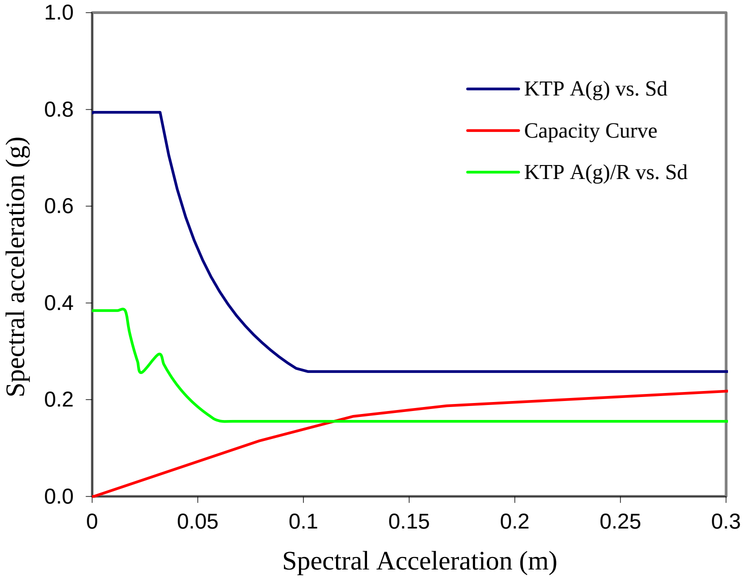


Figure 69. Performance point of Ref 6 story x-direction frame KTP

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.

In *Table 7* are shown the performance points for all considered cases in this study. From the results it is observed that the demands by considering response spectrum from EUROCODE 8 are higher than response spectrum generated from KTP codes, with higher spectral acceleration and displacement. For SHC and SS-H-W-TSO frames it is not obtained a performance point for both frames 3 and 6-story for x and y directions. The reasons of not achieving a performance point may be as a result of lower stiffness and strength values of these two frames or because of iteration-convergence- problem.

Table 7. Performance points of the considered Frames for both x and y directions

	Performance point of Frames	Spectral acceleration Sa (g)	Spectral displacement Sd (m)	Spectral acceleration Sa (g)	Spectral displacement Sd (m)
		EC 8		KTP	
3-Story frames in x direction	Ref	0.23	0.14	0.18	0.08
	TSO	0.18	0.17	0.18	0.25
	SSH	0.18	0.17	0.185	0.16
	SSW	0.21	0.13	0.18	0.07
	SS-H-W	0.18	0.18	0.17	0.17
	SS-H-W-TSO	**No intersection**		**No intersection**	
	SHC	**No intersection**		**No intersection**	
3-Story frames in ydirection	Ref	0.23	0.14	0.18	0.08
	TSO	0.17	0.17	0.17	0.2
	SSH	0.18	0.16	0.165	0.16
	SSW	0.20	0.15	0.19	0.1
	SS-H-W	0.175	0.17	0.16	0.16
	SS-H-W-TSO	**No intersection**		**No intersection**	
	SHC	**No intersection**		**No intersection**	
6-Story frames in x direction	Ref	0.2	0.165	0.18	0.12
	TSO	0.16	0.14	0.16	0.165
	SSH	0.18	0.17	0.165	0.14
	SSW	0.17	0.13	0.155	0.11
	SS-H-W	0.15	0.14	0.15	0.1
	SS-H-W-TSO	**No intersection**		**No intersection**	
	SHC	**No intersection**		**No intersection**	
6-Story frames in ydirection	Ref	0.19	0.175	0.175	0.175
	TSO	0.165	0.13	0.155	0.16
	SSH	0.17	0.16	0.16	0.15
	SSW	0.16	0.14	0.17	0.18
	SS-H-W	0.14	0.13	0.15	0.15
	SS-H-W-TSO	**No intersection**		**No intersection**	
	SHC	**No intersection**		**No intersection**	

CHAPTER 5

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 behavior of reinforced concrete components and masonry infill walls. In this study masonry infill walls were modeled as diagonal strut elements in accordance with FEMA-356 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 EC 8 and KTP codes. Calibration of results is done by using two different programs ETABS and Seismosoft to analyze 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

Average drift ratios obtained in this study shows that the demands of 3 story frames are higher than those of 6 story frames.

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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APPENDIX A

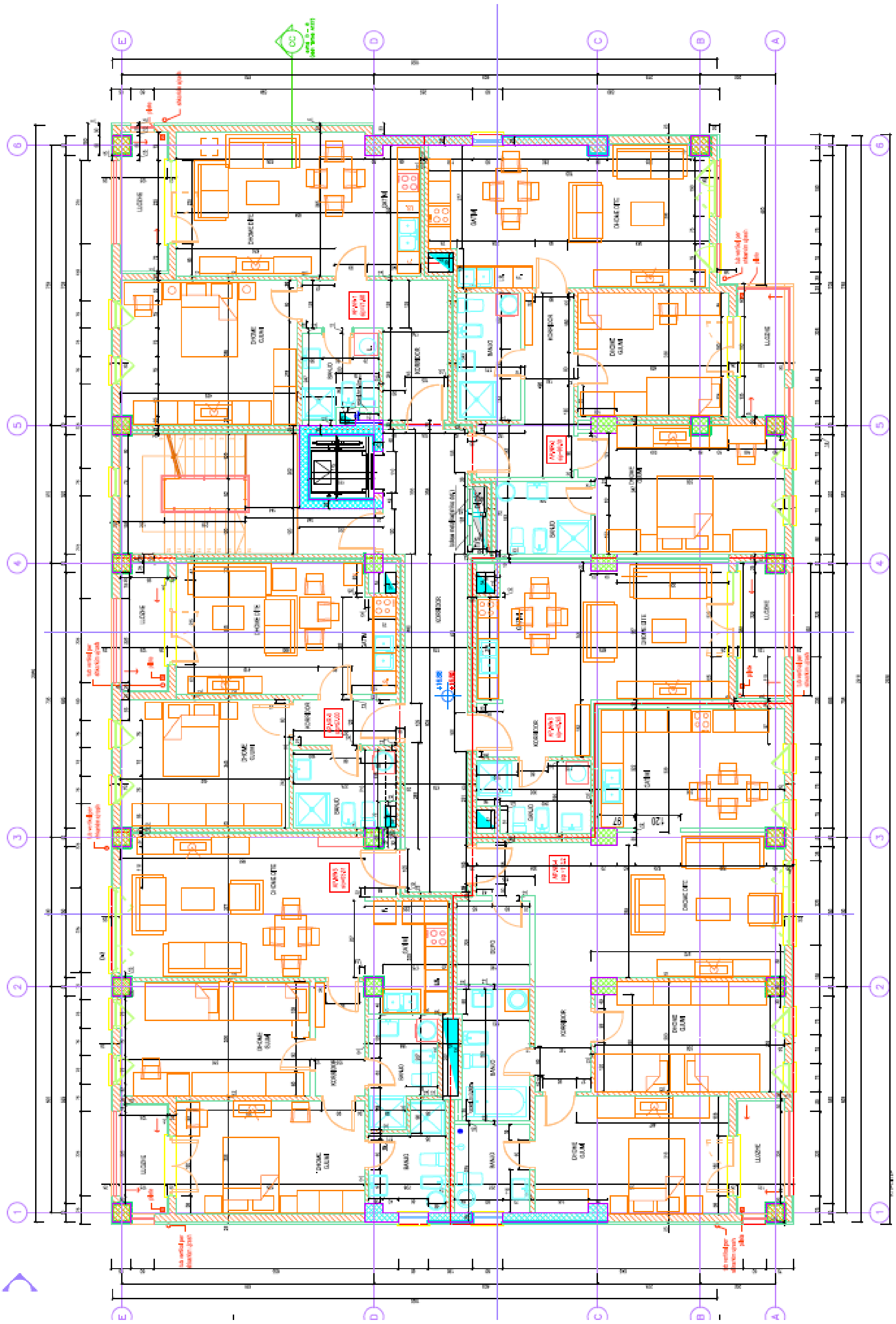

Figure 70. Plan view from Albanian 3 story case building