REVIEW OF THE STRUCTURAL IRREGULARITIES ON ALBANIAN RC BUILDINGS WITH A CASE STUDY

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Approval sheet of the Thesis

This is to certify that we have read this thesis entitled **"Review of structural irregularities on Albanian RC buildings with a case study"** 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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ABSTRACT

REVIEW OF STRUCTURAL IRREGULARITIES ON ALBANIAN RC BUILDINGS WITH A CASE STUDY

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Almost, all Albanian buildings are composed of masonry and reinforced concrete, and just a few of them, like industrial buildings are composed of steel. Unfortunately, the quality of building stock in this country is low in performance and not properly designed according to building codes. Considering that Albanian building code has not been updated since 1989, and many of these buildings are constructed without any structural engineering plan, there is a large presence of structural irregularities.

It is of great importance to focus on structural irregularities due to the hazards they cause. Irregularities vary from the less hazard one, those of damaging structural elements, till to those of causing total collapse of buildings. As Albania is an area prone to high seismic hazard with different magnitudes, it is a must to be aware and take actions while talking about structure irregularities.

Therefore, a typical residential R-C frame- building, 6 story height, was taken in evaluation through this study. This kind of building is mostly representative of latest -90 buildings typology due to their same construction method, period and quality. The structural design model consists of irregularities such as: soft story because of difference in height and soft story due to non-presence of masonry partition walls for open space commercial purposes, poor reinforcement details, structural additions represented as heavy overhangs, semi infilled frames and short column effects. The seismic performance assessment will be displayed by analytical method consisting of pushover analysis, which is an easy way to explain the non-linear response of building. The results provided by pushover demand, capacity spectrum and plastic hinges formation give us a real understanding of structural behaviour.

What observed from analysis shows that every structural irregularity in RC building, affect the structure by decreasing its performance level considering bearing stiffness and deformation capacity. When comparing the presence of irregularities some of the such as combination of soft story with height difference – absence of infill walls – short column due to semi infilled frames, - and overhangs, results in serios damages leading to total loss of stiffness in structure.

Keywords: *Structural irregularities, reinforced concrete frame buildings, soft story, short column effect, poor reinforcement detail, pushover analysis, etc…*

ABSTRAKT

PARREGULLSITE STRUKTURORE NE SHQIPERI TE NDERTESAVE RAMA BETON-ARME ME NJE RAST STUDIMI

Kokoshi, Besjana

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Udhëheqësi: .Assoc.Prof.Dr. Huseyin Bilgin

Pothuajse të gjitha ndërtesat ne shqiperi përbëhen nga muratura dhe betoni I përforcuar, dhe vetëm disa prej tyre, si ndërtesat industriale përbëhen nga çeliku. Për fat të keq, cilësia e stokut të ndërtimit në këtë vend është ne performance te ulët dhe konsiston ne mungese te projektimit te mirfillte sipas standardit ne fuqi. Duke pasur parasysh se kodi I projektimi shqiptar nuk është ndryshuar dhe permiresuar që nga viti 1989 dhe shumë prej këtyre ndërtesave janë ndërtuar pa ndonjë plan strukturor inxhinierik, ekziston një prani e madhe e parregullsive strukturore. Është me rëndësi të madhe të përqendrohemi në parregullsitë strukturore për shkak të rreziqeve që ato shkaktojnë. Parregullsitë ndryshojnë nga ato më pak te rrezikshme, si dëmtime lokale të elementeve strukturore, deri tek ato që shkaktojnë rënien totale të ndërtesave. Meqenëse Shqipëria është një zonë e prirur për rrezik të lartë sizmik me magnitudë të luhatshme, është e domosdoshme të behet ndergjegjesim dhe te ndërmerren masa kur flasim per parregullsitë e strukturës. Prandaj, në këtë studim është marrë një ndërtim tipik rezidencial B/A, me lartësi 6 kate. Ky lloj ndërtimi është kryesisht përfaqësues i tipologjisë së ndërtesave para viteve -90 për shkak të metodës së tyre të ndërtimit, periudhës dhe cilësisë. Modeli i dizajnit strukturor përbëhet nga parregullsi të tilla si: kate te bute për shkak të ndryshimit në lartësi nga katet e tjere, dhe gjithashtu për shkak të mungesës së mureve mbushes te cilet shmangen per te krijuar hapsire dhe shfrytezohen per aktivitete te ndryshme sociale dhe biznesi, ose dhe parkime. Gjithashtu keto parregullsi konsistojne ne detajimin e dobet te armimit te lementeve strukturore, shtesave ne strukturen fillestare te cilat prishin transmetimin e forcave ne menyre te rregullt, etj. Vlerësimi sizmik i performancës do të kryehet me metodën analitike të përbërë nga analiza pushover, e cila është një mënyrë e thjeshtë për të eksploruar sjelljen jolineare të ndërtesës. Rezultatet e nxjerra nga kapaciteti i mbingarkesës, spektri i kapacitetit dhe formimi i cerniereve plastike, na japin nje informacion kuptimplote mbi sjelljen strukturore. Ajo që vërehet nga analiza eshte se çdo parregullsi strukturore në ndërtesat B/A, ndikon negativisht duke ulur nivelin e performancës në aspektin e shtangesise dhe deformimeve qe peson. Kur krahasojmë praninë e parregullsive, të tilla si kombinimi i një kati të dobet me diference në lartësi - mungesa e mureve mbushes - kolona e shkurtër për shkak të ramave gjysmë të mbushura - dhe ballkonet e renda, rezulton me dëmtime serioze që çojnë në humbjen totale të shtangesise në strukturë.

Fjalët kyçe: *parregullsi strukturore, kate të dobta, efekti i kolonave te shkurtra, analiza jo lineare statike, pushover etj*

To Merjem

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Much thanks goes for my parents and husband, who have cared, loved, supported, motivated, encouraged and put up with me, every day of prepearing this thesis.

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CHAPTER 1

INTRODUCTION

1.1 Problem Statement

Nowadays, many buildings are seen to be irregular in configuration for elevation and plan. Unfortunately, during an earthquake scenario they would be subjected to partly collapse and muchover, total collapse mechanism. In case to prevent this, it is necessary to check the behaviour of existing and new structures to resist againist disasters.

This study aims to carry out an evaluation of structural vulnerability of a six story RC frame building, representing mid-rise buildings of Albanian building stock, which is subjected to different structural irregularities such as: heavy overhangs, soft story, short columns, ponding.

The seismic performance assessment by non-linear static pushover will determine the vulnerability of building imposed to irregularities.

1.2 Thesis Objective

This study consists on the following objectives:

- Identifying structural irregularities in RC frame buildings
- Identifying the types of irregularities in Albanian RC frame buildings.
- Identifying the seismic zone where they are built and the period of construction, considering the designing code they respond to.
- Determining the seismic capacity curves and non-linear behaviour of the representative building model under specific irregularities.
- Finally, considering the seismic response of building there will be provided an evaluation of structural seismic vulnerability.

1.3 Organization of the thesis

This thesis is divided in 6 chapters. The organization is done as follows:

Chapter1. Presents the problem statement, thesis objective and scope of works in this study.

Chapter 2. Presents the literature review.

Chapter 3. Includes the Analytical Model of case study RC frame Building.

Chapter 4. Presents the methodology followed for evaluation of RC frame building's case study.

Chapter 5. Presents the analysis results.

Chapter 6. Ends with the conclusions.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

All buildings are prone to destructive earthquake forces during earthquakes. Meanwhile, when some structures undergo to a completely collapse state under these forces, others may remain safe and steady with few damages. The structural damages caused by an earthquake, generally are seen as an engineering challenge which can be avoided since the design phase. [Ozmen and Unay, 2007]

During the last earthquakes worldwide most of the building collapse due to the presence of structural irregularities in frame construction. [Varadharajan, 2014] Structural irregularities are classified as: irregularities in plan and vertical direction. Irregularities in horizontal direction consist of: torsional irregularity, floor discontinuities, projections in plan, nonparallel structural member axes. Irregularities in vertical direction consist of weak storey and soft storey discontinuity of structural elements.

Even in Albania, the building stock is very familiar to structural irregularities, beginning since masonry typology, RC frame buildings, and till now that we have more complex structures and monolith system. The factors affecting structural irregularities are counted to be a lot but the most common ones to be mentioned are: poor quality design due to old non-updated codes, economical issues due to “saving” moto, lack of materials import, lack of civil engineering knowledge. After all these issues responding mainly to communism period, still some structural problems did not find solutions even in further years. Moreover, the existing building performed additions, intervention in structural system and many soft stories were created.

The capacity of original design is exceeded, the force distribution path has changed and people are still not aware of what would these kind of buildings perform in case of even moderate earthquakes.

Unfortunately, Albania is considered as a high seismic region and the widespread presence of buildings irregularities in RC buildings requires the need to update the level of knowledge about their effect on RC frame structures in lateral forces accuracy.

2.2 General

When designing a structure, since at the first steps, there is made a detailing and selection of building's members, so that their designing capacity resists the expected demand of different forces and displacements that may occur at the structure.

Telling the truth, no real structure is that perfectly regular, because there are many factor preventing from having a perfect one. Some of them have been designed irregular to meet the architectural design, such as: ground floors for commercial purposes created by eliminating central columns, reducing the dimensionsnof beams and columns in the upper storeys to fulfill functional requirements and other commercial purposes like storing mechanical equipments, etc.

Plan irregularity

- The building does not have an approximately symmetrical geometric configuration or has re-entrant corners with significant dimensions.
- There is the potential for large torsional moments because there is eccentricity between seismic resisting system and the mass tributary to each level.
- The diaphragm at any single level has significant changes in strength or stiffness.

Vertical irregularity

- The building does not have a symmetrical geometric configuration along the vertical axes or it has horizontal offsets with significant dimensions.
- The mass-stiffness ratios between adjacent stories varies significantly**.** [FEMA P-2012]

2.3. Past studies researches on irregularities

There are many reasons for the poor behavior that perform the irregular structures. In contrary to regular structure, in irregular ones the inelastic behavior can be affected by irregularities and can result in rapid failure of structural elements in these parts. [FEMA P-2012]

To produce safer buildings, the world wide designing codes provide limitations on the allowable degree of structure to have irregular configuration. These limitations are set in the designing codes for both types of irregularities. Such requirenments are intended to define the type of which analysis method to be used for structures. [Vinod K. Sadashiva, 2010]

The most common methods used in the evaluation of building's inelastic response are those based on nonlinear static pushover analysis.

In literatures e find that many researchers studied and tried to update the weaknesses that irregular buildings are responsible for. Mentioning **Fajfar and Fischinger** who proposed to use some invariant story forces in proportion with the deflected shape of the structure. [Peter Fajfar, 2000]. Or even **Eberhard and Sozen** who proposed to define load patterns based on mode shapes derived from secant stiffness at each load step. [Eberhard and Sozen, 2004] Similar to previous, **Park and Eom** came up with a new design method using secant stiffness.

Chintanapakdee and Chopra investigated how accurate is modal pushover analysis procedure for non- rregular frame stating that, the MPA was more reliable than FEMA-356 force distributions for all irregular frames. It is also stated that if enough modes are taken into account, MPA gives closer results to the time history analysis results while comparing with the other force distributions. [Chopra and Chintanapakdee, 2002]

Mwafy and Elnashai studied the accuracy of inelastic static pushover analysis in investigating the seismic response of RC buildings. They came up with the idea that, if the load pattern is carefully chosen, the model may define the inelastic response of the low and mid-rise buildings. Considering the high-rise buildings, they recommended using more load patterns because of the prolem to predict the maximum mode efects. [Mwafy and Elnashai, 2001]

Bayülke et. al. investigated on the earthquake damaged and undamaged RC buildings by non-linear pushover analysis method, in case to predict shear force displacement relations and to compare the limit lateral forces with the lateral load level as calculated from elastic acceleration spectrums for the analytically calculated R factor, concluding that: the buildings with symmetric shear walls in plan do not lose their lateral stiffness in a risky way like those without shear walls. [Bayulke et.al, 2003]

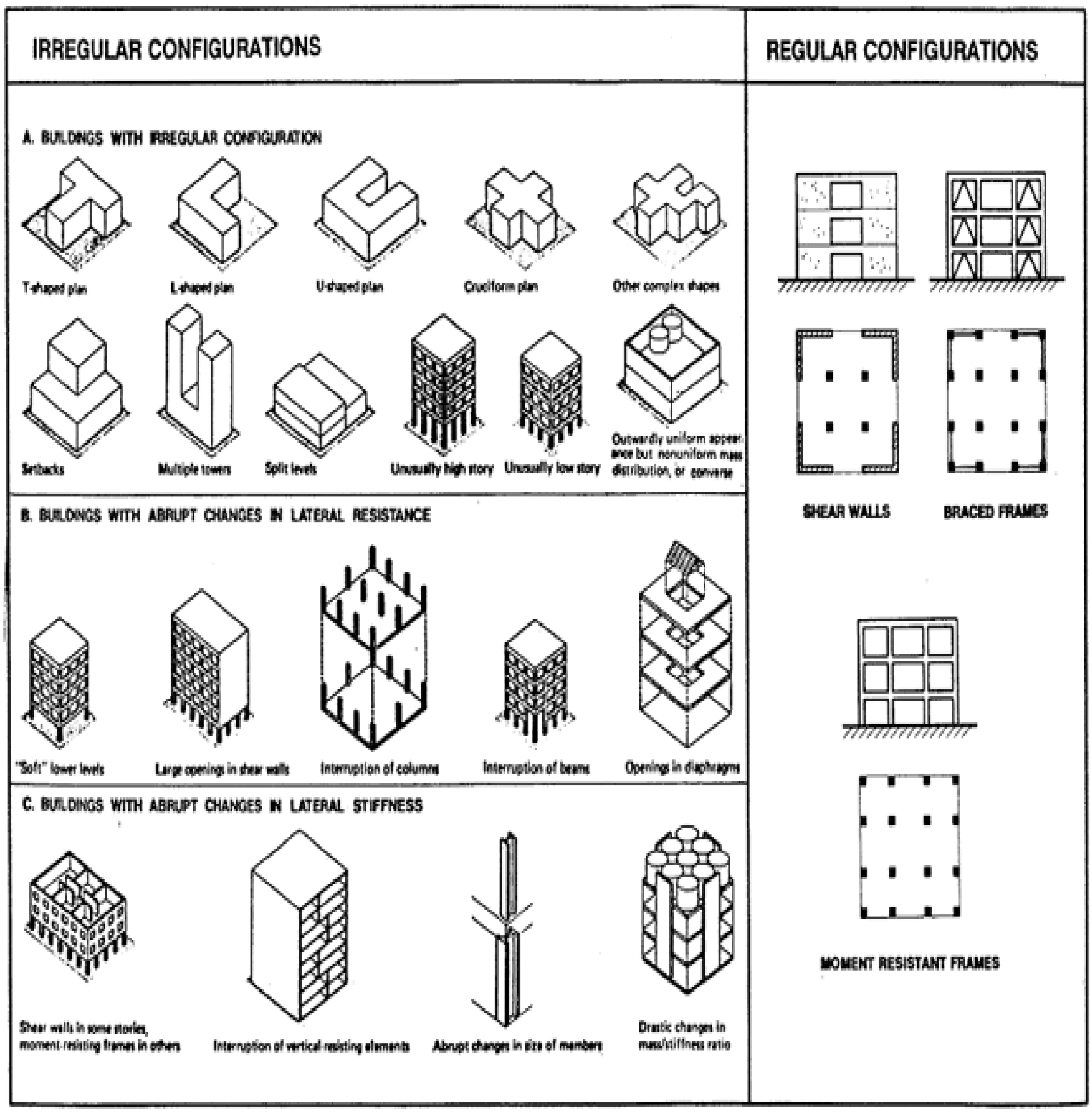


Figure 1. Different structural irregularities

2.4 Overview of Irregularities

The present chapter gives a review about the research studied held on the past for horizontal and vertical irregularities. The limits specified for these irregularities as defined by many codes of practice (IS1893:2002, EC8:2004 etc.) have been discussed. To better understand the effect of structural irregularities in buildings, academics have studied both vertical and horizontal irregularities with many different analytical methods, mentioning: nonlinear static pushover analysis, time history analysis and dynamic analyses.

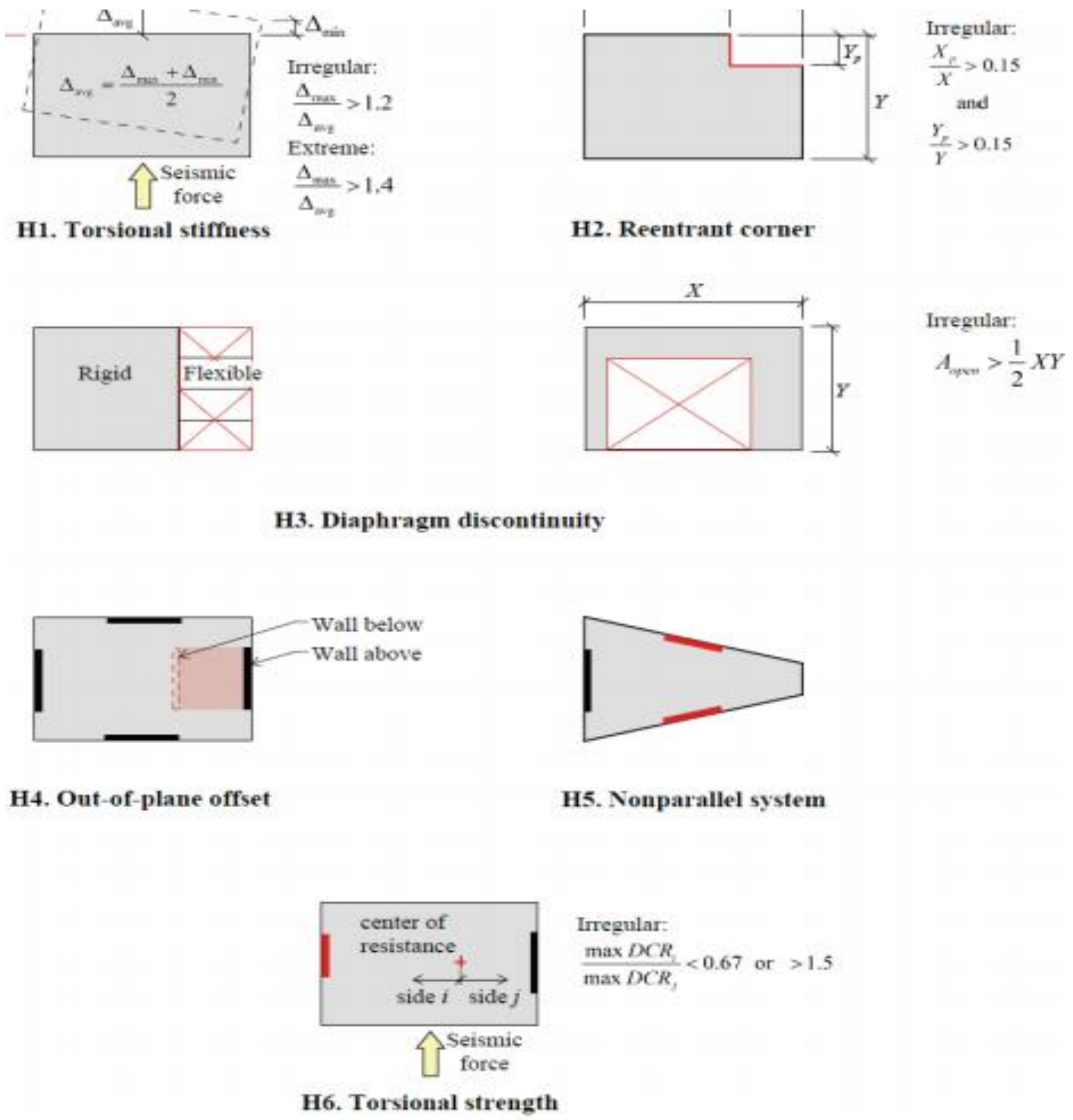


Figure 2. Horizontal irregularities. [FEMA_P 2012]

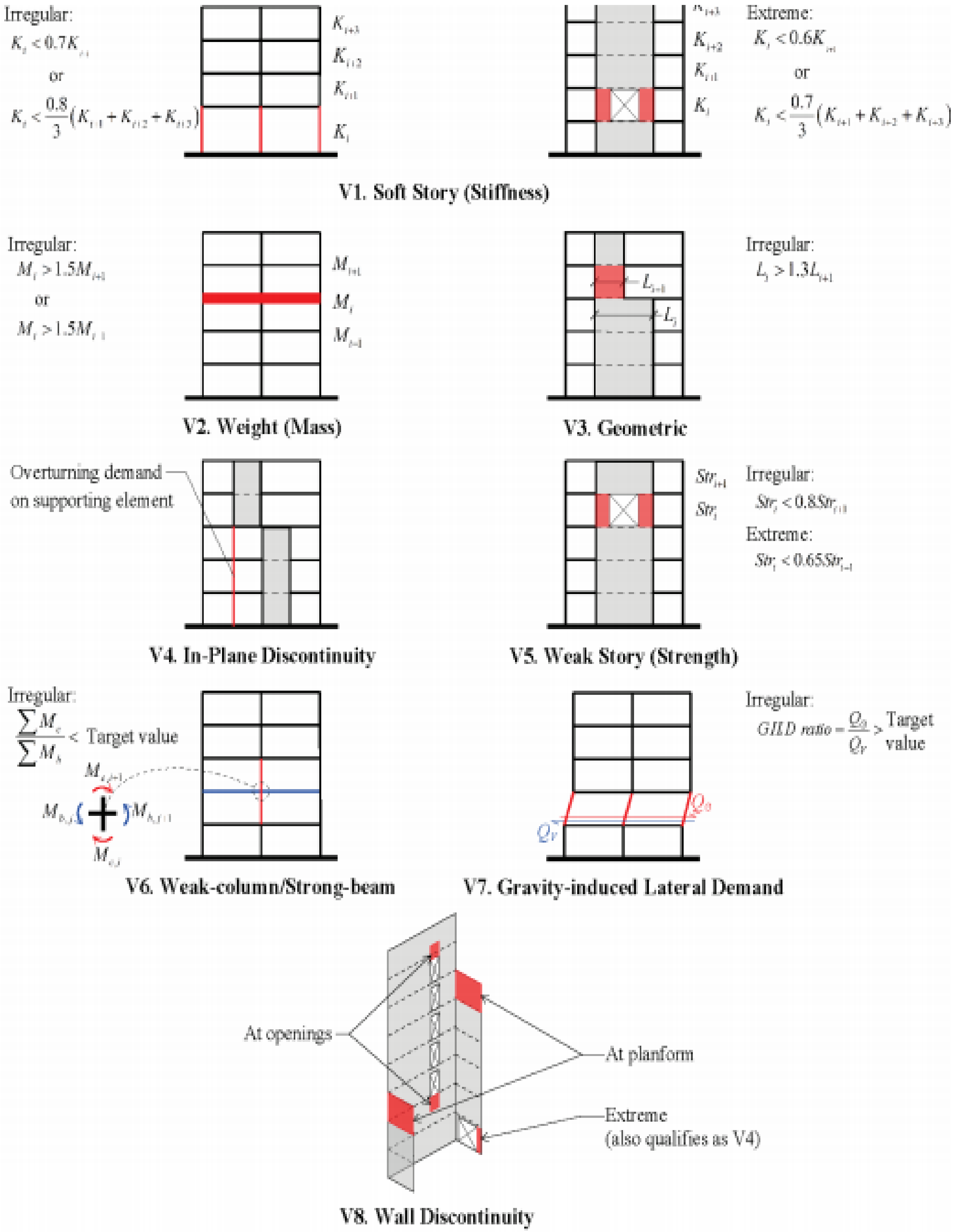


Figure 3. Vertical Irregularities [FEMA_P 2012]

2.4.1 Torsional irregularity.

A horizontal torsional irregularity is a function of the distribution of a structure's stiffness and mass at a given floor level. Most of the codes such as: IBC06 (2006), UBC97 (1997) and ASCE7-10 (2010), have similar limits for torsional irregularity. According to ASCE/SEI 7-16 regulations pertaining to torsional (stiffness) irregularities are triggered if the deflection on one side of the story is greater than 1.2 times the average deflection of that story along the same axis where a load with 5% eccentricity from the story's centre of mass is applied (the 5% eccentricity measured with respect to a building's dimension perpendicular to the direction of applied load).

Torsional irregularity exists if, dmax/davg> 1.2

Extreme torsional irregularity exists, if dmax/davg > 1.4

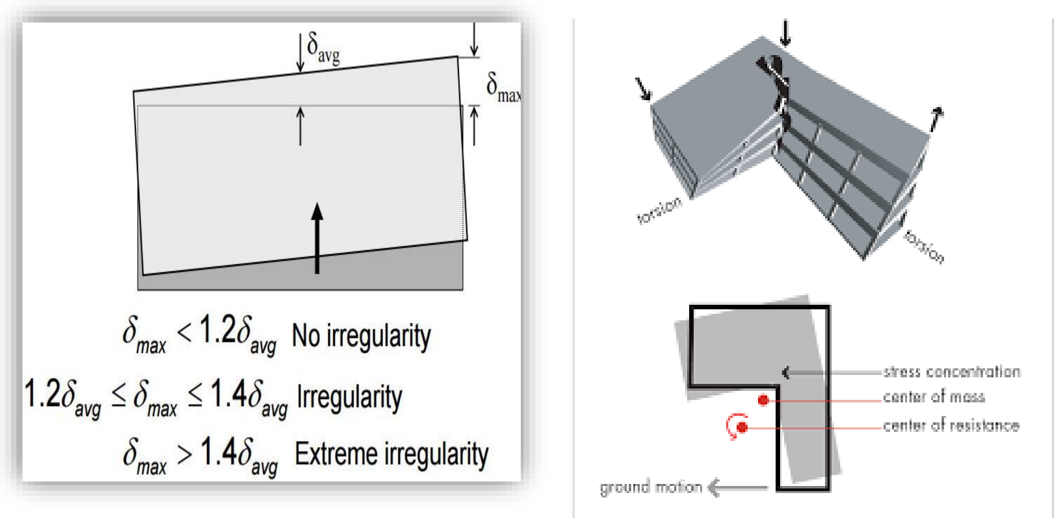


Figure 4. Torsional irregularity limits. (ijsetr.com)

Penelis and Kappos (2005), in their studies, intended to model the inelastic torsional effect of structures in pushover analysis. They applied spectral load vectors on a 3D model, defined from dynamic elastic spectral analysis and the response quantities were obtained via an equivalent SDOF system, which presented both translational and torsional modes. This step was checked for two single-storey and two multi-storey mono-symmetric buildings. In the case of two single-storey, the drift from the mean response resulting from nonlinear dynamic analysis was around 10%, while in the case of multi-storey buildings the difference in the response was about 20%, which was also considered acceptable.

Fajfar et al. (2005), proposed to combine the results provided by pushover analysis of a 3D structural model, considering the N2 method, which gives results for the target displacements and deformations along the height of a building, while the linear dynamic analysis is used to define the torsional effects of horizontal displacements. Using linear dynamic analysis resulted that, at the flexible edge, the elastic envelope of lateral drift is conservative comparing with the inelastic ones [Peruš and Fajfar, 2005]

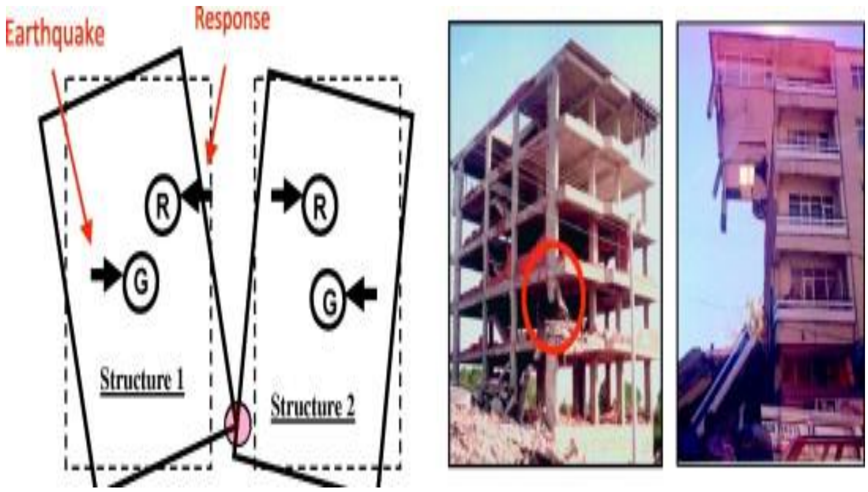


Figure 5. Effects of torsional irregularity to structures during earthquakes. [H.Gokdemir 2013]

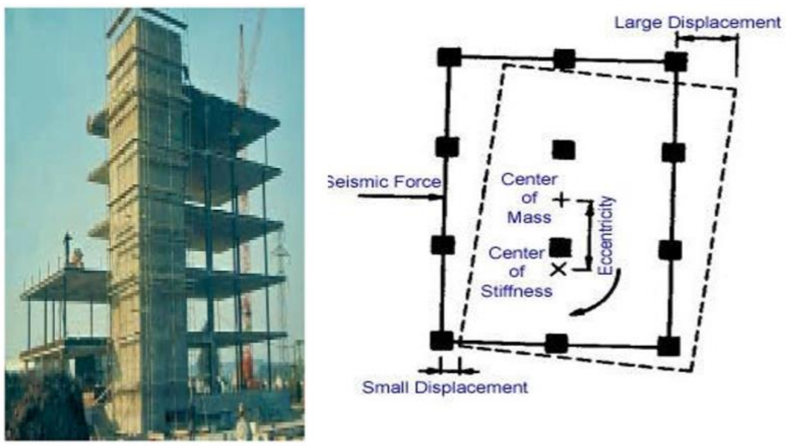


Figure 6. RC frame building prone to torsional effect. (Switzerland, 1994)

In this new RC building, the only bracing against lateral forces and displacements is a RC elevator core, asymmetrically constructed at the corner of the building, leading so to cause a large eccentricity between the centres of mass and stiffness (Switzerland 1994)

Figure 7. Partial collapse of a 6-story commercial building (Chou Ward) with a torsional irregularity. [Photo courtesy of Charlie Kircher]

The torsional irregularity caused the RC building to rotate during the earthquake. During the rotation, there was caused a large displacement of the gravity frames at the corner away from the stiffer walls which exceeded frame capacity causing failure of columns and collapse of the above floors. (Fema_P, 2012)

How to reduce torsional effect?

The most common way to minimize torsional effects is to select floor plans that are regular and not compleced. The response of buildings during earthquakes will be satisfactory if ceratin measures are taken to perform a resonable failure mechanism. Even constructing a slender RC wall, extending the whole height of the building at each elevation in the opposite corner from the shaft would be the best choice.[32]

2.4.2 Soft story irregularity.

Traditionally, aesthetics and functionality have dictated higher first storeys, allowing thus for ample car storage or big reception halls, invariably locating the residences or commerce on the upper floors. This configuration is both visually-pleasing and practical, however, it may lead to excessive mass on the higher floors and lower stiffnessess and strengths on the first floor, leading to a lateral discontinuity in the ductility demand when subjected to strong shaking [González et al., 2010]

The sws condition is also latent in reinforced concrete (rc) frame buildings, whose division walls are structurally joined (by design or as a result of a faulty construction process), adding lateral stiffness and limiting drifts which were not properly accounted for in the building's design. [Díaz, 2008]

According to ASCE/SEI 7-16, soft story irregularities occur where a given story has a stiffness less than 70% of the story above or less than 80% of the average stiffness of the 3 stories above. In **Michalis et al. (2006),** it was shown through analytical models that soft story irregularities have the largest effect on collapse capacity where the soft story occurs at one of the first three stories. Where the soft story is more towards the middle or top of the structure, the study showed that it has little effect on collapse capacity.

Ruiz & Diederich (1989) investigated the seismic performance of sws with brittle and ductile infill walls, monitoring the ductility demand at the first storey. They varied the ratios of elastic lateral stiffnesses and strengths and subjected the buildings to the ew component of the ground motion recorded by the accelerograph at the Ministry of Communication and Transportation of the 1985 Michoacán seism. They found that ductility demands are very sensitive to the ratio of the dominant periods of excitation and response, which is closely related to the occurrence of plastic hinges and the yielding or fracture of the infill walls, commenting that large uncertainties tied to the modelling of nonlinear dynamic response systems at their time made it difficult to derive simple rules which delimit safe design strength or stiffness ratios.

Dadi & Agarwal (2015) These authors attempted to update the nonlinear modelling of soft storey rc frame buildings for performance-based design. They conducted cyclic test on a 1/4 scale prototype soft storey frame building while analytically modelling the failure modes as per ASCE7 and Indian codes' ULS. In this way, they updated the analytical model at three stages of behaviour i.e., linear, nonlinear and failure. They concluded that the nonlinear properties of the reinforcement used in beam components of the frames have a significant influence on the global failure pattern, particularly assuring a flexural failure mode, which is responsible for the ductile response of the system.

In the EUROCODE-8, it is stated that the soft story mechanisms must be prevented, in case not to exeed the local ductility demands in the soft story columns. It is required the following provision in all of the main beams and columns: $\sum MRc > 1.3 \sum MRb$

Where, ∑MRc is the sum of the columns connected to the considered joint and ∑MRb is the sum of the design moments of the beams connected to the same joint. Moreover, when calculating MRc the minimum column moment values within the range of column axial forces produced by seismic design procedure should be considered.[33]

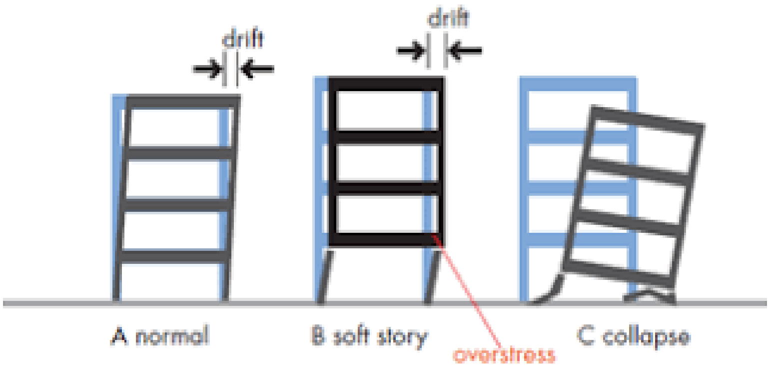


Figure 8. Ground soft story mechanism.

Figure 9. Soft story failure in an Izmit apartament building. Figure . Reinforced concrete framed structure, masonry infill with soft story failure at the bottom level. [Pathmanathan Brabhaharan, 2012]

Apart from the soft ground stories, even an upper storey can result in being weak when comparing to others if the lateral bracings are skipped or consumed, even if the horizontal stiffness is reduced above a specific floor, it may result in a sway mechanism.

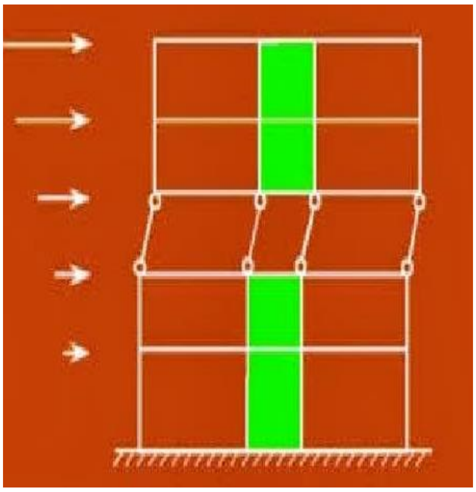
 

Figure 10. Intermediate soft story scheme. Figure 8. Progressive Collapse of Concrete Frames with Unreinforced Masonry Infill Walls Considering In-Plane/Out-of-Plane. [Khalid M. Mosalam and Selim Günay, *2015]*

Figure 11. Collapse of intermediate story in a 6 storey RC frame commercial building in Bhuj. Figure.12. Mid-story collapse, Kobe earthquake.

How to handle soft story effects?

- In order to satisfy the standard in preventing soft storey effect, the lateral stiffness of the first storey should exceed 70% of second storey stiffness, and 80% of average stiffness of 3 storeys above.

- Even steel bracings or concrete walls at the first storey, improve the design wished to achieve.
- Constructing RC walls in all buildings corners with required dimensions and thickness helps avoiding the soft storey effect.[34]

Figure 13 . Retrofitting strategies of soft stories.

2.4.3 Weight irregularity.

Weight irregularity takes place if the mass of one story is greater than 150 %- 200% of that of an adjacent story. Mass irregularity affects the response of the structure by increasing the weight of some floors depending to the others. Weight irregularity is depended on the structural model used, location of irregularity and analysis method.

Michalis et al. (2006) studied the effects of mass/weight irregularity, where he resulted that mass irregularity had a similar but inverse effect to that of a soft story irregularity. Where the mass irregularity occurred on the top floor, it had the greatest effect; where it occurred on the middle or lor lower floors, it had little effect on collapse capacity.

Other studies **[Magliulo et al., 2002; Tremblay and Poncet, 2005]** have also noted that vertical mass eccentric buildings perform well in seismic events.

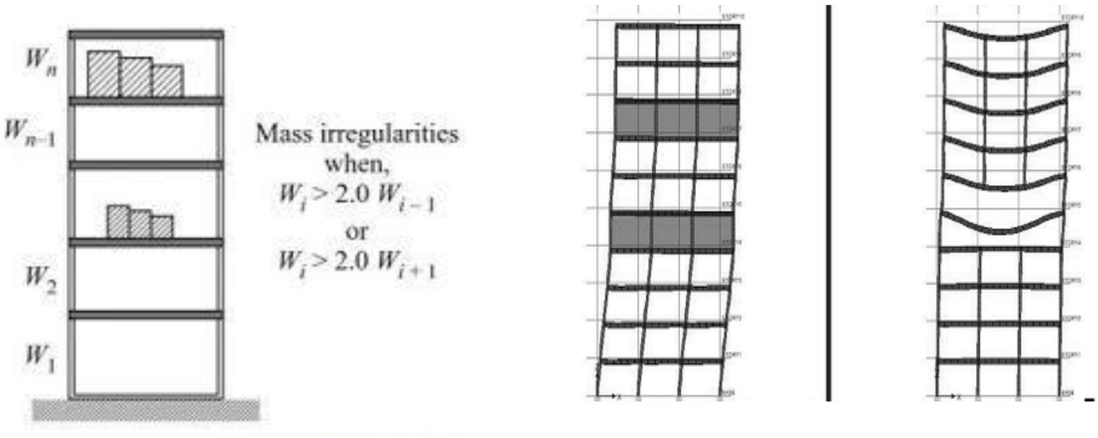


Figure 14. Mass irregularity scheme and limits in RC frame structures.

Figure 15. Brand new shopping centre in Mexico collapses under weight of heavy roof garden. [News Locker, 2018]

2.4.4 Weak story irregularity.

A weak storey is specified to be one story whose horizontal strength is less than 80% of that in the upper storey.

In a study done by **Michalis et al. (2006)** it resulted that weak story irregularities have much larger effect on collapse capacity than mass or soft story irregularities. The effect seems to be the greatest where the weak story is near the bottom of the structure, causing the largest decrease in collapse capacity. However, there is a benefit to structural performance at levels of ground motion farther away from collapse. A fuse effect is created and the damage is concentrated in the weakened story, protecting the upper stories from damage. . [Guevara Perez 2012]

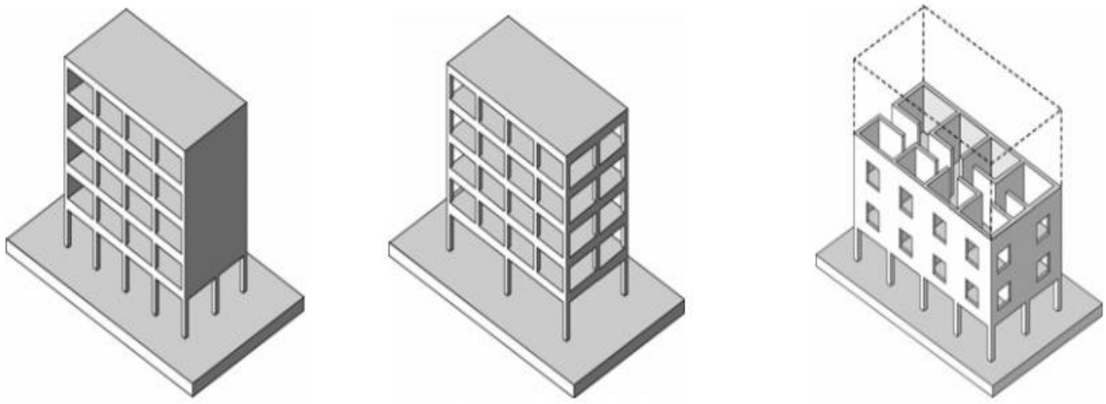


Figure 16. Examples of weak first story irregularity. (L. Teresa Guevara-Perez 2012)

Figure 17. First floor collapse of the main buildings of the Olive View Hospital [by L. Hashizume, E. Loh]

The structural system has significant discontinuities. While the upper four stories show presence of shear walls combined with moment-resisting space frames, the lower 2 stories had only a moment-resisting space frame system. The plan and reinforcement of structural members differed from one story to another. [L. Hashizume, E. Loh, .2002]

2.4.5 Vertical geometric irregularity.

Vertical geometry irregularities occur if the horizontal parameters of the seismic-force-resisting system on any story is greater than 130% of adjacent story. A common example of vertical geometry irregularity is story setback. A 2004 study by **Tena-Colunga** and **De Stefano and Pintucchi, 2008** found that buildings with vertical setback performed well as long as there was adequate redundancy in the lateral system, but systems without redundancy and with vertical setback performed poorly.

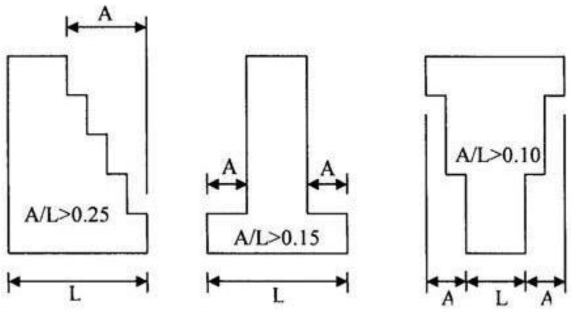


Figure 18**.** Vertical geometric irregularity as per IS 1893:2002 from publication: Effect of Vertical Irregularity on Performance of Reinforced Concrete Framed Buildings.


Figure 19. Stepped RC building with masonry infill. [Yukta Bilas Marhatta, Nepal].

This kind of irregularity shown above disturbs the direct path of load transfer due to the weakness they cause for the bracing elements ductility. Also, they influence at columns and slabs causing displacements. The seismic codes discourage all type of discontinues, because seismic behaviour of structural system having irregular configuration or asymmetrical distribution generally are larger than those of the regular ones.[35]

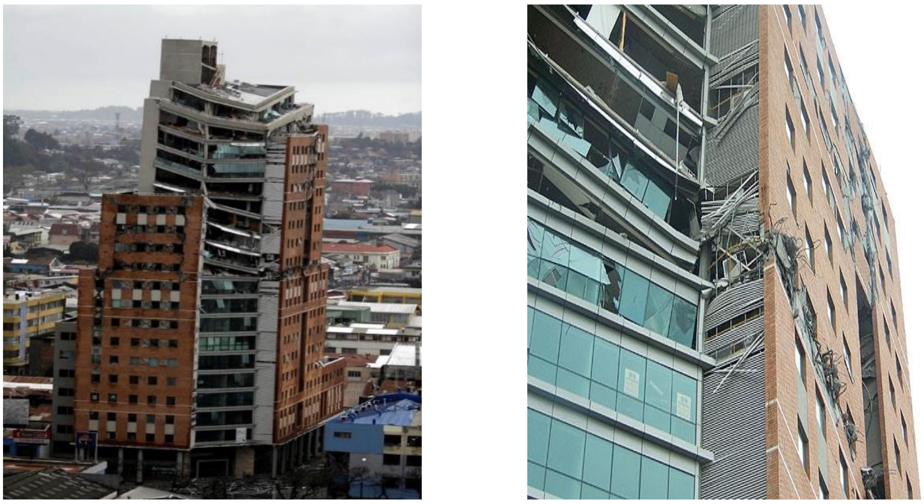


Figure 20. Torre O'Hings high rised building., built in 2007 and consisiting in many structural setbacks along the building height. The earthquake damage included partial collapse at three levels above mid-height.

The designing solution.

- Provide a continuous load path for transfering all loads from their origin point to the shear force resisting elements.
- The transfer of edge loads together with other loads in the diaphragm should be in propotion with shear and tension capacity of the diaphragm.
- In cases when there is presence of offset walls in the wall line, portions of the shear wall on each side of the offset shall be considered as separate shear walls.[36]

2.4.6 Story mechanism: Weak-Column/Strong-Beam.

The Column of a building is important in avoiding complete collapse of the building during an earthquake. Even under deformation, the column must be capable of taking the gravity load of the building along with the occupants in it. To avoid the failure in column elements, codes recommends that a building should follow 'Strong Column and Weak Beam' (SCWB) design. According to SCWB design philosophy, the flexural capacity of Beam (Mb) should be strictly less than that of the Column element (Mc) at any joint. In this way, 'Plastic Hinges' form in beam elements rather than the Column elements of the structure during an earthquake structure during an earthquake. [Swapnil Nayan, 2018]

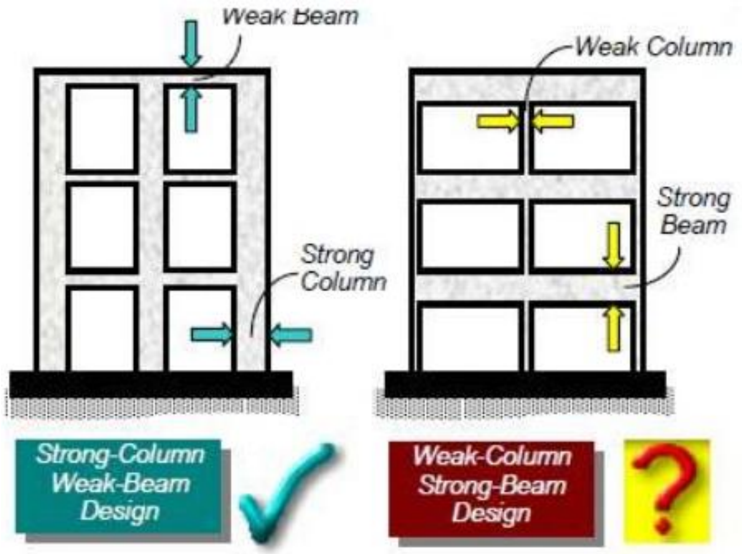


Figure 21. Strong beam- weak column Principle in RC frame structures.(taxonomy.openquake.org)

The formation of plastic hinges dissipates the energy of the earthquake and allows the building to undergo inelastic deformation. This allows sufficient time for the occupants to vacate the building in case of an earthquake. Moreover, it is desirable that the plastic hinges form in the top storeys of the structure before the bottom Storey as the gravity loads is lesser on the top storey. Hence by formation of the plastic hinges in the beam, the structure can take larger deformation without collapse**.** [Pradeep Kumar Ramancharla, 2018]

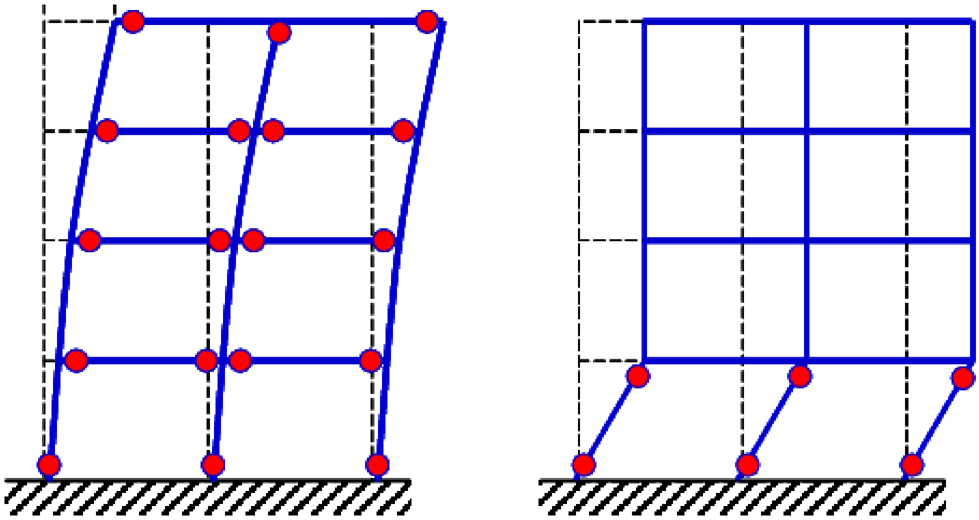


Fig. 22 Failure mechanisms of structures with " strong column-weak beam " design. (Zhe Qu, 2011)

When the failure of a beam happens, it only causes localized damages. But when a column fails, it can affect whole structure stability. That's why it is required to have the beams ductile weak links than having the columns. Sum of the moment capacities of the columns for the design axial loads at a beam column joint must be greater than moment capacities of the beam. ***Σ Mcolumns > 1.2 Σ Mbeams.***

The shear reinforcement should be the proper one to make sure that the strength in shear is greater then the strength in flexure, so preventing a non-ductile shear failure. In case of dissipating proper enough energy within the plastic hinges, their locations should have the required reinforcement details in order to provide with energy absorption capacity and ductility behaviour.

Previous work studying the effects of a weak-column/strong-beam on building performance has shown that the ratio of column to beam strength has a large impact on the collapse safety of the building, and that the impact differs for various height buildings [Haselton et al., 2011].

Figure 23. Corner beam-column joint failure during the Abruzzo earthquake [ReLUIS, 2009]

Figure 24.Damaged parking structure Whittier Narrows (Los Angeles) earthquake, 1987 The deep spandrels create a strong-beam, weak column condition.

Figure 25. Collapsed building due to strong beams and weak columns. from publication: Observed Seismic Behavior of Buildings in Northern Pakistan During the 2005 Kashmir earthquake.

Measures for preventing strong beam-weak column building failure.

- Column has to be designed stronger than beam so that, during strong earthquake it will provide stability and strength for the upper stories.
- The development of plastic hinge should be located at the end of beam, so the energy dissipation occurs in there.
- The plastic hinge region should be properly reinforcement detailed in order to improve ductility and inelasticity.
- (a) Decreasing the size of continuous openings. (b) Separating walls from structure.[37]

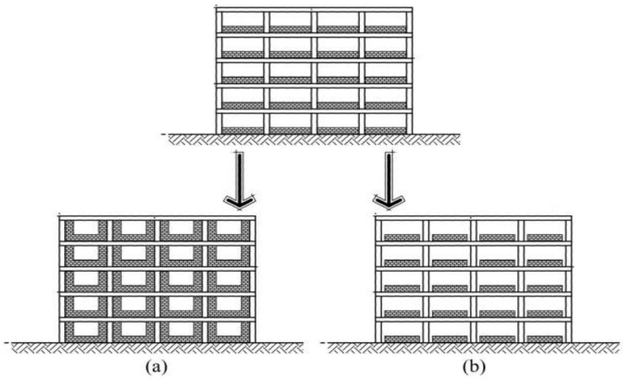


Figure 26. Effective measures to prevent strong beam-weak column: (a) Decreasing the size of continuous openings, (b) separating walls from structure

- By restricting the column-beam factor, RC frame structures could achieve the “strong column-weak beam” failure mechanism under the excitation of strong motions.
- Repair of strong beam-weak column with FRP is useful for rehabilitation of structural members. They are light, flexible, easy to apply and durable. [Abdullah Abang Ali, 2006]

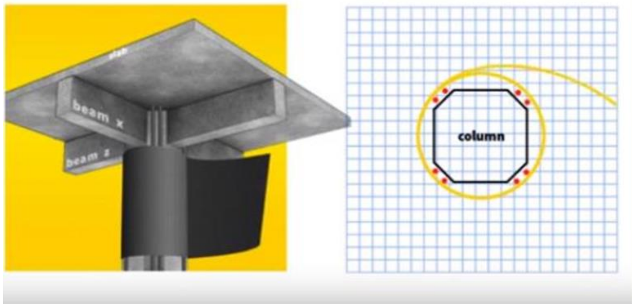


Figure 27. FRP repairing illustration. [Abdullah Abang Ali, 2006]

2.4.7 Short Columns Effect.

Another factor that causes considerable structural damages are the presence of short columns. It has been observed that the RC frame buildings which have columns different in heights in one storey, experience greater damage in the shorter columns than the taller ones in the same storey. [C.V.R. Murty]

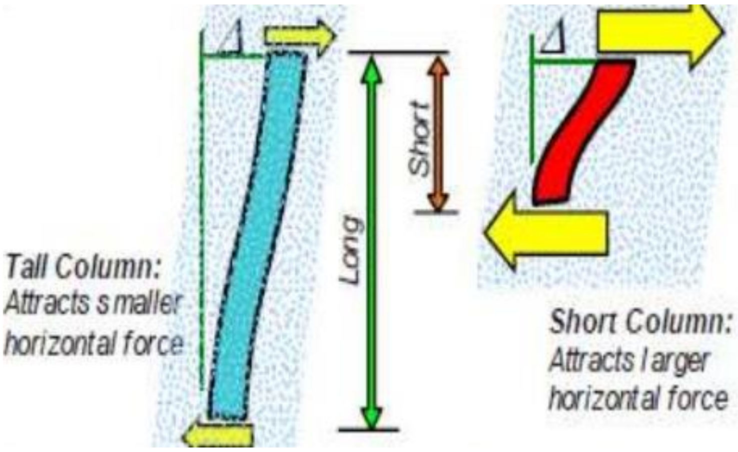


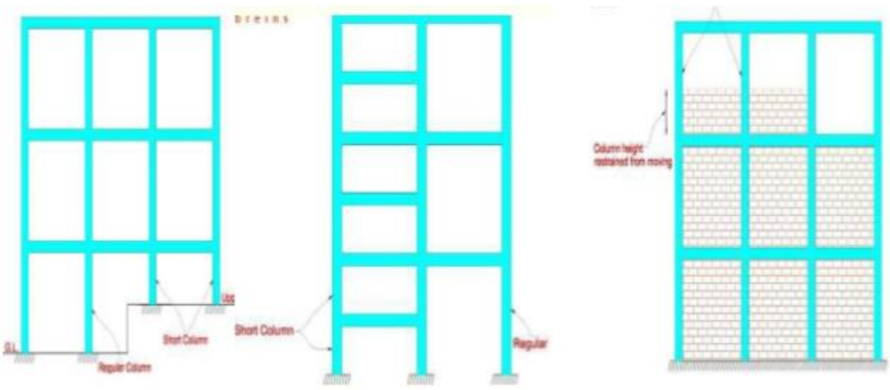
Figure 28. Short column effect. [C.V.R. Murty]

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 needed to deform it. But if the shorter column is not correctly designed to resist such forces it will pass under great damages during earthquake. This behaviour is known as short column effect**.** [Vahidi, 2009].

The definition of "short column" is referred due to these reasons:

1. Columns which are constructed shorter than other columns in one story.
2. Columns that are aimly constructed shorter by adding any partial walls on both or one side of the columns.
3. Short column effect may be also created in cases when a columns support mid-floor are added between two floors.

Typical examples of short column formation.



(a) (b) (c)

Fig 29 (a). Formation of short column due to different ground level.

(b) Formation of short column due to intermediate staircase landing beams in between two floors.

(c) Formation of short column due to partial height of brick masonry infill walls.

Buildings damaged due to short column effect.

Figure. 30 RC frame building under short column effect. (Vahidi 2009) Figure 31. Examples of short-column failure due to the presence of (A) staircase landing and infill, (B) staircase landing beam, and (C) infill panels with openings. [H.Varum 2015 Nepal]

Figure 32. Short column effect due to staircase landing beam. [Binay Shrestha, 2015]

Figure 33. Short column due to partial height infill wall [WHE Report, Nepal]

Figure 34. Shear failure due to short column effect observed in Port-Blair, India, during Sumatra earthquake 2004.

Keyvan Ramin and Foroud Mehrabpour studied a 4-story RC building that consists in the analysis of simple 2-D frames of different floor heights and different number of bays. After analaysing it resulted that the increase in shear was greater in short columns. The maximum stiffness of the structure was also provided by non-linear static pushover analysis.

As said, the short columns are needed to be more resistable and require a reinforcement with more bars and steel should be better used as stirrups than as longitudinal bars. While, for existing buildings, a solution for short columns to resisit shear capacity is to be retrofitted by modern techniques, such as FRP.[38]

The Solution for short column effect.

- As soon as possible they should be avoided since the planning and architectural design phase.
- It is of great importance the reinforcement detailing of RC structures, because it affects the provision over the full height of columns.
- Regarding the existing buildings having short columns, retrofitting is a fast and economical solution.
- Applying continuous FRP systems for strengthening of columns. It improves shear capacity, reinforcement bar lap splice capacity and considerable ductility. [C.V.R. MURTH, 2005]

2.4.8 Heavy Overhangs effect.

Heavy overhangs may be seen as one of these irregularities that adversely affect the seismic behavior. This leads to increase in weight and period of the building, which amplifies the displacement demands. In addition, the possible absence of the beams in the overhang zone causes additional loss in the lateral force capacity and rigidity. Buildings having balconies with heavy concrete parapets and large overhanging spans perform greater damages during the recent earthquakes in the world compared to regular buildings.

H.B Ozmen 2011 studied 144 building models that reflect Turkish reinforced concrete building stock with and without heavy overhangs are investigated in order to evaluate this effect on seismic performance. Models include versions with and without beams in overhang zone. Buildings are designed per 1975 and 1998 Turkish Earthquake Code. He used Sap2000 software for modeling and non-linear static analyses of the buildings. Seismic displacement is determined by non-linear dynamic response history analyses using 264 different acceleration records. Capacity values of IO, LS and CP performance levels were compared with the demands from sets of records having mean peak of ground accelerations of 0.2g, 0.4g and 0.6g. It has been found that, heavy overhangs have some positive and negative effects on structural behavior with the conclusion that negative effects are dominant. The presence or the absence of the beams at the overhang zone has significant effect on the seismic behavior and latter is much more detrimental.

Figure 35. Earthquake failures of cantilever projections buildings in Turkey. [M.Dogan 2007]

Figure 36. Failure of balconies at Shizugawa Hospital during Tsunami 2011 [Fraser et al. 2013]

Fig 37. Failure due to overhangs [Dogˇangu¨n 2004]

2.4.9 Buildings Pounding Effect.

Due to high urbanization development and population increase, building closely space buildings has been seen as a solution. During earthquakes, closer buildings with insufficient separation, vibrate out of phase and so the experience pounding effect between them. The pounding of buildings causes much damage and moreover, it may cause complete collapse. Structural pounding occurs when two adjacent buildings collide. This collision happens because of insufficient distance between structures or even difference in stiffness and mass. However, many designing codes gives limitation for a minimum seismic gap, but sometimes this gap seems to be as a waste of prime real estate.

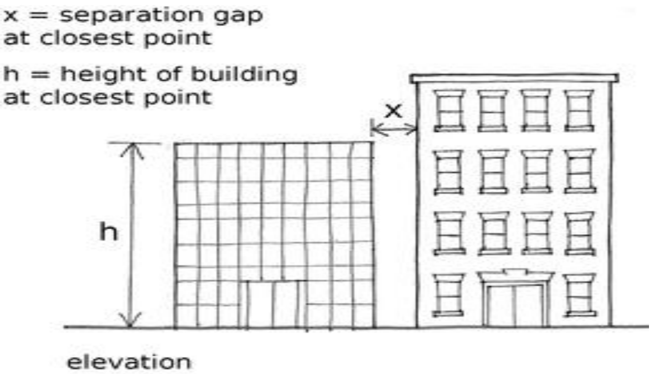


Figure 38. Pounding potential. (Taxonomy.org)

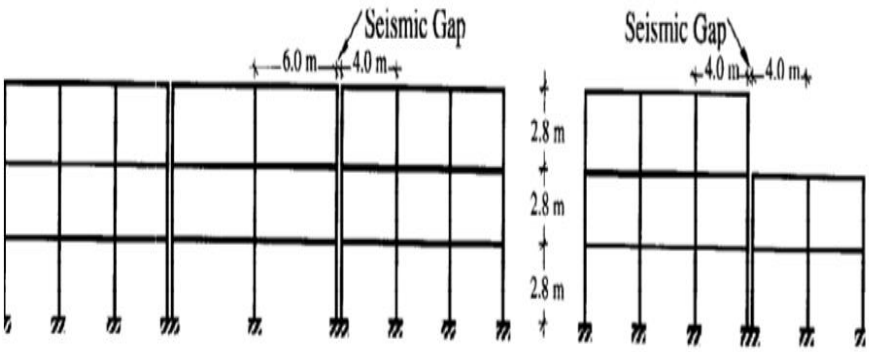
Calculation of gap space between buildings.

S = √ (Q12 + Q22) is a SRSS (Square Root of the Sum of the Squares) Method

Q1 = highest displacement of building -1; Q2 = highest displacement of building – 2

S should not be greater than the distance between adjacent buildings.

In some case, the gap size depends only on the maximum displacements that present each building. Sometimes, the simple sum of the displacements of each building. And in other cases a smaller value that may be a percentage of the previous one, or a quadratic combination of both building maximum displacements. In some codes, this gap value is taken while considering the soil type and condition, also the seismic activity.



a) row of three buildings

b) buildings with different height

Seismic pounding damages was found to be accurrate between closer buildings during the 1985 Mexico, 1994 Northridge, 1995 Kobe, 1999 Kocaeli and 2008 Sichuan earthquakes. [2010 Anil C. Wijeyewickrema]

Building damages due to seismic pounding effect.

Figure 39. Buildings Pounding Damages Observed in the 2011 Christchurch earthquake Christchurch Earthquake.

Figure 40. Collapse of buildings results in pounding in adjacent buildings in Chautara, Sindhupalchowk.

Figure 41. Collapse of adjacent buildings during the Bingöl earthquake.2003.

Solution to prevent seismic ponding damages.

- Providing safe separation distances.

Figure 42. The three-metre gap between two buildings on Euston Road in central London.

- Constructing new RC walls.

A solution to protect the building during a ponding phenomena, can be by reducing the relative displacement, through provision of an additional stiffer member, such as: bracings, shear wall and by combined system, to ensure stability out of phase vibration under provided gap. [Amruta Sadanand Tapashetti1 2014]

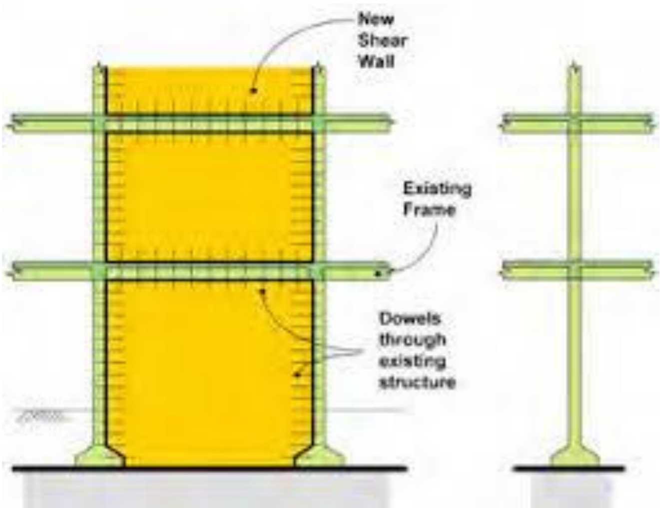


Figure 43. Construction of New Shear Wall method.

- Combined system of RC wall and dampers.

Considering many experimental works has been obtained that the FVD technique ensure additional stiffness, energy dissipation and strength capacity during strong winds or moderate earthquakes.[M.Gabriella, 2004]

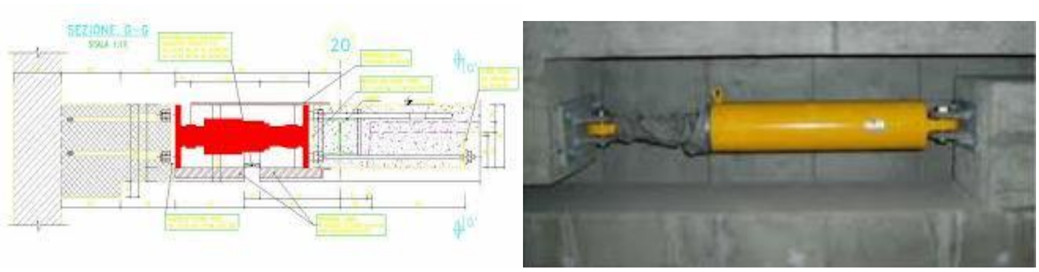


Figure 44. Use of Viscous Dampers and Shock Transmission Units in the Seismic Protection of Buildings. [M.Gabriella CASTELLANO, 2004]

2.4.10. Improper Detailing of Reinforcement and Lack of Construction Quality.

Despite the major causes of building collapse, such as structural irregularities that were mentioned above, there are even some other reasons that might seem minor and neglectable, but actually they cause large building damages.

Poor knowledge of concrete work, such as: improper curing, improper water cement ratio, poor sub grade soil compaction, poor placement of concrete, etc. also leads to the poor quality of concrete work. Poor planning of work also causes the defective execution. Example of such execution errors are column not being in plumb, providing inadequate reinforcements, errors in joint detailing, detailing of cantilever beam etc. In detailing the stirrups in the columns, there has been always seen that some structures do not fullfill the requirements for lateral shear as required in structural designing standarts such as IS 4326-1976 and IS: 13920-1993.[Kevin Stanley 2015]

Some important rules or provisions, and of low cost for reinforcement design of structural members, consist of over-design flexural members to resist flexural hinge formation durimg shear failure, use proper shaped shear links and strirrups with specific hooks degree, and also confine high compressed concrete for columns. For a frame, the philosophy of strong-column, weak-beam should be applied as much as practical. Designing codes provide rules and techiques for full ductility details, as mentioned in IS: 13920, which suggests the use of the High Reduction Factor R=5 to make the design good and economical. In cases when ductility details are not applied, the Reduction Factor is specified as only 3.0, making the design force to become 1.67 times the case when full ductile detailing is applied. This leads to be more expensive and partly unsafe due to the brittle behaviuor of memebers.[Syed Mehdi 2012]

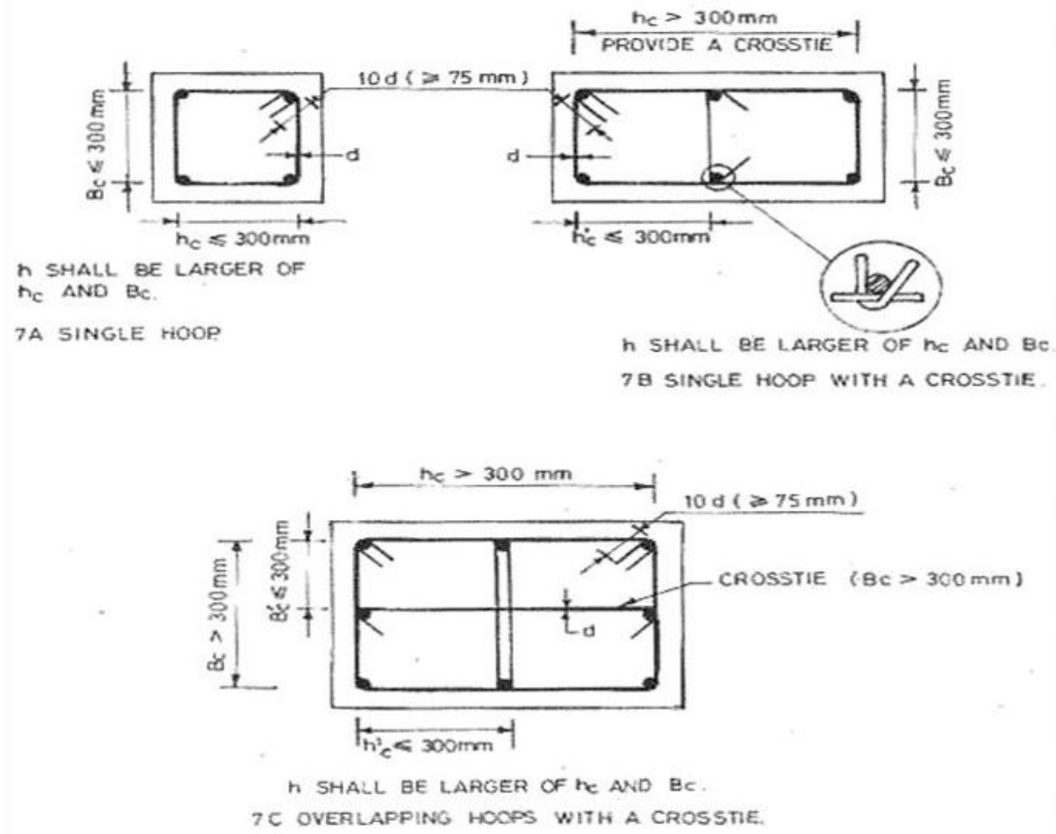


Figure 45. Ductile Detailing Confirming to IS:13920

Figure 46. A Zoom+ view of building in on-going construction with not proper reinforcement detailing for Beam-Column joint.

Figure 48. Partial Building Collapse due to failure of beam-column joints in the Izmit, Turkey earthquake of August 17, 1999.

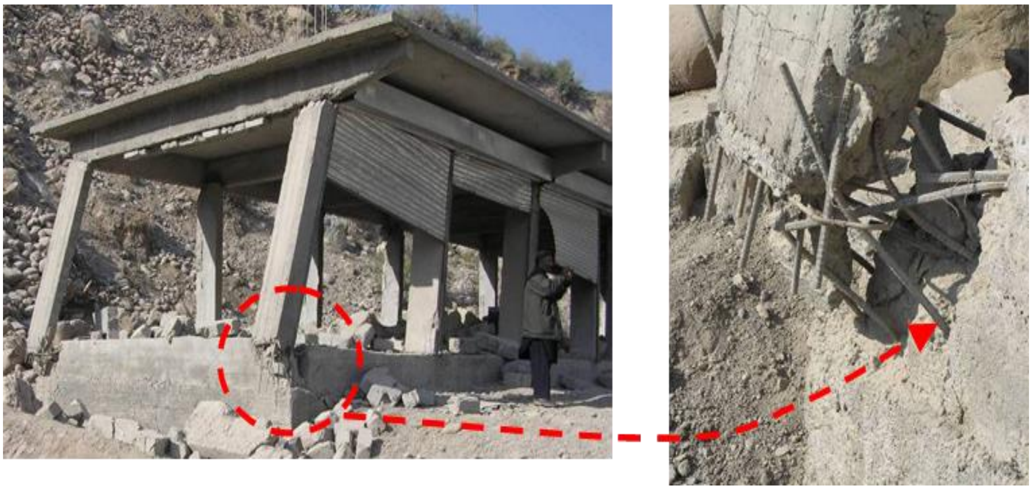


Figure 49.Very poor lap location leading to partial collapse of the building. (Gopal Chapagain, Nepal)

Poor or inadequate supervision leads to the poor construction work, and thus it is also one of the important reasons for building collapse. The construction engineer should be present at site to supervise all processes. He must have the right materials sampling, beign test in specialised laborator.[Ayodeji Emmanuel Oke, 2011]

Figure 50(left).Pante Pirak Supermarket, a 3-story building, the collapse was due to poor quality of construction, and right, Seismic deficiencies - very poor quality ofconcrete (2003 Boumerdes earthquake)

As a review of all, respecting even the provision codes, the irregularity standards for both horizontal and vertical irregularities are listed briefly in table below.

IRREGULARITY LIMITS PRESCRIBED BY IS 1893:2002, EC8:2004, UBC 97, NBCC 2005

Type of Irregularity	IS 1893:2002[1]	EC8 2004[2]	UBC 97[3]	NBCC 2005[4]
Horizontal				
a) Re-entrant corners	$R_i \leq 15\%$ (Fig.2)	$R_i \leq 5\%$	$R_i \leq 15\%$	-
b) Torsional irregularity	$d_{max} \leq 1.2\ d_{avg}$	$r_x > 3.33\ e_{ox}$ $r_y > 3.33\ e_{oy}$ r_x and $r_y > l_s$,	$d_{max} \leq 1.2\ d_{avg}$	$d_{max} \leq 1.7\ d_{avg}$
c) Diaphragm Discontinuity	$O_a > 50\%$	$r_x^2 > l_s^2 + e_{ox}^2$	$O_d > 50\%$	-
	$S_d > 50\%$	$r_y^2 > l_s^2 + e_{oy}^2$	$S_d > 50\%$	
Vertical				
a) Mass	$M_i < 2\ M_a$	Should not reduce abruptly	$M_i < 1.5\ M_a$	$M_i < 1.5\ M_a$
b) Stiffness	$S_i < 0.7S_i+1$ Or $S_i < 0.8\ (S_{i+1} + S_{i+2} + S_{i+3})$ (Fig.2b)	$S_i < 0.7S_{i+1}$ Or $S_i < 0.8\ (S_{i+1} + S_{i+2} + S_{i+3})$	$S_i < 0.7S_{i+1}$ Or $S_i < 0.8\ (S_{i+1} + S_{i+2} + S_{i+3})$	$S_i < 0.7S_{i+1}$ Or $S_i < 0.8\ (S_{i+1} + S_{i+2} + S_{i+3})$
c) Soft Storey	$S_i < 0.7S_{i+1}$ or $S_i < 0.8\ (S_{i+1} + S_{i+2} + S_{i+3})$	-	$S_i < 0.7S_{i+1}$ Or $S_i < 0.8\ (S_{i+1} + S_{i+2} + S_{i+3})$	$S_i < S_{i+1}$
d) Weak Storey	$S_i < 0.8S_{i+1}$	-	$S_i < 0.8S_{i+1}$	-
e) Setback irregularity	$SB_i < 1.5\ SB_a$ (Fig 2c)	$Rd < 0.3T_W < 0.1\ T_W$ at any level	$SB_i < 1.3\ SB_a$	$SB_i < 1.3\ SB_a$

Table 1. Irregularity limits for structures.[39]

2.5 Earthquakes vulnerability in several types of construction.

It just takes some minutes of shaking and the result consist on an earthquake vulnerability affecting many buildings, communities, countries…

All of these are later in need of a huge mobilization starting from governance, plans, design, construct and reconstruct, and every else step taken to minimize the seismic hazard.

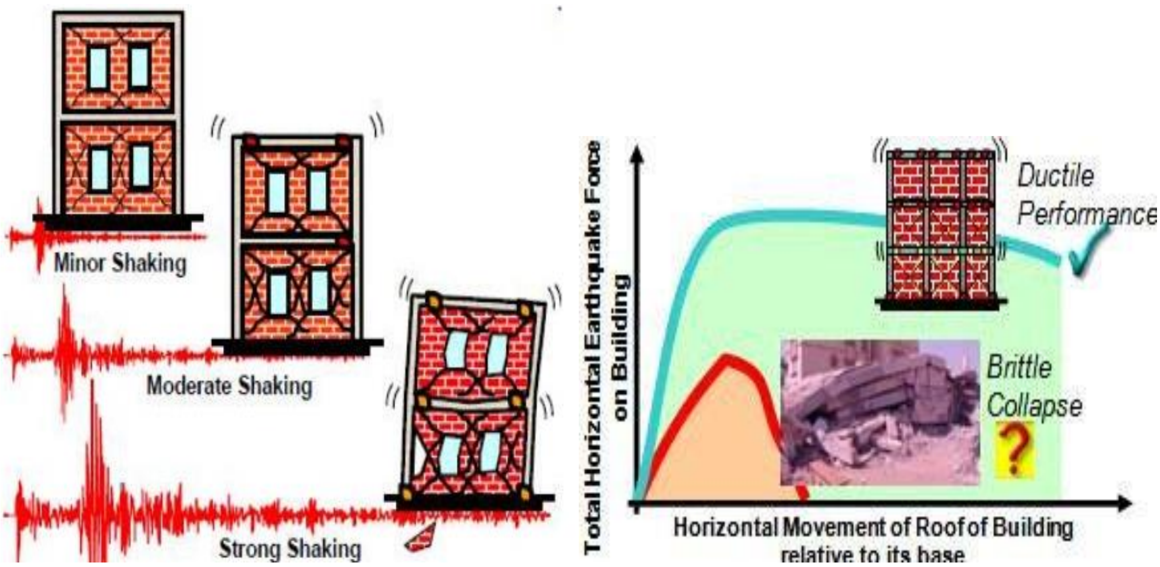


Figure 51. Buildings performance under seismic activity.

Some factors contributing to earthquake vulnerability of building stock in developing countries are:

1. **Large occupation already in seismic areas**.

There is a large human placement located in earthquake risked areas. Many of buildings in such areas have resulted to be old and of poor quality due to construction, maintainance and even age. All of these are factors which makes the building really vulnerable in an earthquake scenario.

2. **Non-engineered constructions.**

Through some estimations done, it has resulted that the presence of non-engineered constructions are accounted to be more than 50% and worst ever, in some case more than even 90%, like in Kathmandu. The earthquake history of this century has shown 75% of disasters occurred due to the demolishment and collapse of those buildings which were not properly designed, with lack of material and construction quality. [Coburn, 1992]

3. Use of poor building typologies.

The typology of building construction is a big factor indicating injuries risk during an earthquake. Statistics for 1950-1990 shows that the most life loss of victims happended in the collapse of masonry buildings. Also non-engineered concrete-frame buildings are vulnerable and in cases of collapse they are havier and cause higher percentage of people killed than masonry buildings.

4. Inadequate control in building construction

Unfortunately, in many countries, the seismic building code is still not mandatory but a recommended practice. Sometimes, this happens because the institutions do not have the capacity for implementing strict standards to provide a mandatory seismic code for construction of buildings. What is most to warry about, consist on milion of buildings which even though being in large seismicity prone country, they still grow in construction without the seismic provision and resistance, every year.[40]

2.5.1 Typical structures damages during recent earthquakes.

Figure 52. Example of non-ductile concrete frame building collapse at the Veterans Administration (VA) complex in the 1971 San Fernando earthquake.

Fig 53. Damage and collapse of buildings of the Olive View Hospital and Sylmar hospital due to the 1971 San Fernando earthquake.

Fig 54. Partial collapse of a 6-story commercial building (Chou Ward) with a torsional irregularity.

The above building was a reinforced concrete structure with perimeter shear walls on two orthogonal sides of the building and gravity framing on the other two orthogonal sides of the building. The torsional irregularity caused the building to rotate during the earthquake. As the building rotated, large displacement of the gravity frames at the corner away from the stiffer walls exceeded frame capacity causing failure of columns and collapse or sagging of floors above.

Figure 55. 2015 Gorkha earthquake

In Gorkha earthquake, 2015. (fig 55) such failures happened due to the:

- **a.** Ground floor of buildings without infill walls.
- **b.** Ground floor of an apartment building without infill wall.
- **c.** Soft storey and structural pounding failure.
- **d.** 16 mm diameter rebar but lack in quantity (only 4 and corroded) from a collapsed building in Kathmandu. (link.springer.com)

2.5.2 Sub-heading. Earthquakes in Albania and Vulnerability of RC frame buildings.

The Vulnerability Analysis has a special importance in evaluating the damage to the buildings of the educational, residential and health systems, so it is necessary to understand their structural behavior against the earthquakes as much as possible.

Civil Engineering is closely related to seismology, earthquakes and other natural disasters. We need to have safe buildings to withstand this natural phenomenon. But how possible is this in Albania?

Earthquakes pose a high risk to people living and working in irregular constructed buildings. The damages that earthquakes generate, are generally limited to an area around the epicenter. Disasters are caused by a combination of strong earthquake vibration, earthquake size, poor construction quality (low structural capacity leading to poor performance during quake action) and high population density in the area where the earthquake falls. [E. Luca, 2012]

The two major factors that transform an earthquake into a devastating disaster are the vulnerability of the building (not adequately built), and unfavorable soil conditions under the building.

It has been noted that Albania has been characterized by intense small and medium-sized earthquakes. However, in the area where Albania lies, some catastrophic earthquakes have occurred over the centuries, which have destroyed to the point that entire cities have been obliterated. [N. POJANI, 2010]

Due to this fact, since the early times, the Albanian community has created some rules and advice to have somewhat stronger buildings. Over time (especially over the last century), these empirical rules and experience were transformed into a complete official building code, which has steadily updated through significant changes and improvements that have been reflected up to the last code design.

Of course, it would be optimal to have a level of security for almost all functional structures, but this leads to an impossible economic and financial solution. This led the experts to decide that the most important, vital and emergency buildings would have a higher level of security against the foreseen risks. One of the reasons for a building to have a higher level of security is the number of people inside the building at the time of the event.[41]

How safe are buildings in Albania?

Even in Albania, the building stock is very familiar to structural irregularities, beginning since masonry typology, RC frame buildings, and till now that we have more complex structures and monolith system. The factors affecting structural irregularities are counted to be a lot but the most common ones to be mentioned are: poor quality design due to old non-updated codes, economical issues due to "saving" moto, lack of materials import, lack of civil engineering knowledge. After all these issues responding mainly to communism period, still some structural problems did not find solutions even in further years. Moreover, the existing building performed additions, intervention in structural system and many soft stories were created. The capacity of original design is exceeded, the force distribution path has changed and people are still not aware of what would these kind of buildings perform in case of even moderate earthquakes.

What experts say?

Buildings built before the '90s are not safe for seismic movements. The Genova tragedy alarmed the Albanian experts that the constructions of this period in Albania show the risk of degradation.

Arben Dervishi, one of the pedagogues who has been part of a group that has proved the sustainability of these buildings, speaks of the importance of studies to see where amortization goes. He said that: - "There is no control and maintainance required. Even for infrastructure engineering works, there is no security. I do not want to alarm, but they are degraded."

Prevention Fund seems to be the solutions but Albania does not have that, there are only funds when the damage is done. "

Deputy Transport Minister Artan Shkreli, after the tragedy that followed the collapse of the bridge in Genova considered it an unchallenged call for reflection. Evidence made on objects built in '80 or 90 years ago according to him yields intimidating results for degradation, so a new control standard and a new non-partial radical intervention such as Genova**. (Albanian magazine reporter, 2018)**

In the figures below there are some pictures showing building damages under earthquakes in different areas of Albania.

Fig 56. Building damages during an earthquake of 4.5 magnitude in Oher, 2012 due to non-ductility of structure and poor materials quality.

Fig 57. Building collapse due to an earthquake of 5.1 magnitude in Bulqize, 2018. Building are lack of shear walls and they also are constructed with no proper structural design.

Fig 58. Agriculture ministry’s building damages and residential buildings due to an earthquake near Tirana in 2018. Some of building perform soft story effect and others are lack of material quality which has been loosing its class by time.

Fig 59. Albanian buildings damaged during earthquake disasters in Elbasan. Many buildings designed according to old designing codes, with no good reinforcement design for shear.

CHAPTER 3

ANALYTICAL MODEL OF CASE STUDY RC FRAME BUILDING.

3.1 Description of the Case Study Structure.

One RC building, 6 story height is chosed to represent a mid-rise building in Berat, Albania. This kind of building is mostly representative of all buildings typology constructed at last of ’90, due to their similar construction practice, codes referred, period and quality. It is a column-beam frame structure, without any shear walls, but infill masonry walls.

It is a regular frame building, 15m by 19m in plan. It has 4 bays by 4 m and 1 bay of 3 m along X direction and 2 bays by 4.5 m and 2 bays by 3 m along Y direction.

Typical floor height is 3m, apart from the ground floor which is 3.5 m in height. Total height of building is 18.5 meter.

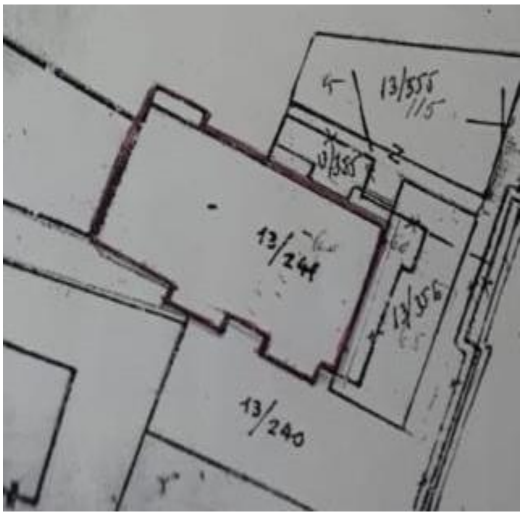


Fig 60. top view of selected building and it-s map location.(6-th story apartment photo)

STRUCTURAL PLAN.

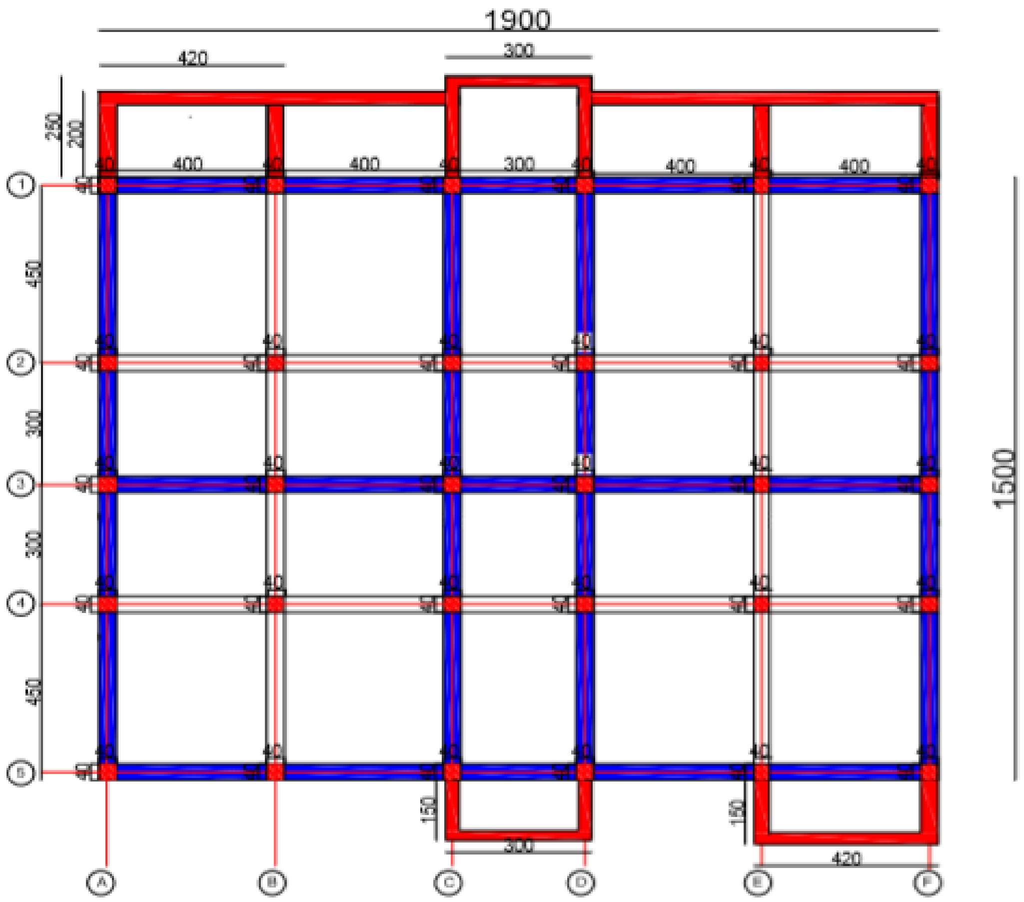


Figure 61. Structural planning of six story building. (units in cm)

Masonry infill walls t = 300mm

The RC columns are typically 400 x 400

The column and beam dimensions used in this study are typical frame element proportion used in most building of mid-rise category in Albania. The RC columns are typically 400 x 400 mm. Beams are of 400 x 300mm.

Refering Albanian old code, there are less measurements taken when designing for seismicity effect on buildings. Material properties are of most common materials used in Albanian construction such as; C20/25 MPa for the concrete and 355 MPa for the reinforcement steel. The slabs are of SAP type, panels of hollow light bricks with steel confinement. Substructure is realised by RC footings under columns.

The vertical loads are live and dead loads of slab, wall loads on beams and dead loads of columns and beams.

To evaluate the seismic performance of the selected building it is modified to have the structural properties observed in damaged buildings during the last earthquakes in Albania and worldwide. Structural irregularities consist in: soft story and short columns, large and heavy overhangs and soft story with heavy overhangs.

Floor height is 3m high each floor, except of the first ground story which due to commercial activities is 3.5meter height. This indicates the lower stiffness of the first story compared to the others above, leading it to a soft story. Except of difference in height it is considered a soft story even due to the lack of masonry wall for having more free space in the shops or bars of ground story.

ELEVATION VIEW

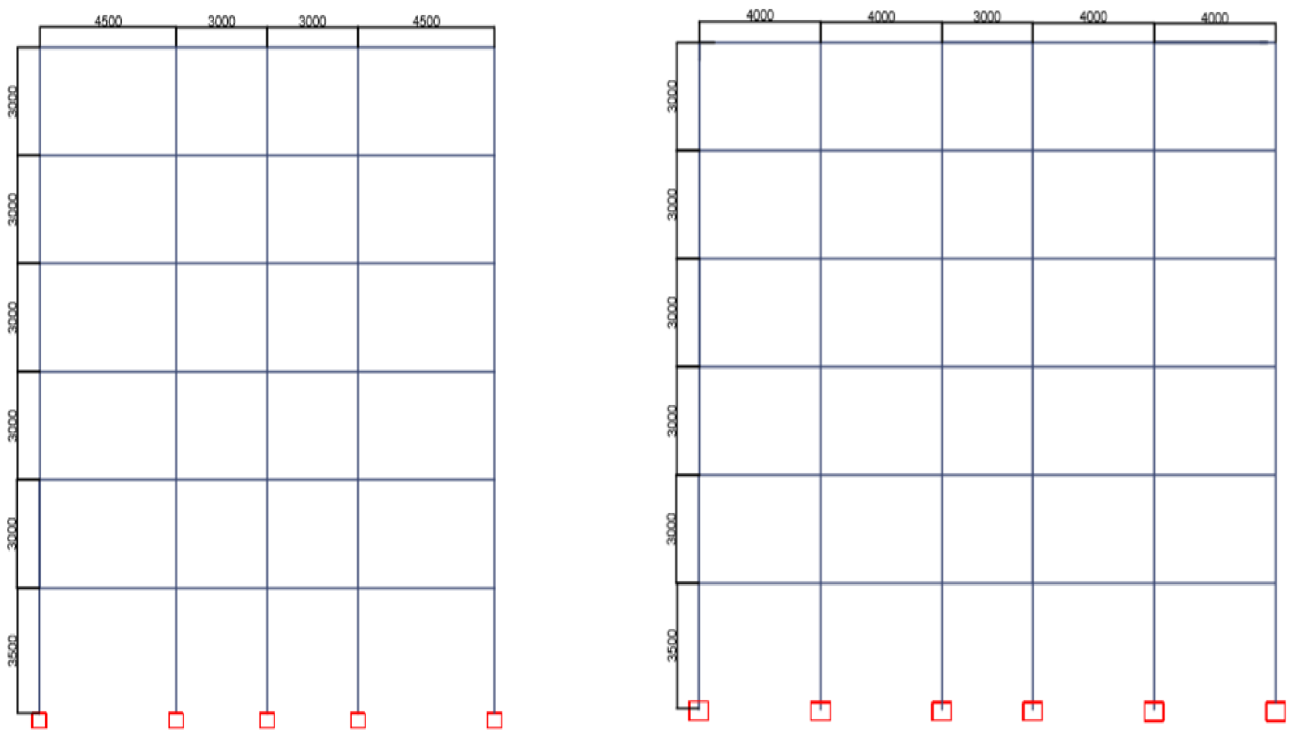


Fig. 62 Section view of building’s axes AA; FF (left) and 1-1;5-5 (right)

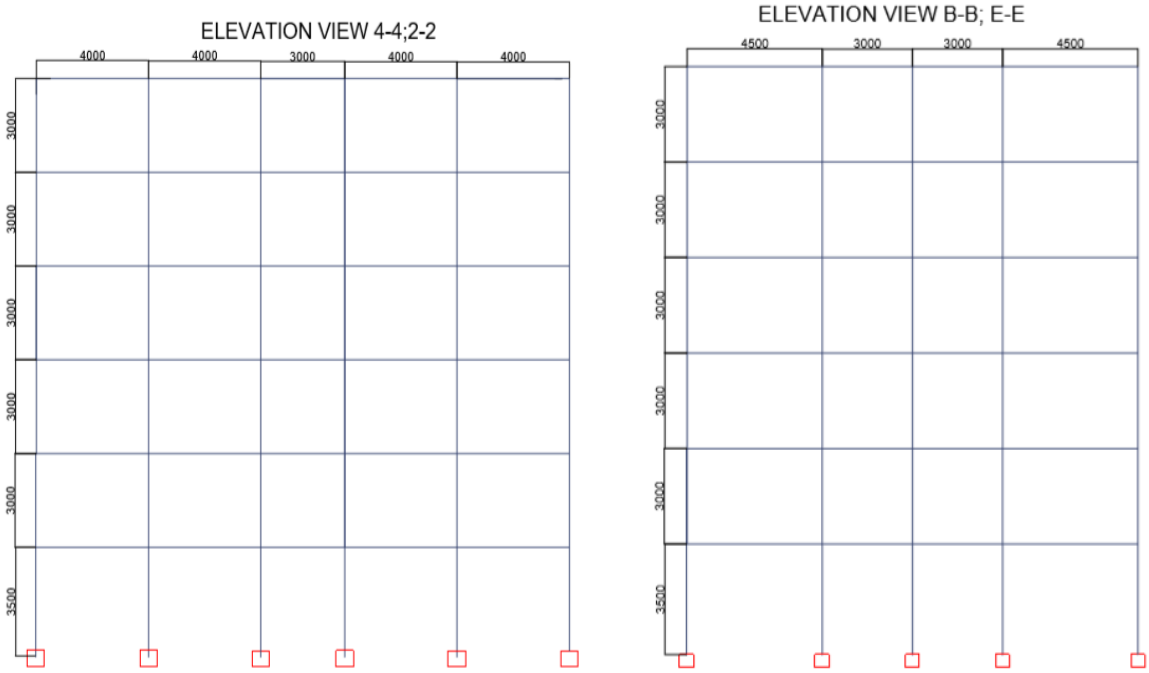


Fig 63. Axes cross-section of 6-th story building.

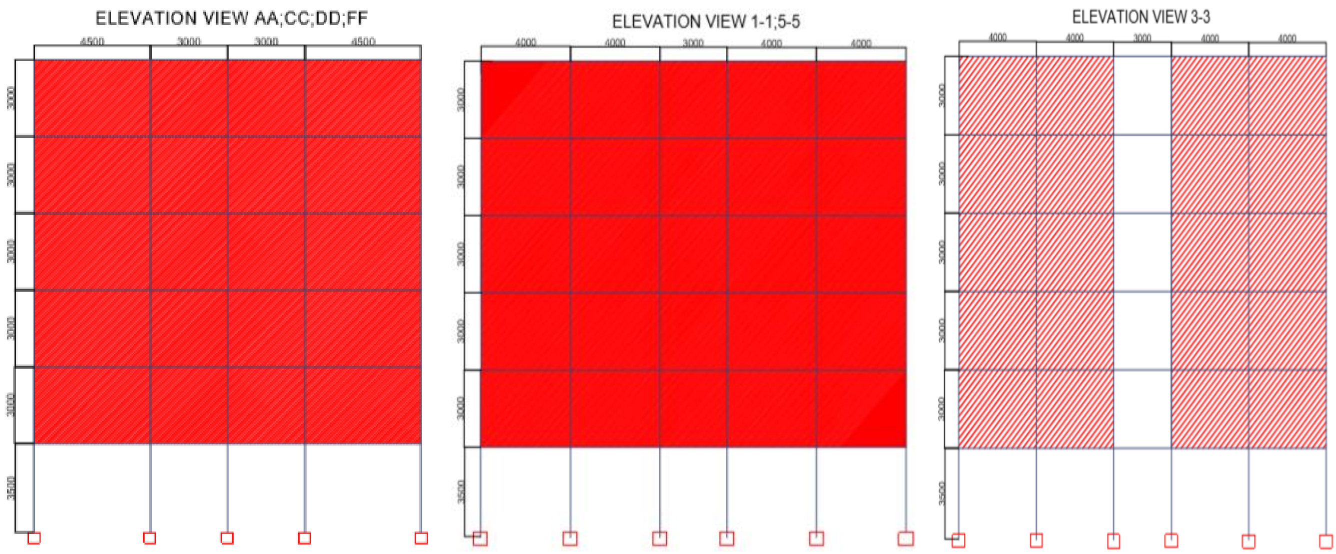


Fig 64. Masonry infill walls in RC frame, indicated by red color.

Masonry infill walls are generally used for increasing the initial stiffness and strength of RC frame buildings. It is mostly taken as a non-structural element. In many buildings in Albania, it is very common to have the first storey of masonry infilled reinforcement concrete (RC) frame building open, so that to provide parking space or any other purposes in the first storey. This is also termed as “Soft Storey”. The storeys in upper levels consisnts in brick infilled wall panels with various opening in it. Such buildings are very undesirable in seismic areas because many vertical irregularities are created in such buildings which have always performed very poor behavior during earthquakes history. (Kulkarni 2013). Based on past earthquake dmages experiences, they cause several undesirable effects under seismic loading: short-column effect, soft-storey effect, torsion, and out-of-plane collapse.

Moreover, in sections above, there’s also considered the formation of soft story irregularity because of taking out the masonry infill walls in the first story.

Reinforcement details.

Regarding reinforcement details used in the present building, got to admit that considering the period of construction there was lack of steel import in Albanian industry, so that, even the reinforcement details used in RC frame buildings were poor in quantity and quality too. Talking about seismic measurements they are not carefully taken in consideration. This can be obtained by having a look at the joints of RC frame, where beams are designed at a method that there are no shear links preventing seismic movements, but they follow the same space in distribution all along the beam.

Moreover, the columns do not fullfill the design recommendations, the shear links on top and bottom are of a space of 15 cm distance, when actually had to be of 10 cm space. To add up with, the shear links are of smaller diameter than recommended, they are of ϕ6 diameter instead of ϕ8 dm.

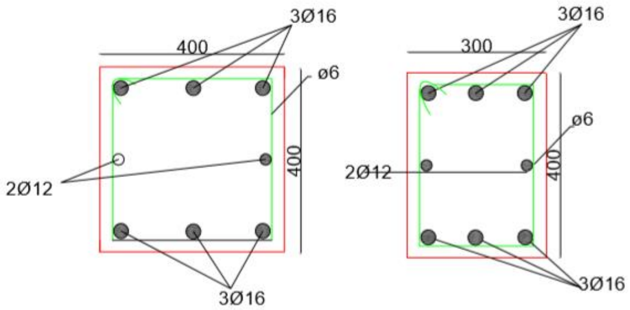


Fig 65. Column (left) and Beam (right) reinforcement details for 6-story frame (units in mm)

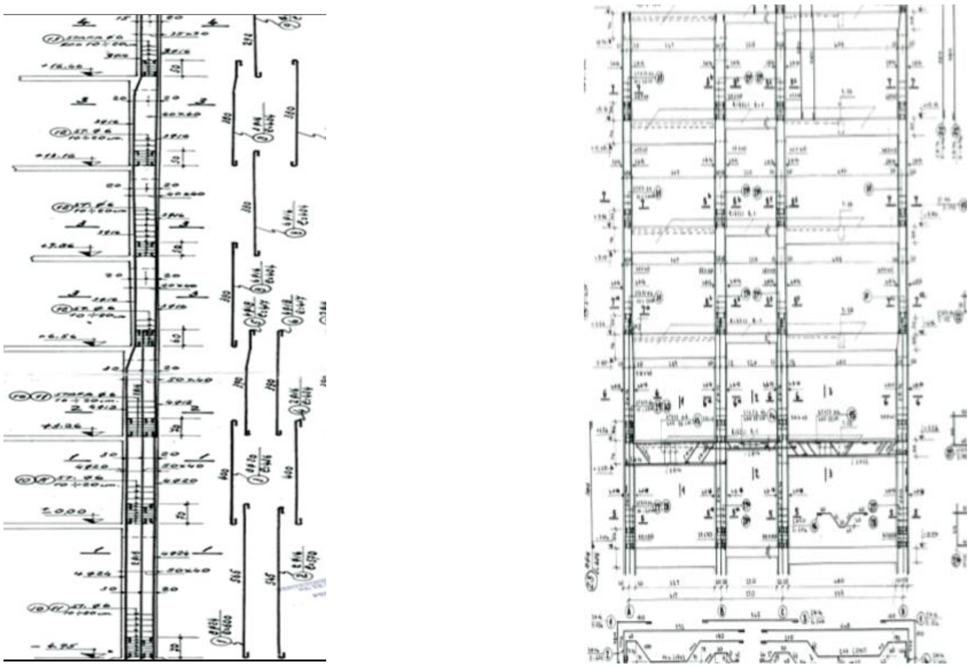


Fig.66 Reinforcement details in columns and all along RC typical frame structures. (old archives photo, E.Luci)

Building irregularities.

In this study, short columns are present due to semi-infilled frames, mid-story beams at the stairway shafts of the building, semi-buried basement or band windows. In Albania, typically basement floors are utilized for various purposes. In order to illuminate the basement floor, band windows are constructed over the soil level, where the wall between columns is built up to a certain level and a gap is left for the window. Because of the rigidity of the wall below window level columns do not bend, so they are forced to bend within the short length of the window gap. The columns are exposed to massive shear forces under the effect of bending.

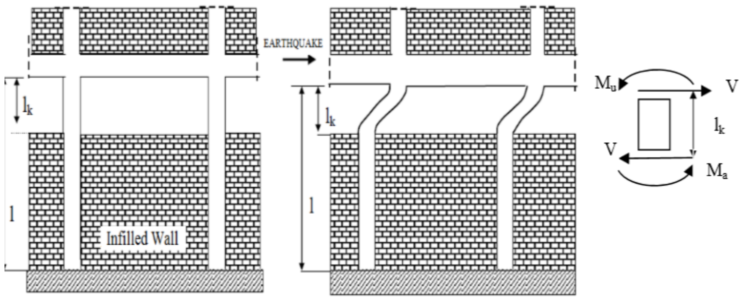


Fig 67. Construction frame with infill wall and behavior of short column when horizontal load (e.g., earthquake) acts on structures

Apart from many structural irregularities due to poor construction quality and materials, many occupnats of these buildings started to make additions in the existing structures. In our case, the second floor is subjected to many overhangs just in one side of building. They shift the mass center upwards and displace it from the centre of rigidity, being dangerous and negatively affecting the building under seismic load.

Fig 68. second floor overhangs due to human intervention by adding extra rooms.

The Model Considered Frames are listed below.

MODEL DENTIFIER FOR CONSIDERED FRAMES	DESCRIPTION
REF	Reference Building (without any irregularity).
OSO	One sided overhang strory.
TSO	Two sided overhang building
SSH	Soft story due to 3.5 m ground story height (instead of 3 m).
SSHI	Soft story due to both height and infill effect.
SSHOSO	Soft story due to both height and infill, and one sided overhang
SHC	Short column due to sem-infilled bays.
SSHITSO	Soft story due to both height and infill, and two sided overhang.
SSWTSO	Soft story due to absence of masonry infill wall at ground story and two sided overhang

Table 2. Considered frames.

CHAPTER 4

METHODOLOGY

4.1 Methodology followed for evaluation of RC frame buildings seismic vulnerabilities.

The purpose of the vulnerability evaluation is to provide the possibility of a certain damage level to a certain type of building due to an earthquake scenario. Different methods for assessing vulnerability, which have been proposed in the past for the use of loss calculation can be divided into 3 main methods: **empirical, analytical, hybrid.** (ISET Journal of Earthquake Technology)

Analytical Method.

The base of all the analytical methods for assessing the vulnerability is the **nonlinear analysis** of the structure.

These methods are more detailed in terms of vulnerability evaluation, which not only allow detailed investigation, but also take care of the direct account of the various characteristics of building objects. The last is a certain disadvantage of empirical method.

Based on the analytical methods is the non-linear analysis of the building through which is possible to find the performance point by **Pushover analysis.**

4.2 Modeling Approach In SAP 2000.

The software used for this study is SAP 2000 which is known for analyzing general structures including bridges, stadiums, towers, industrial plants, offshore structures, buildings, dam, silos, etc. Analysis and design results are provided for the whole object, giving information that is easier to interprets and consistent with physical nature. [42]

Define grid specification for our REF building.

Define Grid System Data

System Name: GLOBAL

X Grid Data

Grid ID	Ordinate (m)	Line Type	Visible	Bubble Loc	Grid Color
A	0	Primary	Yes	End	
B	4	Primary	Yes	End	
C	8	Primary	Yes	End	
D	11	Primary	Yes	End	
E	15	Primary	Yes	End	
F	19	Primary	Yes	End	

Add Delete

Y Grid Data

Grid ID	Ordinate (m)	Line Type	Visible	Bubble Loc	Grid Color
1	0	Primary	Yes	Start	
2	4.5	Primary	Yes	Start	
3	7.5	Primary	Yes	Start	
4	10.5	Primary	Yes	Start	
5	15	Primary	Yes	Start	

Add Delete

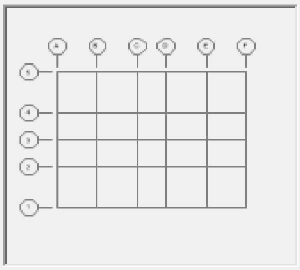
Z Grid Data

Grid ID	Ordinate (m)	Line Type	Visible	Bubble Loc
Z2	3.5	Primary	Yes	End
Z3	6.5	Primary	Yes	End
Z4	9.5	Primary	Yes	End
Z5	12.5	Primary	Yes	End
Z6	15.5	Primary	Yes	End
	18.5	Primary	Yes	End

Add Delete

Grid Lines

Quick Start...



Display Grids as

Ordinates Spacing

Hide All Grid Lines

Glue to Grid Lines

Bubble Size 0.875

Reset to Default Color

Reorder Ordinates

OK Cancel

Table 3. Define grid system data for REF building.

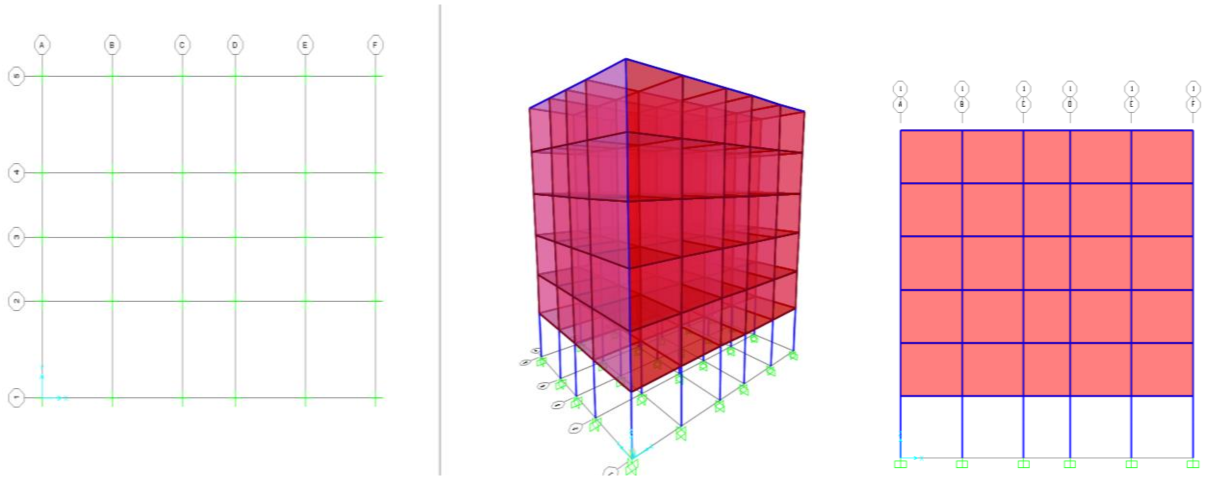


Fig 69. X-Y plan, 3D view and X-Z elevation of REF building.

Define Material.

İn this step we have to define the materials used in building and their properties for reinforced concrete members (columns/beams) and the specific steel grade required for their reinforcement.

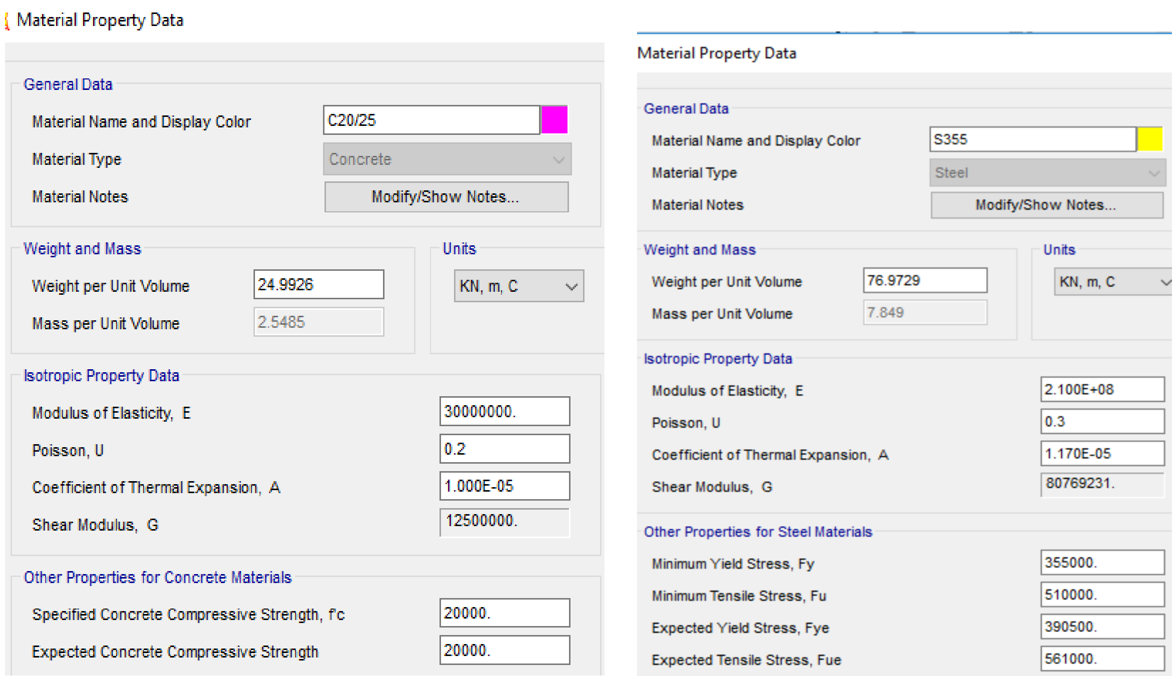


Fig 70. Material properties of concrete and steel.

Define Section frame properties.

We input the typical columns and beams of structure with their respective parameters and design type, such as M3 design type for beams and P-M2-M3 design for columns.

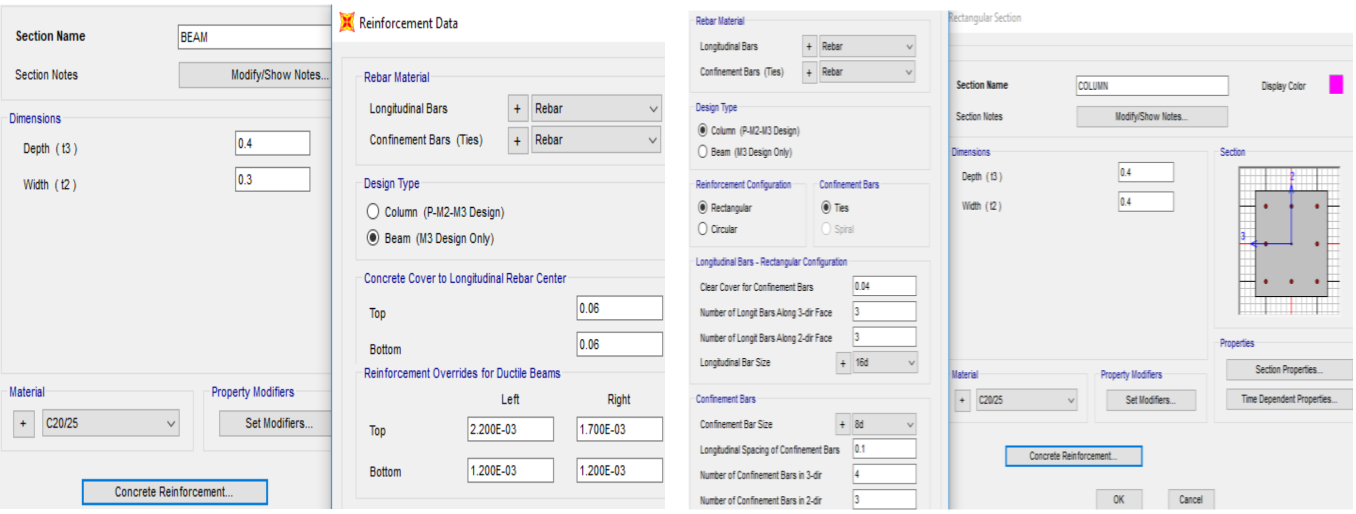


Fig 71. Beam reinforcement data and column.

Define loads.

Firstly we define each member’s self weight.

MEMBER	DIMENSIONS	LOAD CALCULATION	SELF WEIGHT
Columns	40*40	0.40*0.40*25	4 kN/m
Beams	30*40	0.30**0.40*25	3 kN/m
Brick wall	300 mm thick	0.30*19(wall) + 2 * 0.012*20(plaster)	6.18 kN/m2
Slab	150 mm thick	0.15*2.5	3.75 kN/m2

Table 4. Building members self weight calculation

For eassily applying in Sap modelling we define the gravity loads as: dead loads = 1 (dead wall, dead slab, dead) and live loads = 0.3. We also define all load cases to be considered in our model approach, which will be shown in details. Load cases consist in linear static, non-linear static and modal.

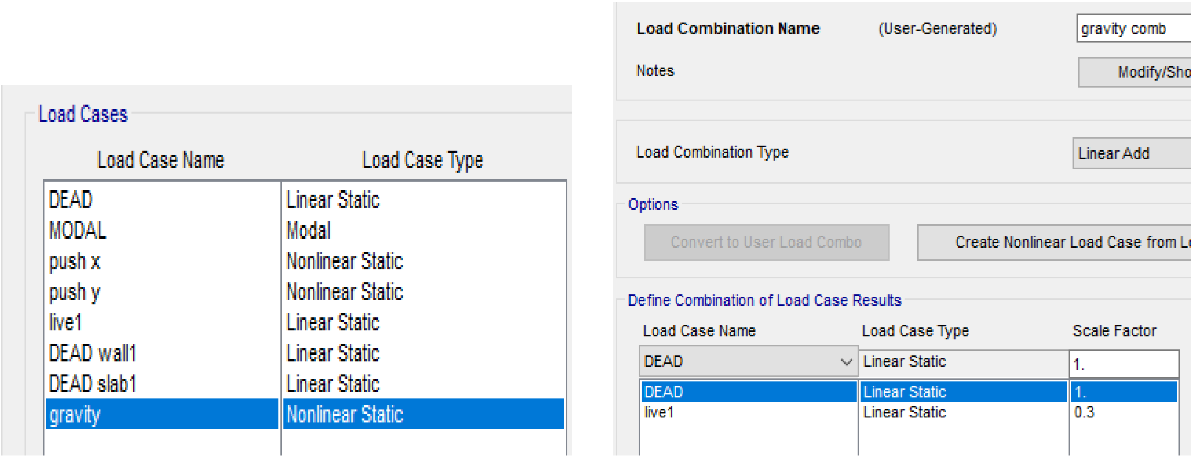


Fig 72. Defining load cases and load combination.

After selecting all structure's beams we also assign slab load distributed on Beams as dead slab distributed load.

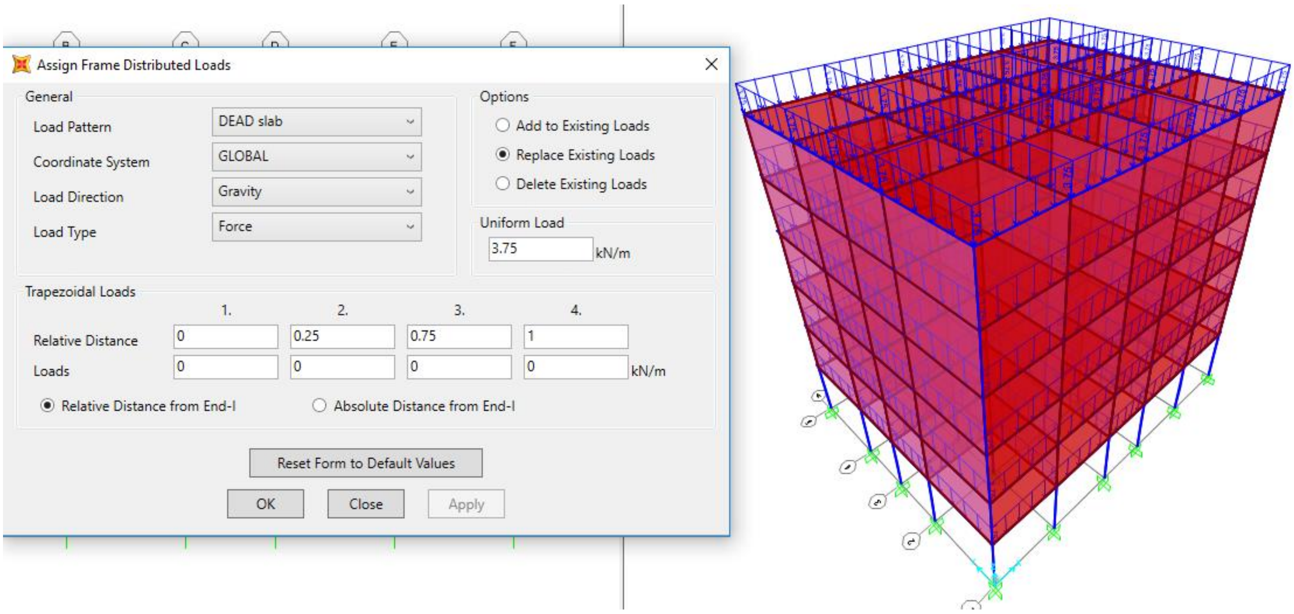


Fig 73. Frame distributed load, dead slab = 3.75KN/m

Approaching for modal analysis.

Modal analysis is a study of the dynamic properties of structures under vibration excitation. It measures and analyses the dynamic response of structures when indicated by an input. Modal analysis uses the overall stiffness of the building to find different periods under which it will resonate naturally. These vibration periods are very important to be defined in earthquake engineering, as it is crucial that a building's natural frequency does not match the frequency of expected earthquakes in the region in which the building is to be construct, otherwise, the structure may continue to develop the resonate and experience structural failures.

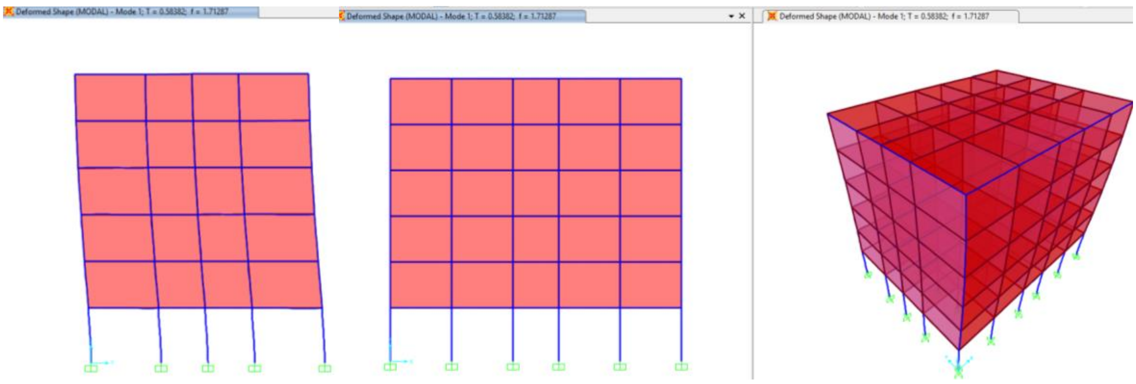


Fig 74. Deformed shape of ref building in modal analysis, step 1.

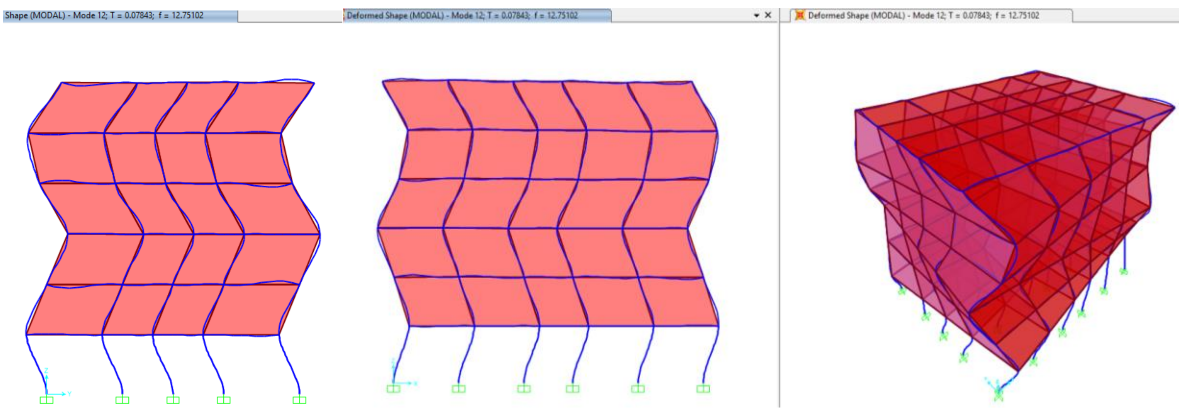


Fig 75. Deformed shape of ref building in modal analysis, step 12.

Modal analysis values for REF building.

İn the table below are displayed found modes, Eigen values, frequency and periods in each modal step.

```
Original stiffness at shift : EV= 0.0000000E+00, f=     .000000, T=  -INFINITY-
Number of eigenvalues below shift =      0
Found mode      1 of     12:  EV= 1.1582664E+02, f=    1.712870, T=    0.583816
Found mode      2 of     12:  EV= 1.1724716E+02, f=    1.723341, T=    0.580268
Found mode      3 of     12:  EV= 1.3010287E+02, f=    1.815363, T=    0.550854
Found mode      4 of     12:  EV= 1.0448027E+03, f=    5.144430, T=    0.194385
Found mode      5 of     12:  EV= 1.0576197E+03, f=    5.175888, T=    0.193204
Found mode      6 of     12:  EV= 1.1814182E+03, f=    5.470436, T=    0.182801
Found mode      7 of     12:  EV= 2.9362347E+03, f=    8.624134, T=    0.115954
Found mode      8 of     12:  EV= 2.9701919E+03, f=    8.673860, T=    0.115289
Found mode      9 of     12:  EV= 3.3201851E+03, f=    9.170675, T=    0.109043
Found mode     10 of     12:  EV= 5.5906605E+03, f=   11.900130, T=    0.084033
Found mode     11 of     12:  EV= 5.6433870E+03, f=   11.956114, T=    0.083639
Found mode     12 of     12:  EV= 6.4187395E+03, f=   12.751022, T=    0.078425

NUMBER OF EIGEN MODES FOUND              =          12
NUMBER OF ITERATIONS PERFORMED           =          12
NUMBER OF STIFFNESS SHIFTS               =           0
```

Table 5. Modal analysis values for ref building.

Every mode is independent of all others. They all have different frequencies (with lower modes having lower frequencies) and different shapes (with lower modes having greater amplitude).

As the lower modes vibrate with greater amplitude, they cause the most displacement and stress in a structure, called fundamental modes. [Mintu Choudhury, 2015]

4.2.1 Approaching for Pushover Analysis. (REF building)

After designing the RC frame structure, a nonlinear pushover analysis is carried out to evaluate the structural seismic behaviour. Pushover analysis consists of the application of gravity loads and a certain lateral load pattern.

The applied lateral loads defined by PUSHX in the X direction showing the forces that would be experienced by the structures when subjected to a ground shaking.

At each step the structure experiences a different state for stiffness, where IO, LS and CP stand for immediate occupancy, life safety and collapse prevention.

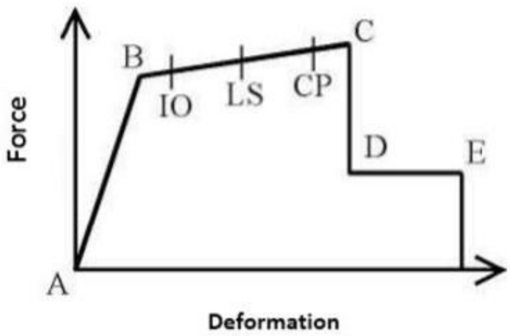


fig 76.Load- deformation curve. (Fema 356, 2000)

Defining non-linear static load cases and their required parameters.

The defined load cases to perform non linear static analysis are represented below, consisting of Pushx, Pushy and Gravity.

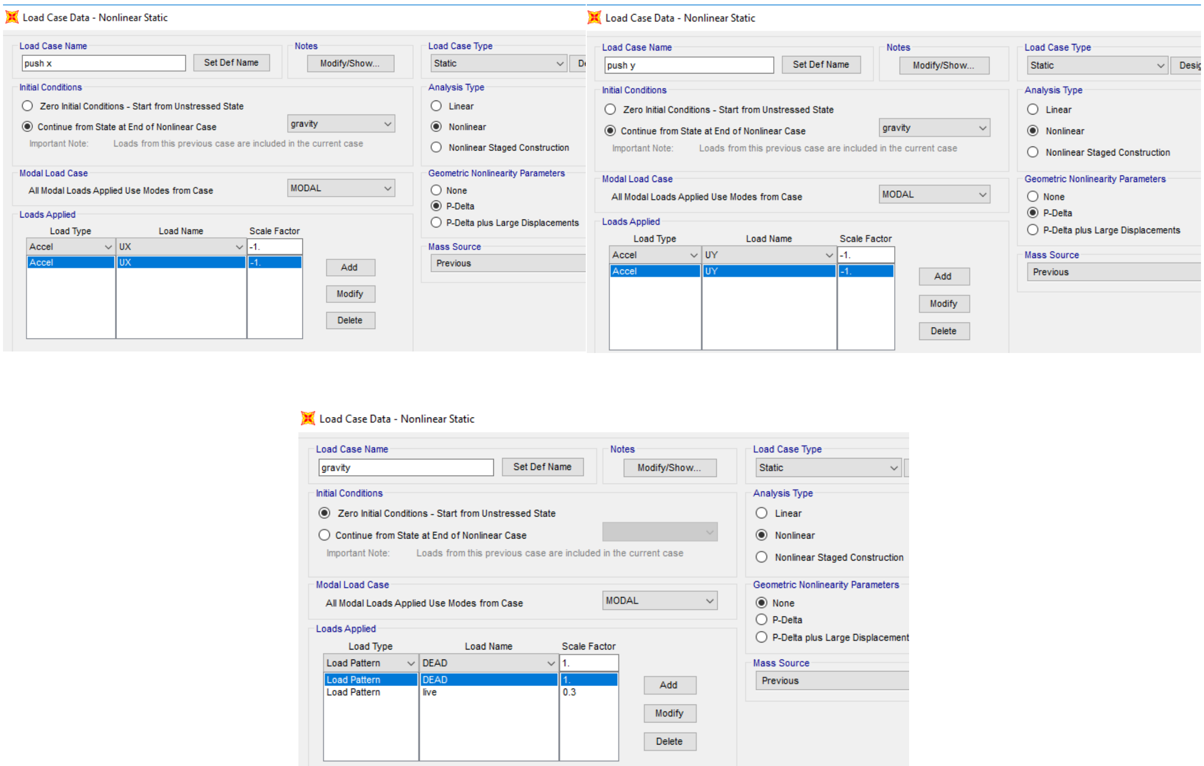


Fig 77. Non-linear static load cases.

Define frame hinges.

One of the main steps to perform pushover nonlinear analysis is to assign the plastic hinges in the structural model.

Sap 2000 gives default-hinge properties and recommend PMM hinges for columns and M3 hinges for beams as described in FEMA – 356. The flextural default hinges M3 were assigned to the beam at two ends. The interacting (P-M2-M3) frame hinges type, a coupled hinge property is also assigned for all columns at upper and lower ends .

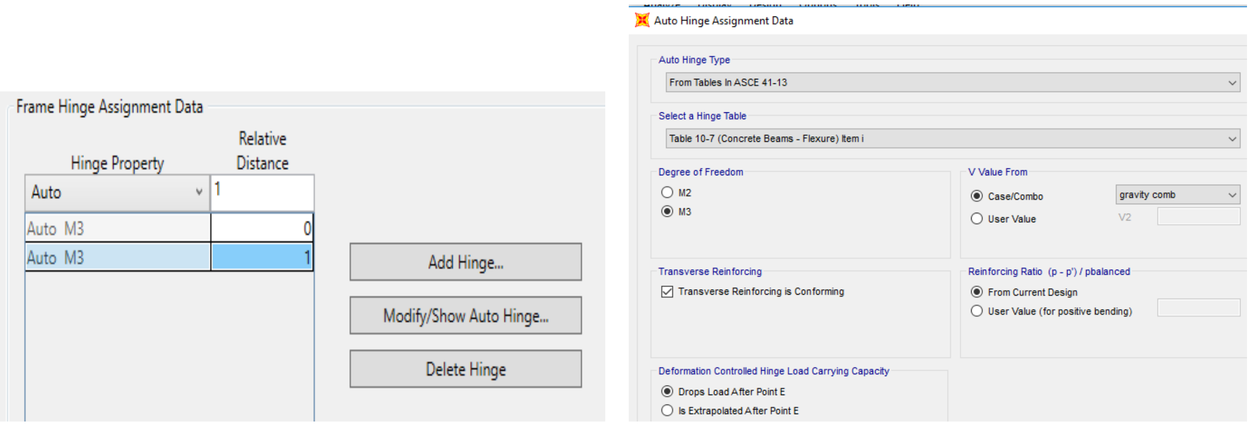


Fig 78. Beam hinges assignment

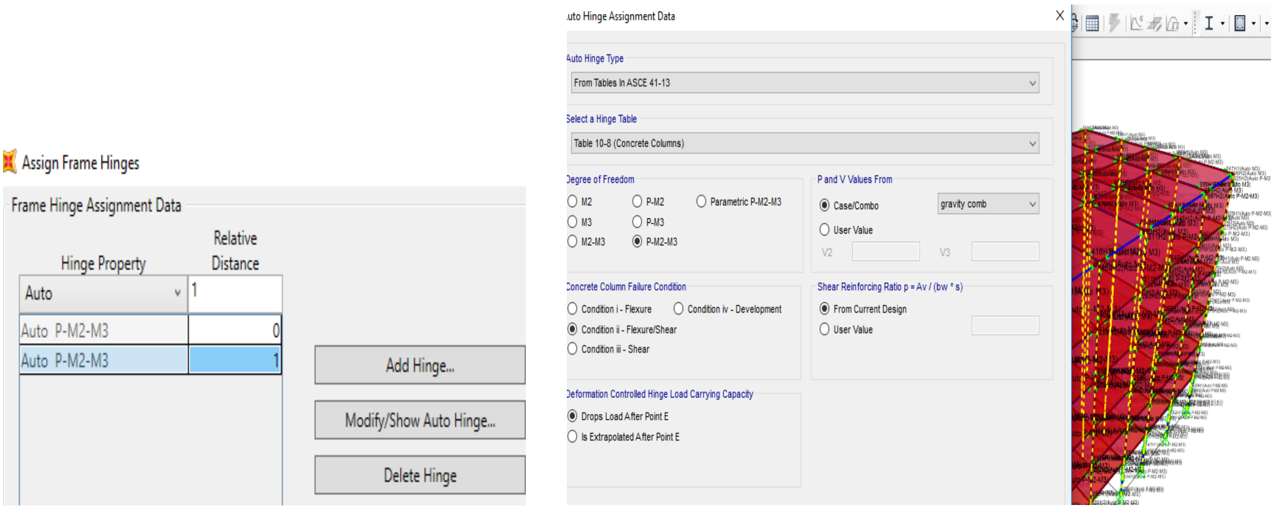
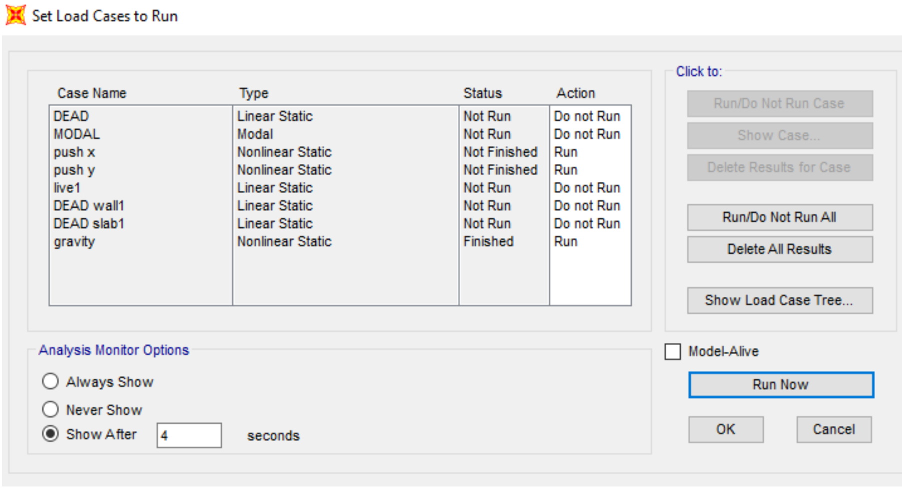


Fig 79. Column hinges assignment.

Run non-linear analysis. (REF building)



After assigning the building hinges we do run analysis and display the deformed shapes for both push X and push Y in their respective maximum steps. Immediately there are displayed even the plastic hinges, starting from lower floor members in the first steps and continuing with the formation of hinges in upper elevations. Each of them, indicated by respective colors shows the building stiffness performance.

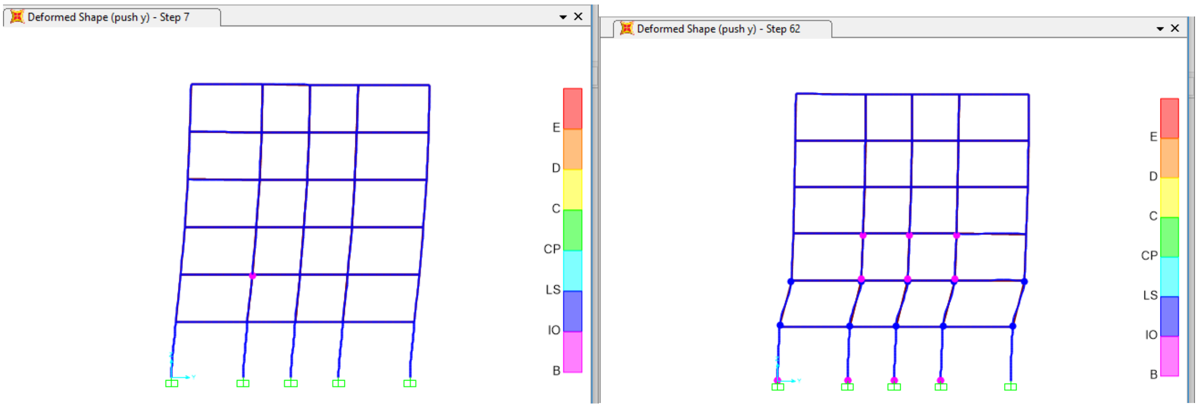


Fig 80. Plastic hinges appearing firstly in Step 7 and reaching maximum in step 62, PUSHY.

The range of formation of hinges tells us that collapse occurs in beams earlier than columns. According to the philpsophy of strong column- weak beam, makes us lead to the fact that this failure mechanism is satisfied.

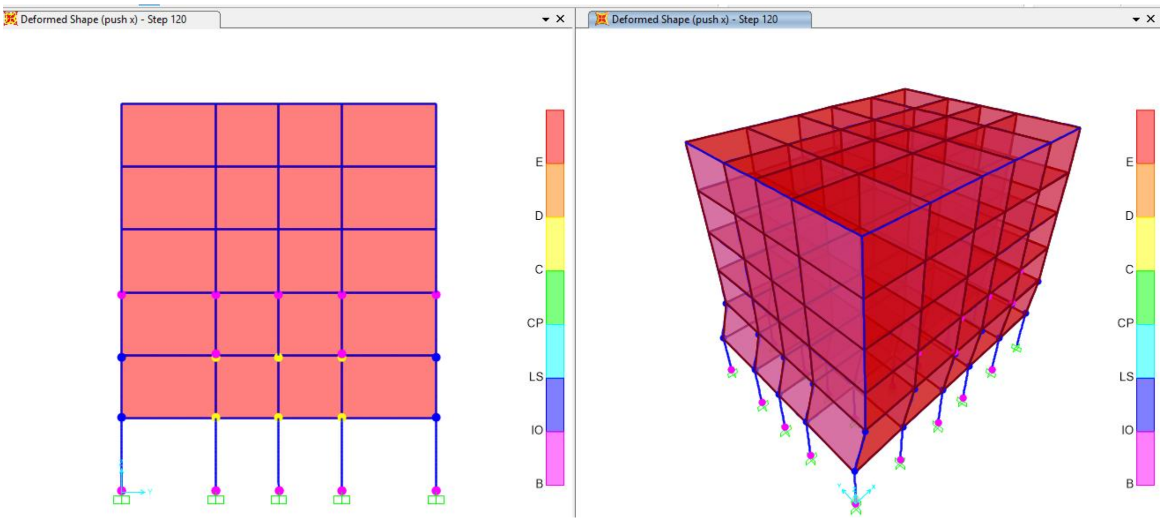


Fig 81. Deformed shape and Plastic hinges formation under Push X, maximum step.

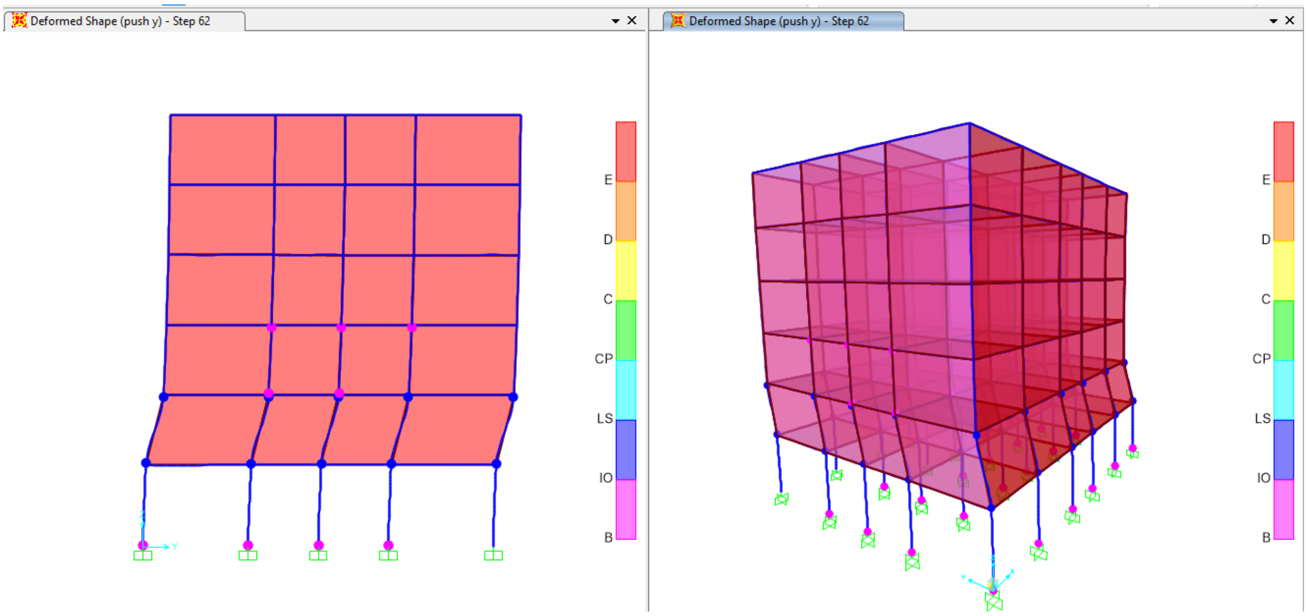


Fig 82. Deformed shape and plastic hinges formation under Push Y.

Display Hinge results. (REF building)

Hinge results are displayed with their respective values.

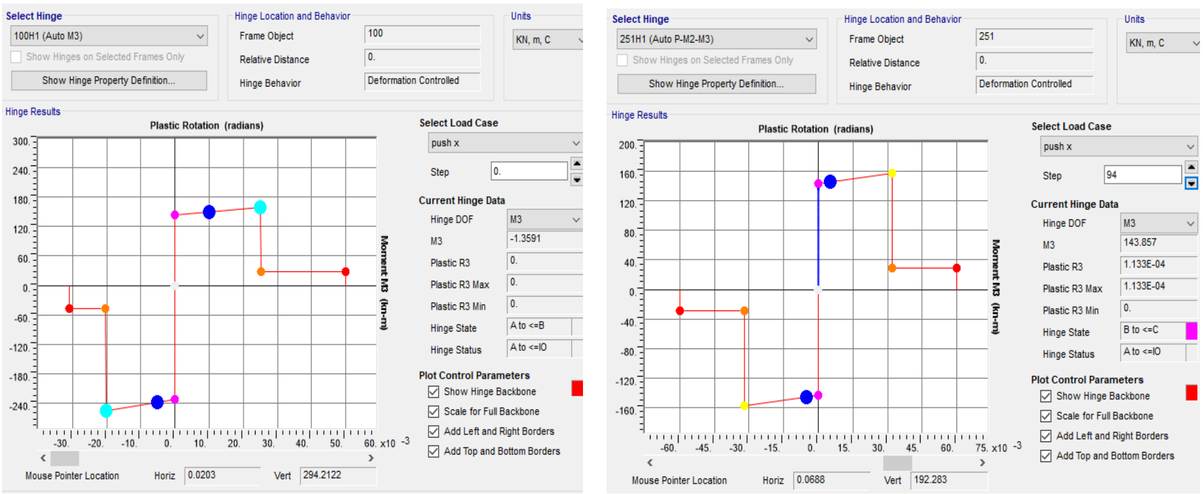


Fig 83. Hinges result for beam and column in pushX.

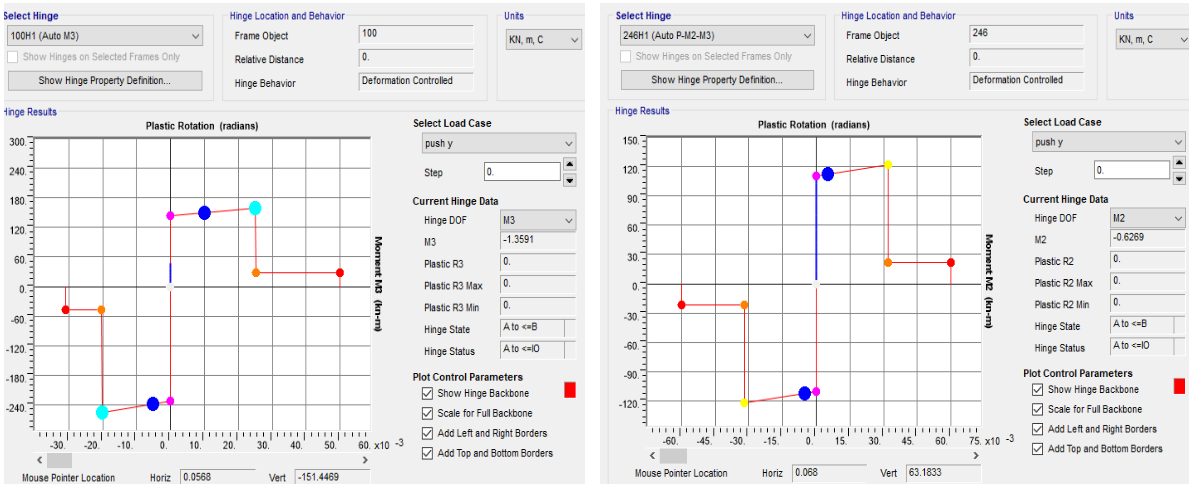


Fig 84. Hinges result for beam and column in pushY.

The results for formation of plastic hinge and stage of hinge at different levels of building performance are given through pushover analysis. The data gives information about the weak element in the structure. For getting a clear understanding about the formation of hinges at each step of displacement, higher value of multiple steps is selected.

Display Pushover Static Curves for REF building.

Pushover curve, is a plot of base shear vs displacement of the structure. The pushover curve for RC frame building is analysed both in in X and Y direction.

Representing capacity curve under Displacement-Base Reaction parameters.

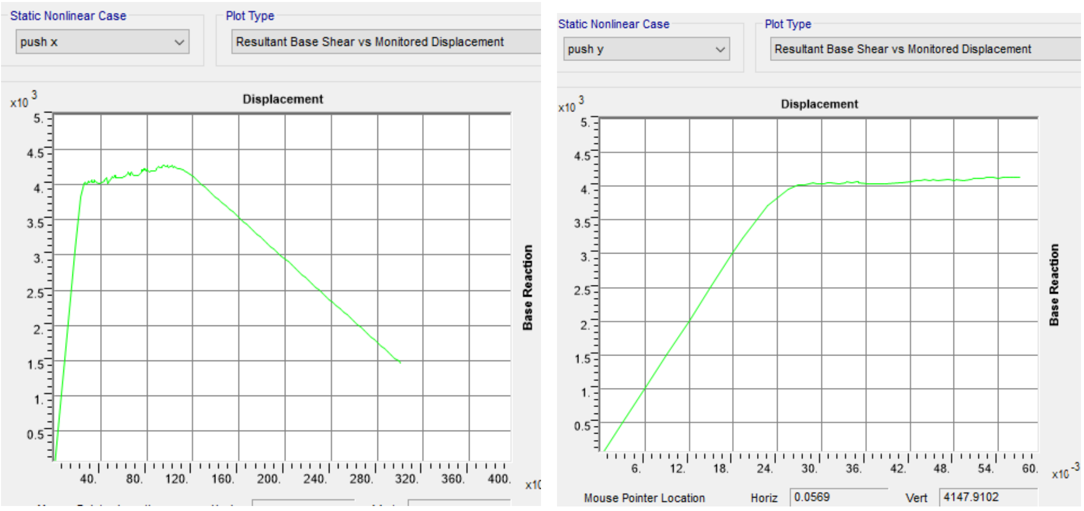


Fig 85. Resultant Base shear vs Monitored Displacement in both cases, push x and push y.

Representing capacity curve under ATC-40 Capacity spectrum.

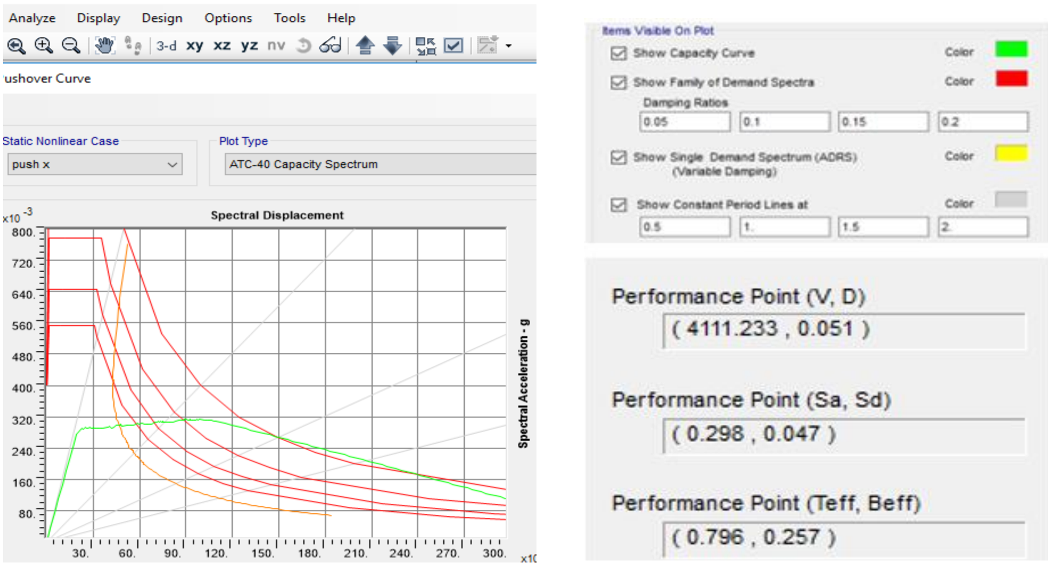


Fig 86. Displayed curve in case of spectral displacement- acceleration-g, Pushx.

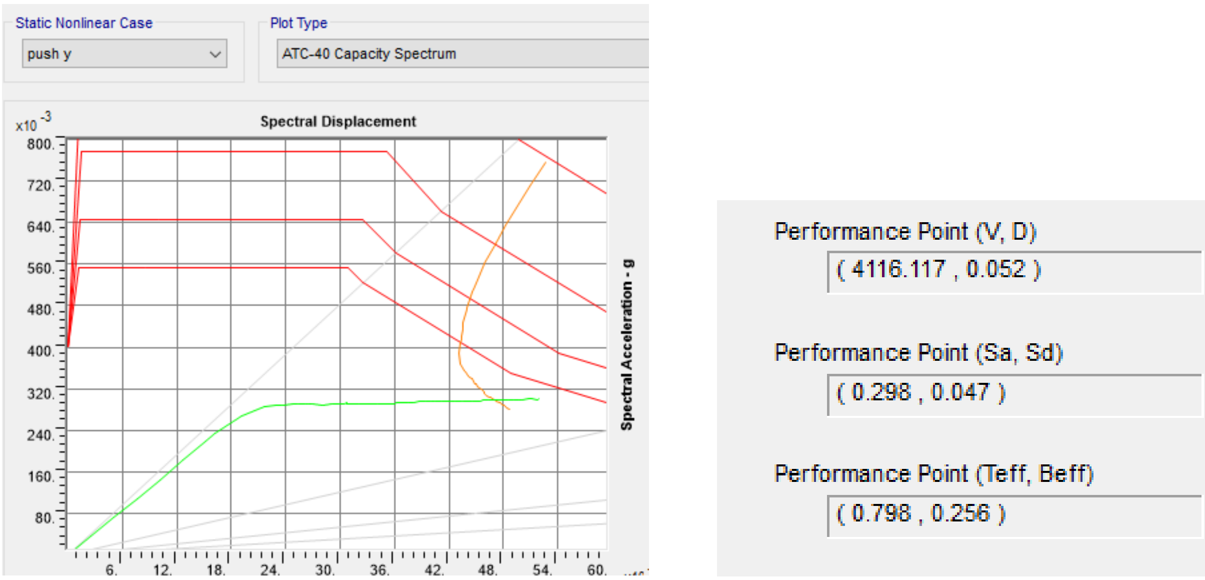


Fig 87. Displayed curve in case of spectral displacement- acceleration-g, Pushy.

Displaying Capacity Curve for FEMA 356- Coefficient Method.

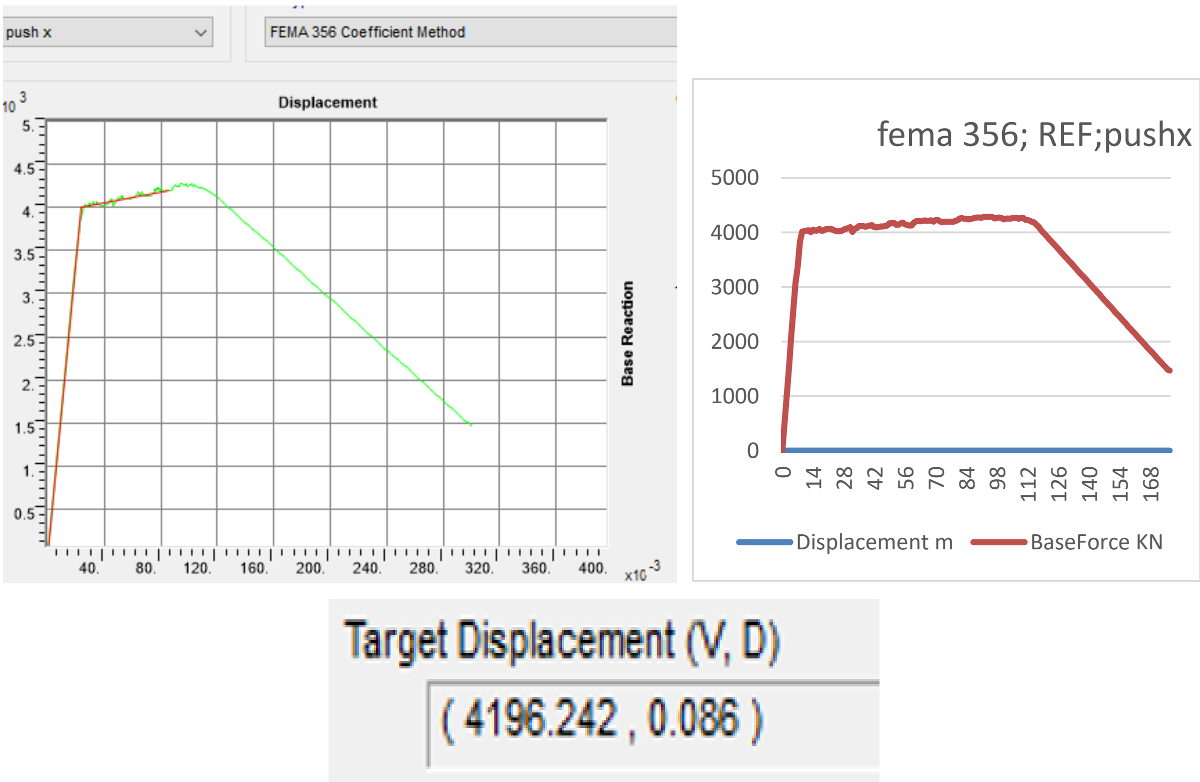


Fig 88. Displayed curve for FEMA – 356 coefficient method. Push X

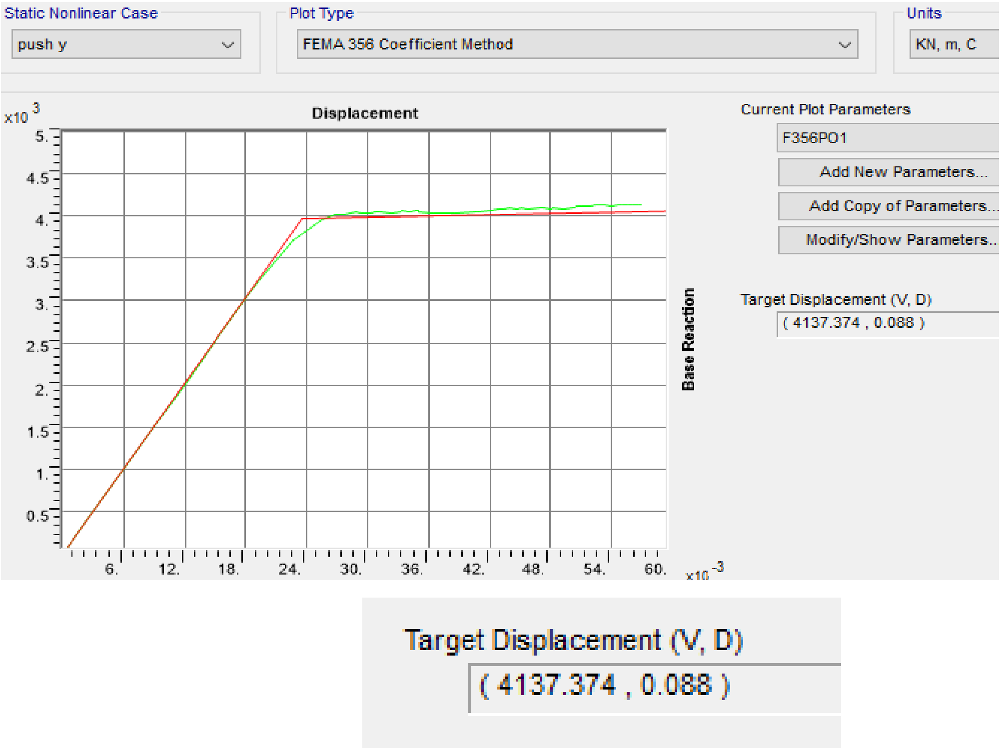


Fig 89. Displayed curve for FEMA – 356 coefficient method. Push Y.

Displaying capacity curve for FEMA 440- Displacement Modification.

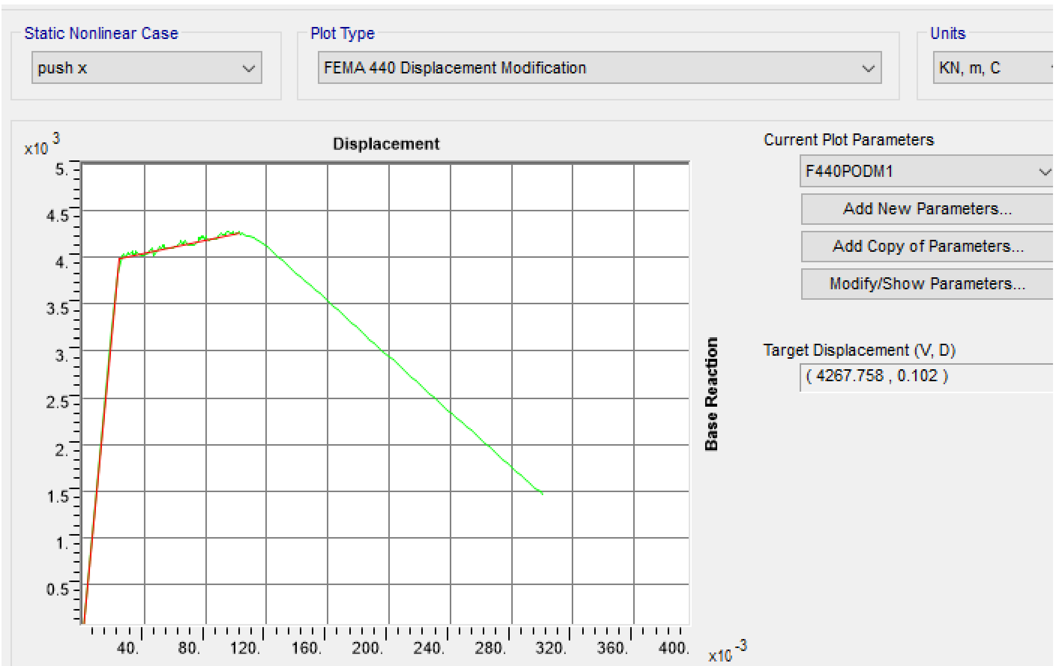


Fig 90. Curve of Base Reaction-Displacement. Push X.

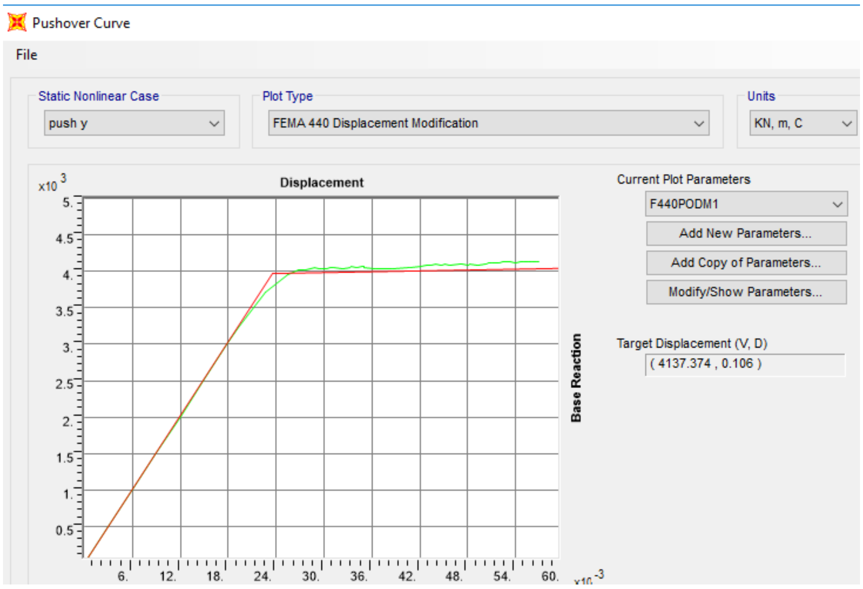


Fig 91. Curve of Base Reaction-Displacement. Push Y.

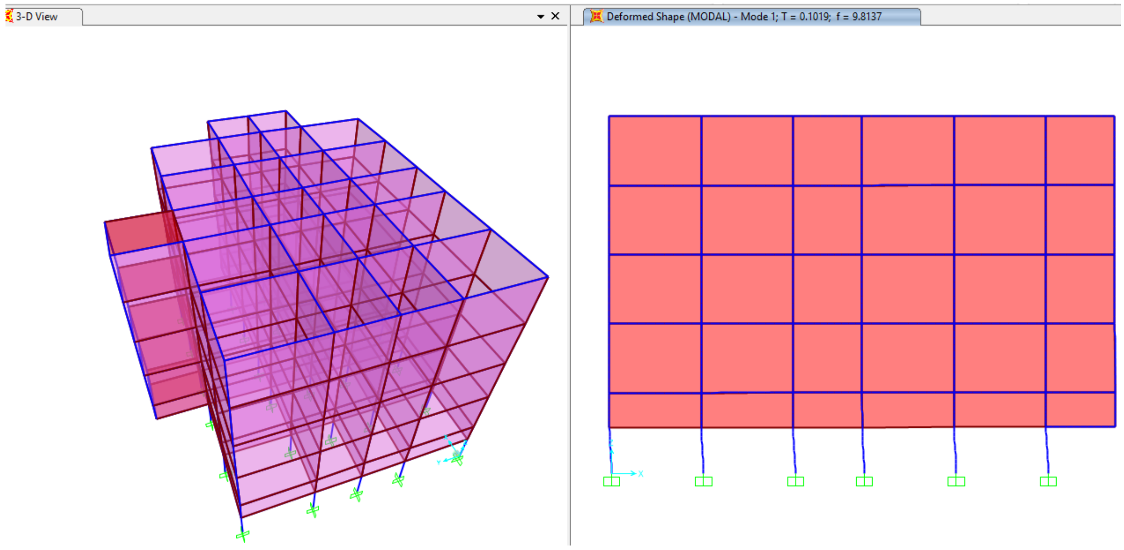
4.2.2 Analysis for modified frames with irregularities.

As previously mentioned in this thesis our building will be modified to present most common irregularities of RC frame buildings, such as: Soft story due to height, soft story due to height and semi infilled frames which also consist in formation of short columns. Presence of floor addition specified as heavy overhang one sided, also two sided overhangs. For each of these cases the pushover analysis will take place and the capacity curves will be displayed with respective values and performance points.

Case 1.

The structure below is modified with respect to the above mentioned irregularities. The ground story is designed to be 3.5 meter in height and above it there's a 2 m height intermediate story due to many commercial reasons and human interventions done. The difference in height changes the load path and the presence of new members, such as short columns takes place.

SHC	**SHORT COLUMN DUE TO SEM-INFILLED BAYS.**
SSHITSO	**Soft story due to both height and infill, and two sided overhang.**



After assigning all required parameters for loads, frame sections and material as done for Reference Frame we model the irregularities and display the deformed shape for modal analysis. The table below gives the values of period and frequencies performed in each modal step.

```
Original stiffness at shift : EV= 0.0000000E+00, f=     .000000, T=  -INFINITY-
Number of eigenvalues below shift =       0
Found mode       1 of     12:  EV= 3.8021139E+03, f=    9.813698, T=    0.101898
Found mode       2 of     12:  EV= 4.4349268E+03, f=   10.598963, T=    0.094349
Found mode       3 of     12:  EV= 4.9513049E+03, f=   11.199019, T=    0.089294
Found mode       4 of     12:  EV= 2.0712173E+04, f=   22.905141, T=    0.043658
Found mode       5 of     12:  EV= 2.5252632E+04, f=   25.291434, T=    0.039539
Found mode       6 of     12:  EV= 3.4662016E+04, f=   29.631050, T=    0.033748
Found mode       7 of     12:  EV= 5.8790852E+04, f=   38.590020, T=    0.025913
Found mode       8 of     12:  EV= 6.9347933E+04, f=   41.911856, T=    0.023860
Found mode       9 of     12:  EV= 8.0206593E+04, f=   45.073903, T=    0.022186
Found mode      10 of     12:  EV= 8.1719361E+04, f=   45.496985, T=    0.021979
Found mode      11 of     12:  EV= 8.8599889E+04, f=   47.373636, T=    0.021109
Found mode      12 of     12:  EV= 1.0315526E+05, f=   51.117056, T=    0.019563

NUMBER OF EIGEN MODES FOUND                   =          12
NUMBER OF ITERATIONS PERFORMED                =           5
NUMBER OF STIFFNESS SHIFTS                    =           0
```

Table 6. Modal analysis for SHC-SSHITSO.

After assigning the hinges for all members, Beams (M3) and columns (P-M2-M3), we check for plastic hinges formation displayes with their respective colors. The plastic hinges are seen as follow in both cases of loads, pushX and Push Y.

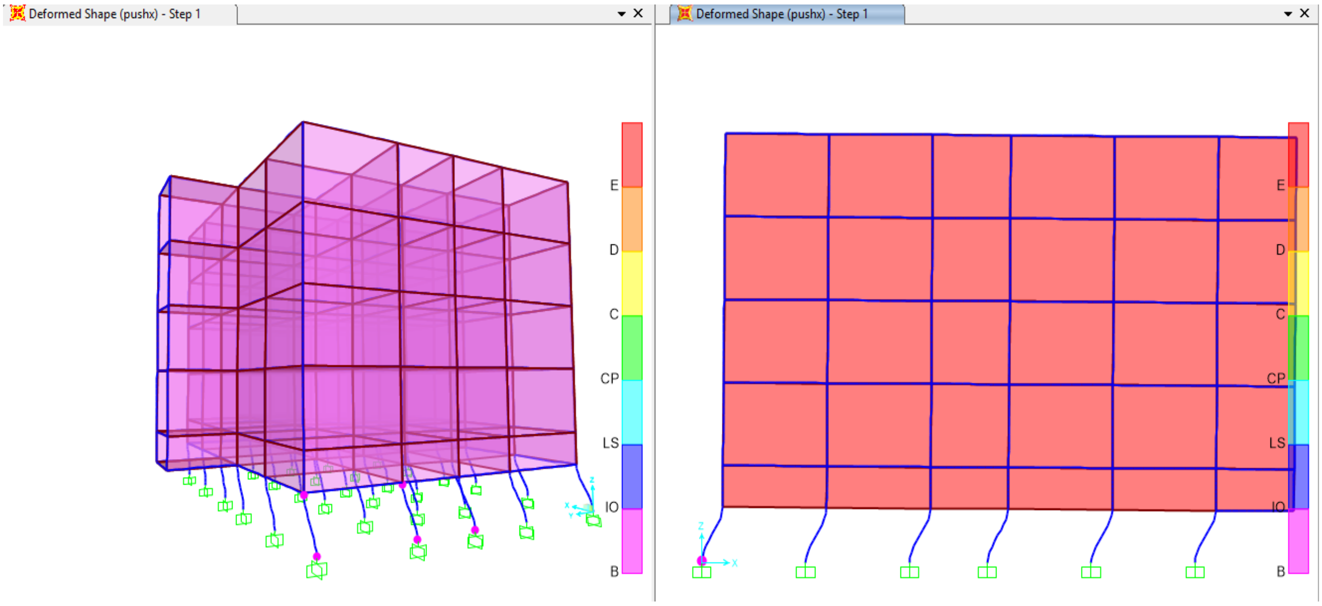


Fig 92. Deformed shape under Push X load case and plastic hinges formation step 1.

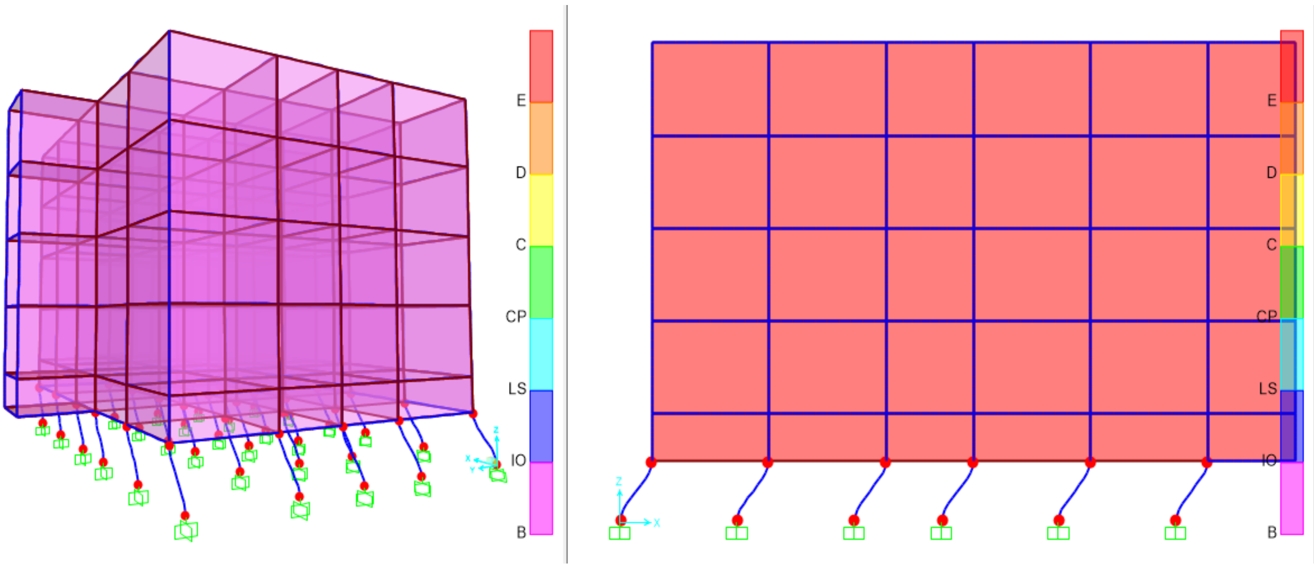


Fig 93. Deformed shape under Push X load case and plastic hinges formation maximum step.

Then, we also check the deformed shape and plastic hinges formation even for Load case Push Y.

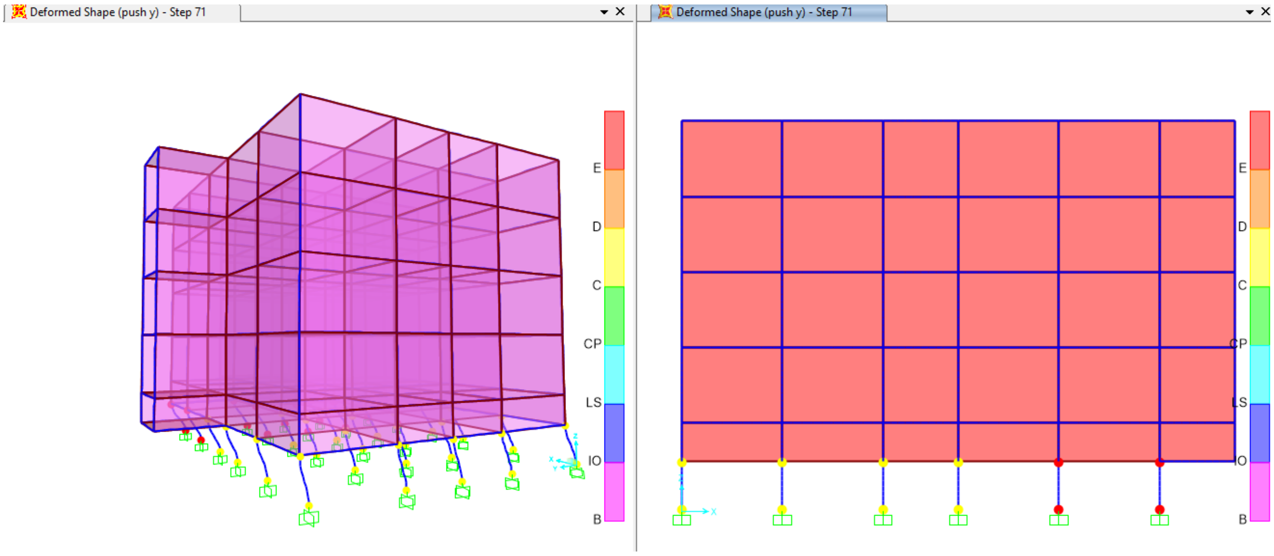


Fig 94. Deformed shape under Push Y load case and plastic hinges formation maximum step.

As seen above, the building performes large deformation in both load cases. Defined from plastic hinges formation, it is observed that the very first plastic hinges appear in the ground floor columns and than immediately at the short columns of interstory, semi infilled frame. This gives us the information of the weak stiffness in which ground story performes.

After that we run analysis for pushover curve results. The table displayed for values of performance in each step will be attached in appendix section.

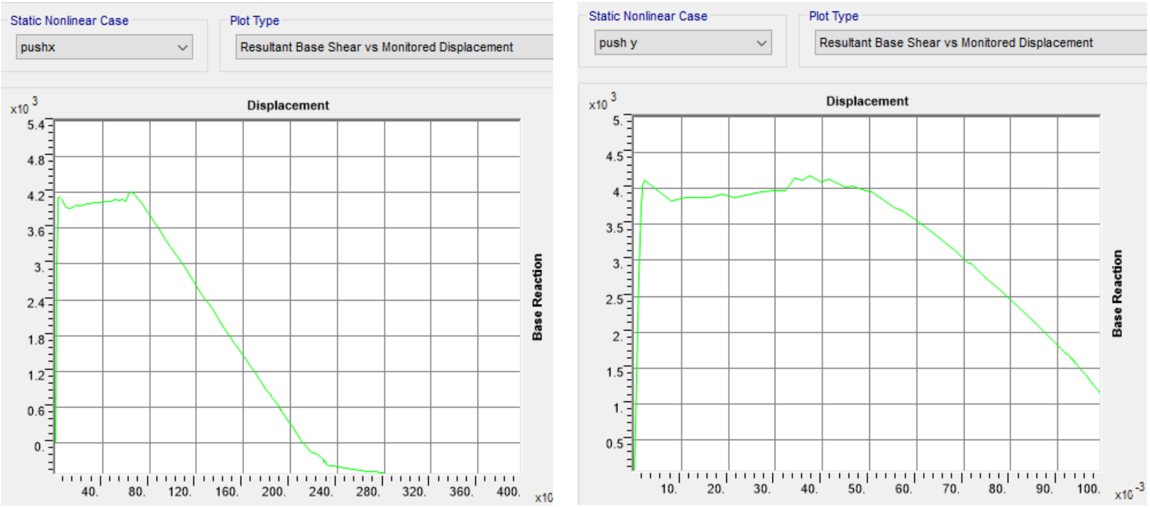


Fig 95. Base reaction - displacement for SHC-SSHITSO in Pushx and pushy.

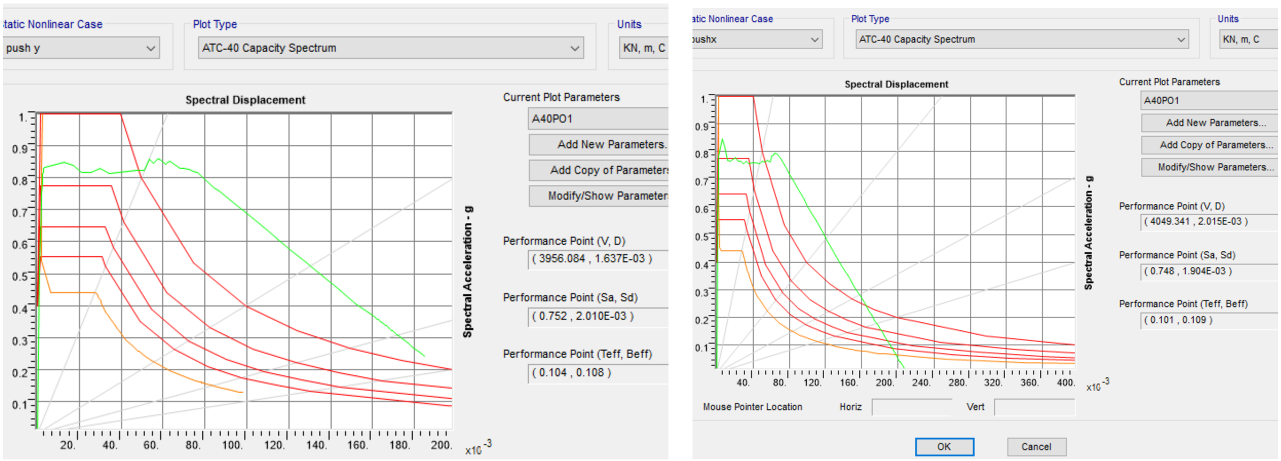


Fig 96. Capacity Spectrum ATC-40 for SHC-SSHITSO in Pushx and pushy.

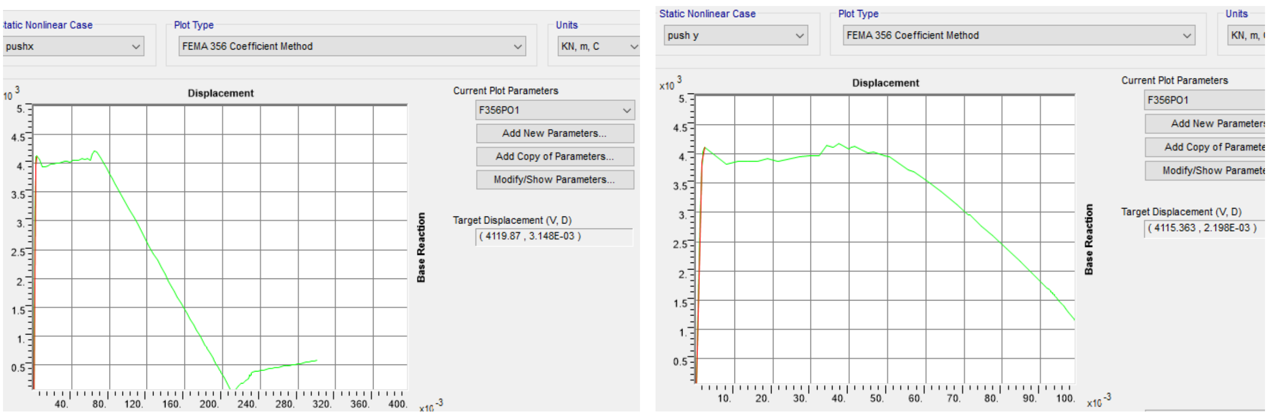


Fig 97. Capacity curve under FEMA 356-Coefficient Method for SHC-SSHITSO in Pushx and pushy

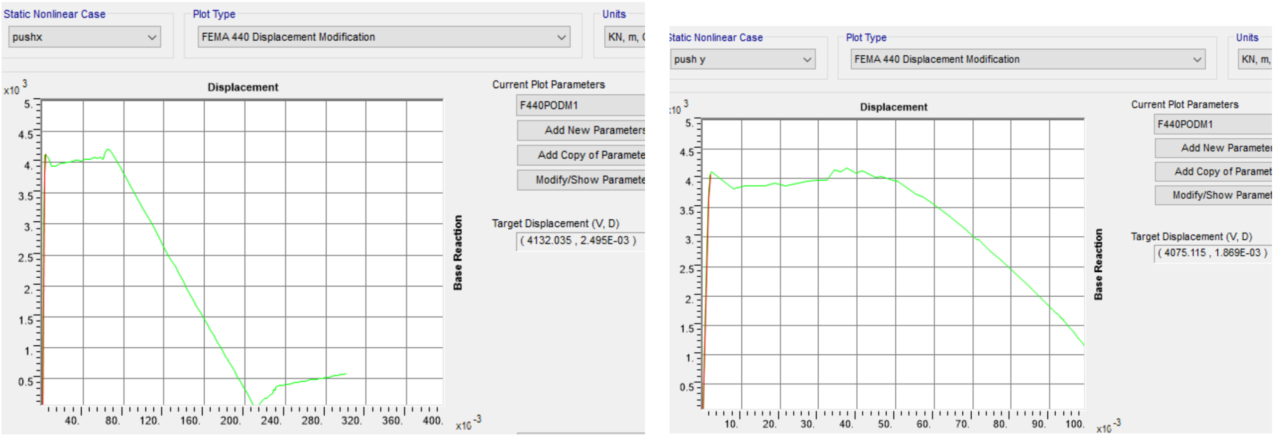


Fig 98. Capacity curve under FEMA 440 Displacement modification for SHC-SSHITSO in Pushx and pushy.

CASE 2.

SSHOSO **SOFT STORY DUE TO BOTH HEIGHT AND INFILL, AND ONE SIDED OVERHANG**

Check the deformed shape under modal analysis.

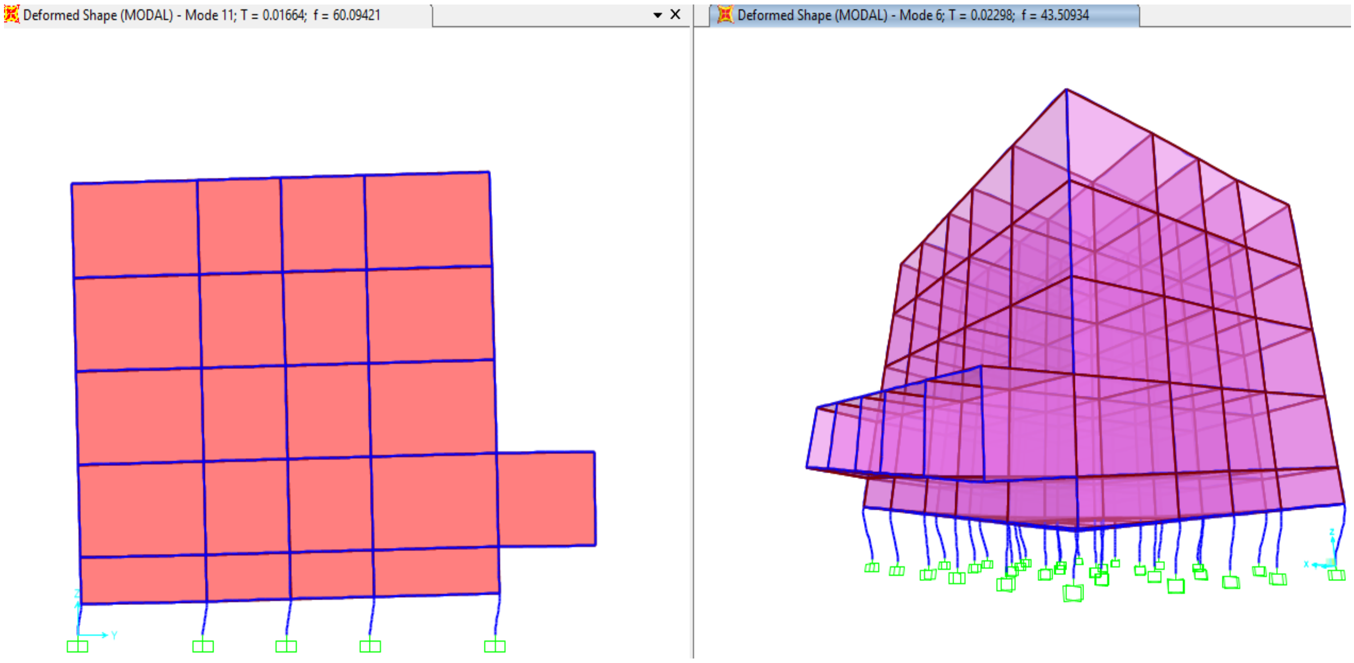


Fig 99. Deformed shape of elevation under mode 11 and 3D model under mode 6 for SSHOSO case.

When running the modal analysis, we also display the table showing the building performance in 12 steps, by defining period and frequence of SSHOSO case.

```
Original stiffness at shift : EV= 0.0000000E+00, f=     .000000, T=  -INFINITY-
Number of eigenvalues below shift =       0
Found mode       1 of      12:  EV= 4.5378203E+03, f=   10.721209, T=    0.093273
Found mode       2 of      12:  EV= 4.7191889E+03, f=   10.933364, T=    0.091463
Found mode       3 of      12:  EV= 5.2804673E+03, f=   11.565284, T=    0.086466
Found mode       4 of      12:  EV= 2.3070728E+04, f=   24.174126, T=    0.041367
Found mode       5 of      12:  EV= 3.6583610E+04, f=   30.441315, T=    0.032850
Found mode       6 of      12:  EV= 7.4735102E+04, f=   43.509335, T=    0.022984
Found mode       7 of      12:  EV= 7.6824942E+04, f=   44.113473, T=    0.022669
Found mode       8 of      12:  EV= 9.7495537E+04, f=   49.694978, T=    0.020123
Found mode       9 of      12:  EV= 1.0274144E+05, f=   51.014421, T=    0.019602
Found mode      10 of      12:  EV= 1.0484190E+05, f=   51.533254, T=    0.019405
Found mode      11 of      12:  EV= 1.4256899E+05, f=   60.094215, T=    0.016641
Found mode      12 of      12:  EV= 2.9607377E+05, f=   86.600440, T=    0.011547

NUMBER OF EIGEN MODES FOUND                      =          12
NUMBER OF ITERATIONS PERFORMED                   =           7
NUMBER OF STIFFNESS SHIFTS                       =           0
```

Table 7. Modal analysis for SSHOSO case.

Then, we assign hinges in every member and run for non-linear static analysis, through which we will provide results for plastic hinges formation and capacity curves.

- *Plastic-hinges formation.*

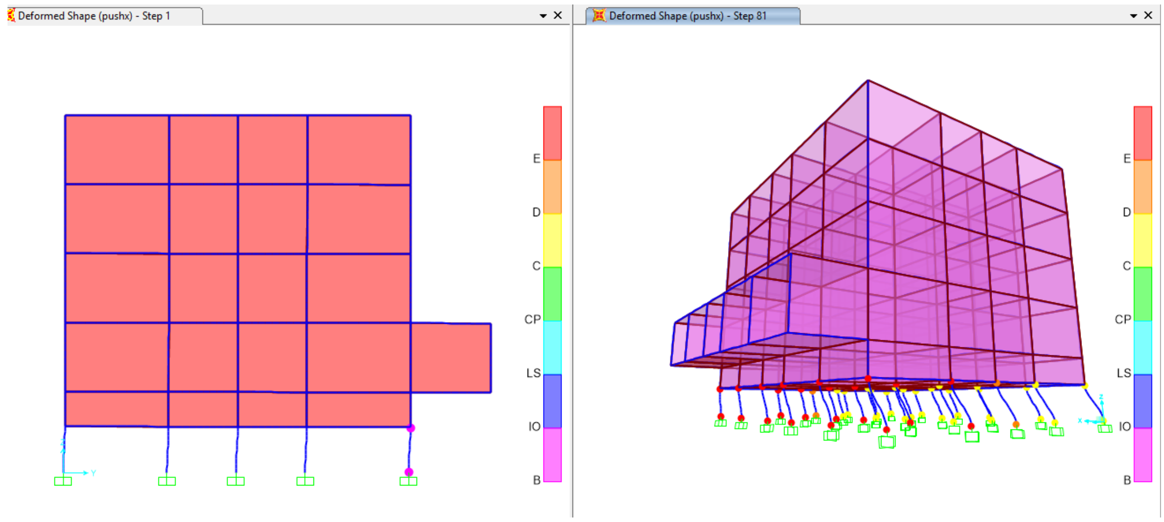


Fig 100. Plastic hinges formation for SSHOSO case in push x.

As seen form the models the first plastic hinges tend to appear immediately at the ground story under the overhang members side, representing the range from unloaded state (A) to its effective yield (B). Then, with the increasing of steps the columns of ground story performes weaker and tending to a total loss of resistance.

- *Display pushover curves.*

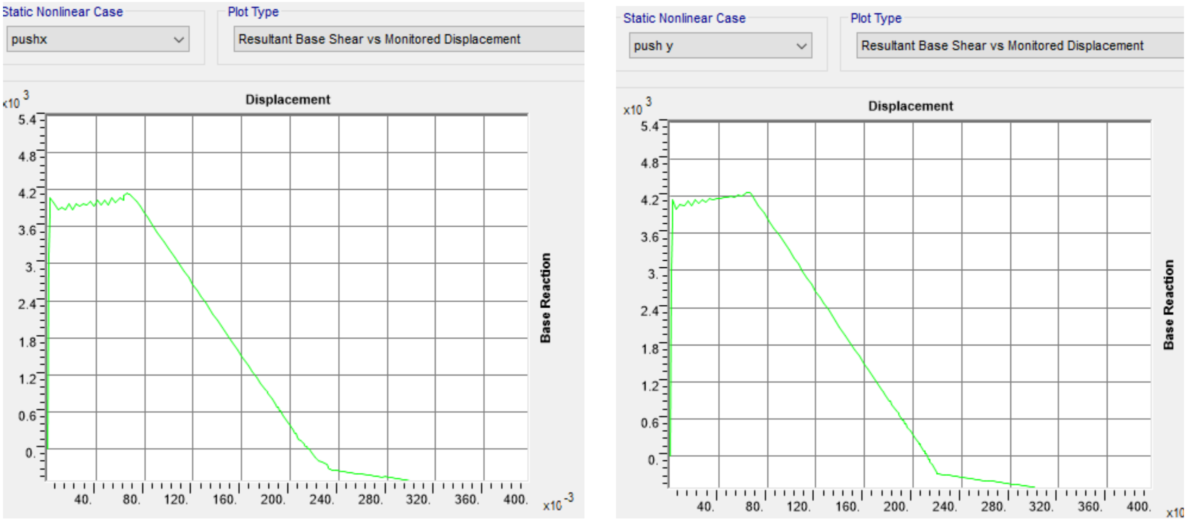


Fig 101. Displayed capacity curve, resultant base shear vs Monitored Displacement for push x and push y, SSHOSO case.

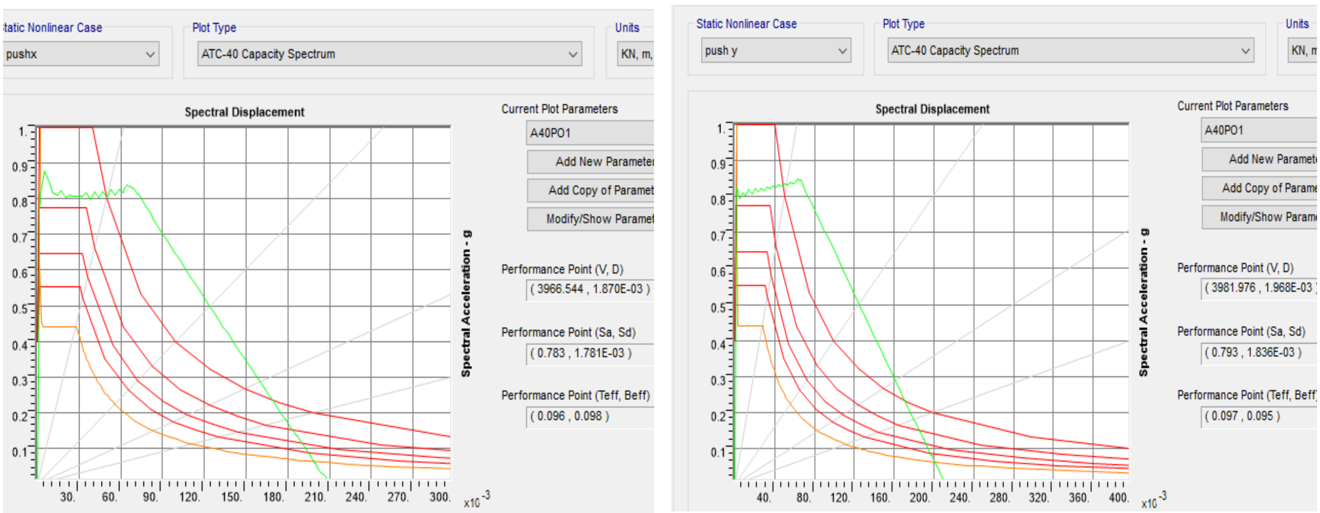


Fig 102. Displayed Atc-40 capacity spectrum for push x and push y, SSHOSO case.

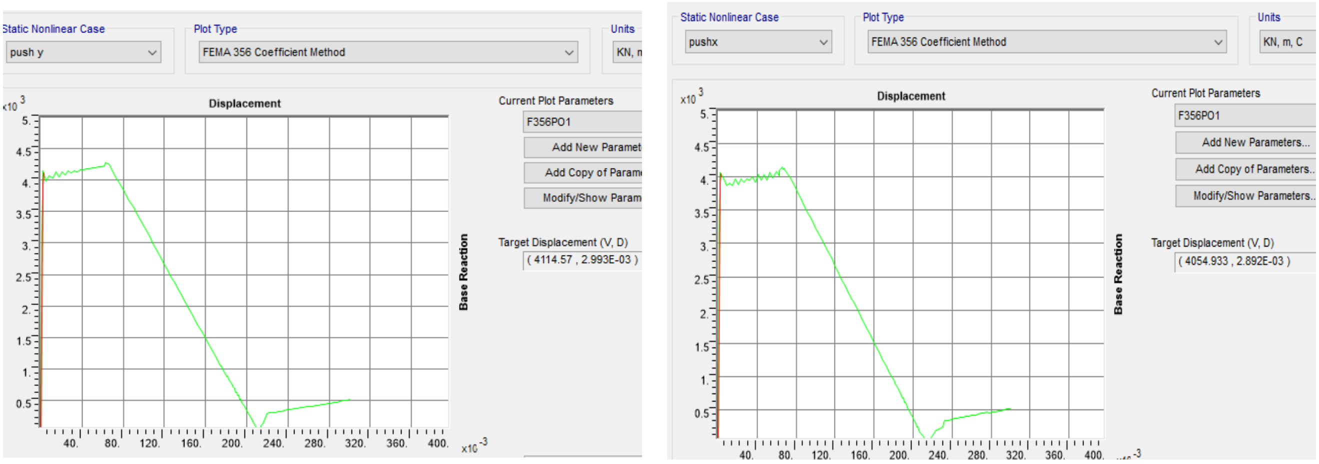


Fig 103 . Displayed Fema 356 Coefficient Method curve in push x and push y, SSHOSO case.

CASE 3.

SSWTSO **SOFT STORY DUE TO ABSENCE OF MASONRY INFILL WALL AT GROUND STORY AND TWO SIDED OVERHANG**

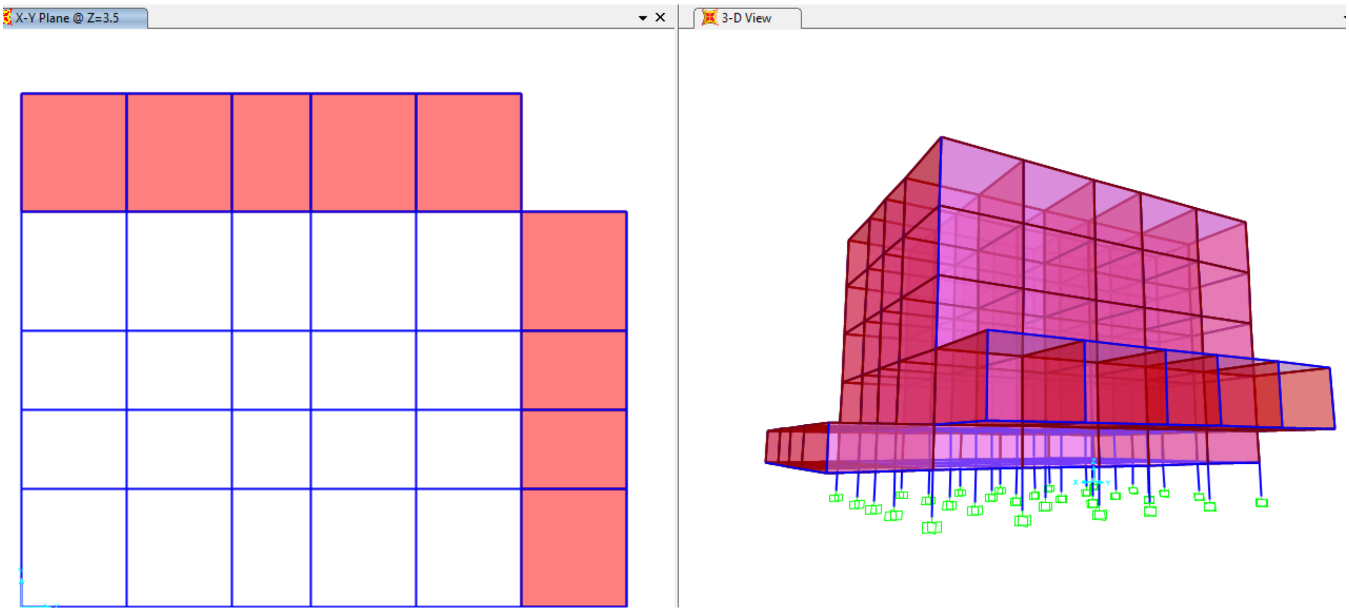


Fig 104. Undeformed shape of plan and 3D SSWTSO case model

Run for modal analysis.

When running the modal analysis, we also display the table showing the building performance in 12 steps, by defining period and frequence of SSWTSO case.

```
Original stiffness at shift : EV= 0.0000000E+00, f=    .000000, T=  -INFINITY-
Number of eigenvalues below shift =       0
Found mode      1 of     12:  EV= 8.9986950E+02, f=   4.774302, T=    0.209455
Found mode      2 of     12:  EV= 1.0343447E+03, f=   5.118619, T=    0.195365
Found mode      3 of     12:  EV= 1.1230009E+03, f=   5.333474, T=    0.187495
Found mode      4 of     12:  EV= 1.8730597E+04, f=  21.781909, T=    0.045910
Found mode      5 of     12:  EV= 4.3518775E+04, f=  33.201558, T=    0.030119
Found mode      6 of     12:  EV= 4.8118329E+04, f=  34.912054, T=    0.028643
Found mode      7 of     12:  EV= 5.6912289E+04, f=  37.968475, T=    0.026338
Found mode      8 of     12:  EV= 8.5674763E+04, f=  46.585052, T=    0.021466
Found mode      9 of     12:  EV= 8.6480935E+04, f=  46.803714, T=    0.021366
Found mode     10 of     12:  EV= 1.3133651E+05, f=  57.678355, T=    0.017338

Forming stiffness, new shift: EV= 2.5747867E+05, f=  80.758968, T=    0.012383
Number of eigenvalues below shift =      10
Found mode     11 of     12:  EV= 2.7102537E+05, f=  82.856219, T=    0.012069

Forming stiffness, new shift: EV= 3.2420675E+05, f=  90.621481, T=    0.011035
Number of eigenvalues below shift =      11
Found mode     12 of     12:  EV= 3.2771749E+05, f=  91.110817, T=    0.010976

NUMBER OF EIGEN MODES FOUND              =          12
NUMBER OF ITERATIONS PERFORMED           =          25
NUMBER OF STIFFNESS SHIFTS               =           2
```

Table 8. Modal analysis fro SSWTSO case.

Then, we assign hinges in every member and run for non-linear static analysis, through which we will provide results for plastic hinges formation and capacity curves for both push x and push y load case.

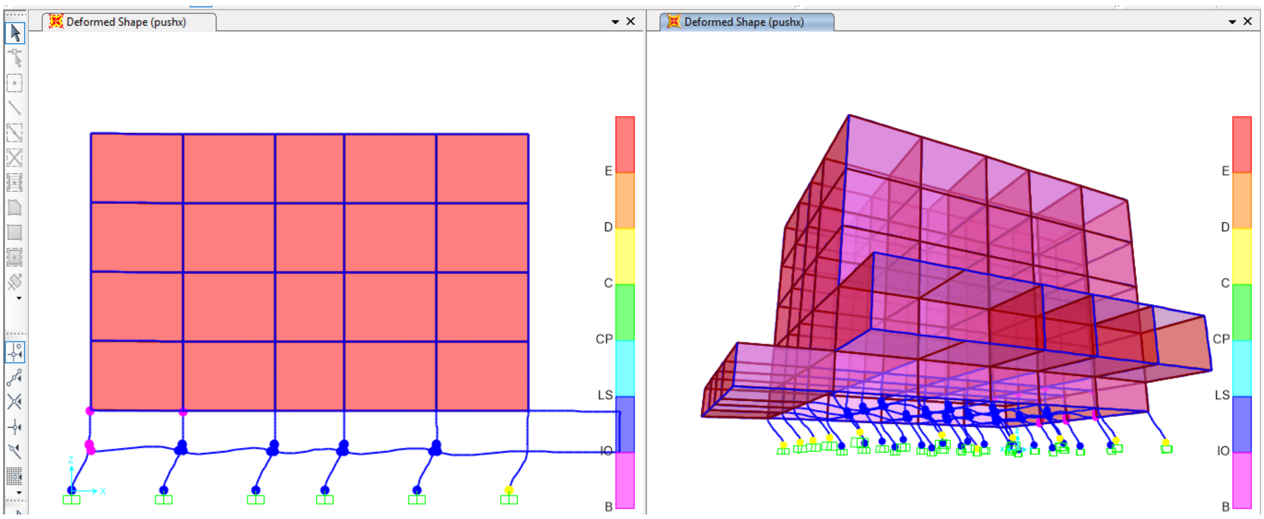
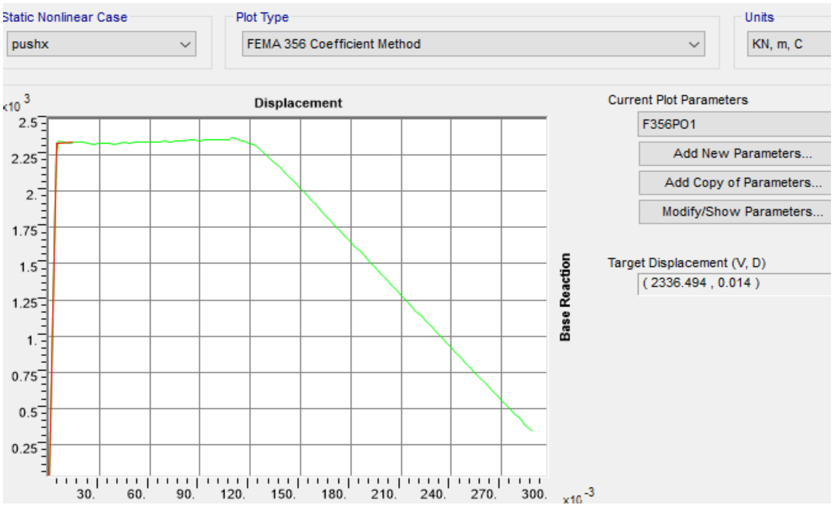


Fig 105. Plastic hinges formation for SSWTSO case in push Y.

As seen form the models the first plastic hinges tend to appear immediately at the ground story and specially at the floor where there are no partition walls, representing the range from unloaded state (A) to its effective yield (B). Then, with the increasing of steps the columns of ground story performes weaker and followed by an inelastic but linear response of reduced stiffness from B to C with respective colors, until C –D which shows a sudden reduction in load resistance, followed by a reduced resistance from D to E.

Display Pushover Curve for SSWTSO, FEMA 356 coefficient method both in push y and push x with respective values in tables attached.



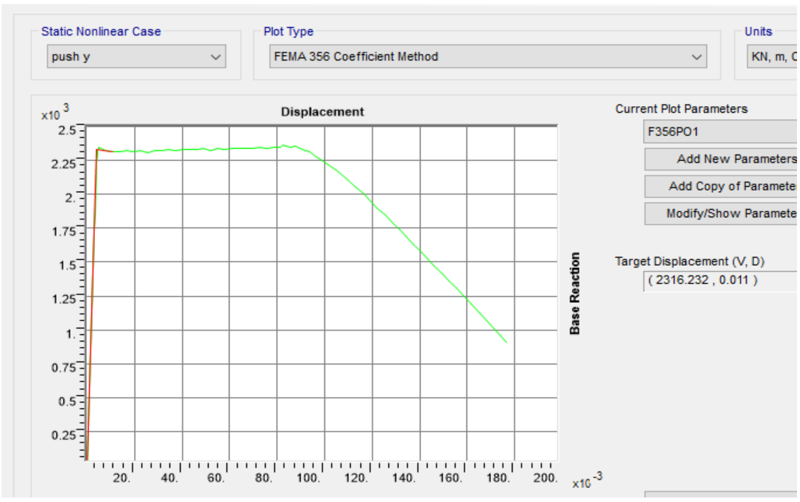


Fig 106. Base reaction- displacement curve for SSWTSO in pushx and pushy.

CHAPTER 5

Results and discussions

5.1 Analysis Results.

A six story RC frame structure represented as reference building and then modified under certain structural irregularities was analysed. The frame was subjected to design load cases in linear and non-linear terms. Each frame pushover curves for the buildings in X direction and Y direction were displayed following the attached results for every step of evaluation. These curves present the behavior of frames in aspects of stiffness and of ductility. Maximum base shear from pushover analysis and displacement values are shown in each model frame of this study. There's also displayed Capacity spectrum analysis, which is the capacity curve of spectral accelerations VS, and spectral displacement (Sa, Vs, Sd) coordinates for each case building in both pushx and push y load. The performance point is obtained by capacity curve and base shear-displacement curve. The capacity curves show unique features for each case. In common they begin to appear linar direction but then they perform deviation as a result of beams and columns undergoing inelastic behaviour. From the graphs it is seen that the presence of soft story irregularity both weakness and softens the frame. Also the presence of overhangs when displayed in baseshear-deformation curve seems to perform lower capavity in comparison with the REF frame. What else observed is that from the deformation ratios presented in tables for SSWTSO it is obviously seen that presence of soft story because of absence of masonry-infill-walls at ground level is more dangerous and deformed shaped than the case of the presence of soft story because of higher ground story height.

The worst case observed is that of presence of soft story due to height irregularity, open ground story with absence of masonry partition walls and two sided overhanged (SSWTSO).

Following there will be a comparison between modified building cases and reference building. The values (push x load and push y) obtained from modal analysis and FEMA 356 are chosen to be the ones for illustrating the comparison.

- **FROM MODAL ANALYSIS STEP-1.**

Model case	***step***	***frequency***	***period***
Reference building	1	1.71	0.583816
SHC-SSHITSO	1	9.81	0.101898
SSHOSO	1	10.72	0.091463
SSWTSO	1	4.77	0.209455

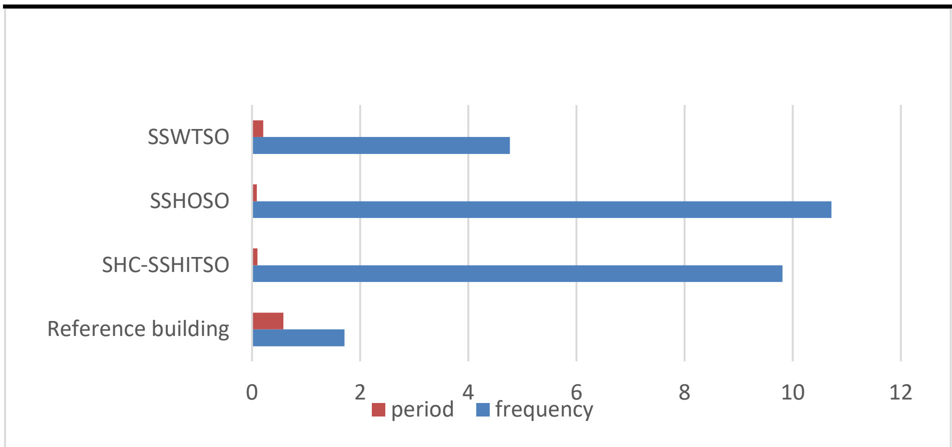


Fig 107. Comparison among models in modal analysis, step-1

TARGET DISPLACEMENT.

The magnitude of force increases until the structure achieves the target displacement. It represents the top displacement of a structure when subjected to design level ground excitation. This way it determined the performance criteria of a building.

There are shown the target displacement values provided by pushover curve, fema 356. It is shown or forth model cases, both in X and Y push.

From values provided we can see that the Reference building is the one requiring more force to displace and reaching a large displacement criterion which provides a favorable failure mechanism with reasonable stiffness and good post-yield deformability. We will better see these while pushover curves appearing.

- **TARGET DISPLACEMENT FROM FEMA 356-PUSHX.**

Model case	*Force (KN)*	*Displacement (m)*
REF. Building	4196.242	0.086
SHC-SSHITSO	4119.87	0.031
SSHOSO	4054.933	0.028
SSWTSO	2336.494	0.014

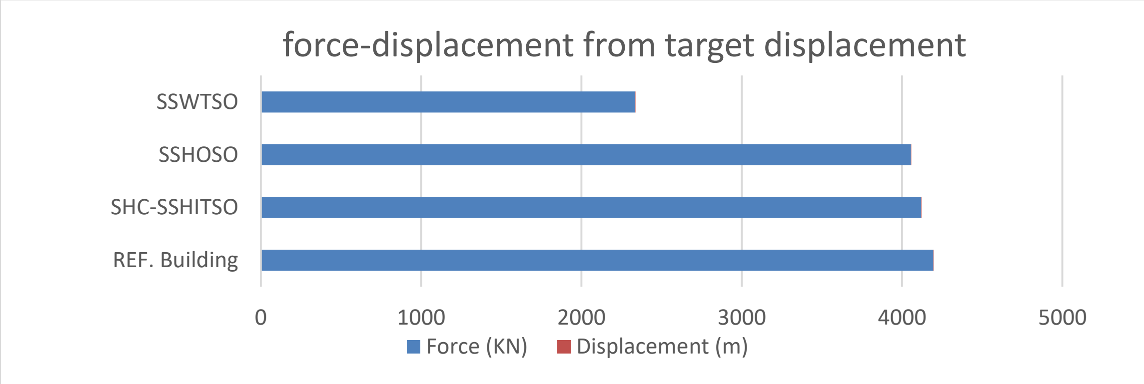


Table 9. Target displacement for all considered frames in pushX, fema 356.

- **TARGET DISPLACEMENT FROM FEMA 356-PUSHY.**

Model case	*Force (KN)*	*Displacement (m)*
REF. Building	4137.374	0.088
SHC-SSHITSO	4115.363	0.021
SSHOSO	4114.57	0.029
SSWTSO	2316.232	0.011

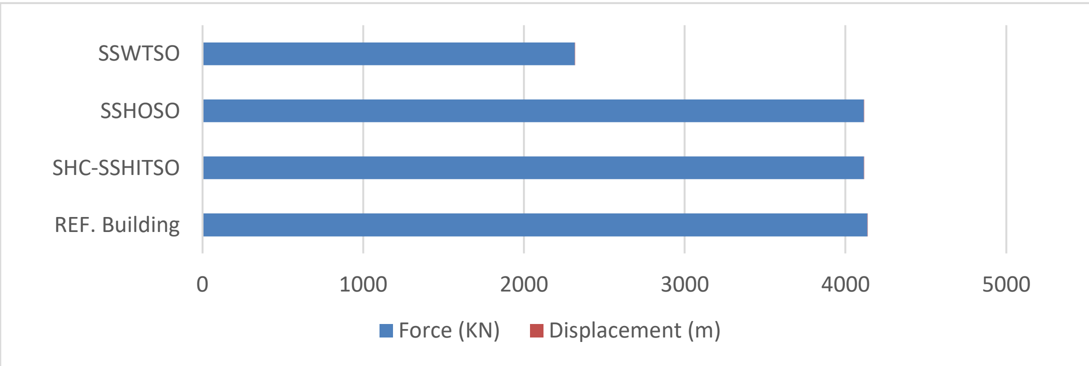


Table 10. Target displacement for all considered frames in pushY, fema 356.

- **COMPARISON BETWEEN REFERENCE CASE AND MODIFIED MODELS IN PUSH Y BY FEMA 356- COEFFICIENT METHOD DISPLAYED CURVE**.

Buildings are required to be designed and detailed such that they develop satisfactory failure mechanism that includes specified lateral strength, favorable stiffness and most important, good post-yield deformation capacity.

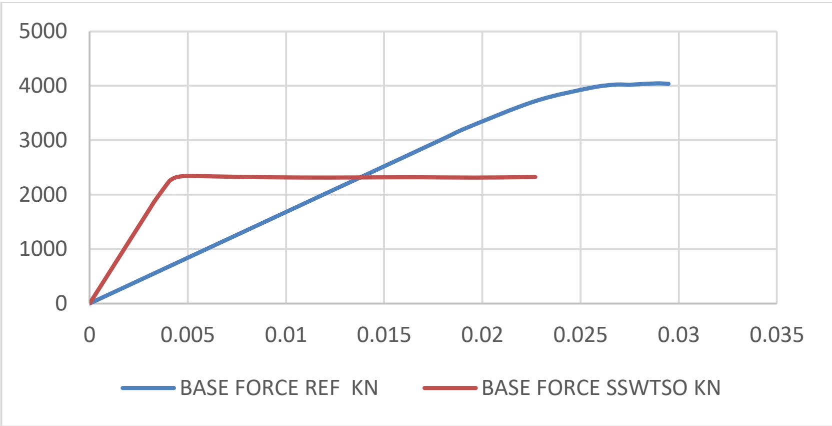


Fig 108. Fema 356 capacity curve for reference case vs sswtso, in push y load.

Comparing reference frame and the case of Soft story due to irregularity in height – absence of masonry infill walls and even more, two sided overhanged, results that reference frame shows better ductility, while SSWTSO performs earlier yield deformability. Currently: Reference case 2.9% and SSWTSO 2.2%

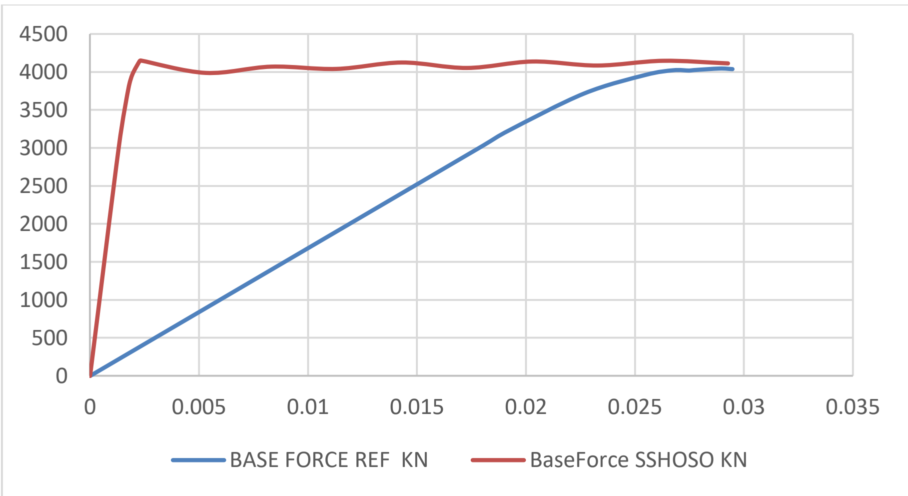


Fig 109. Fema 356 capacity curve for reference case vs sshoso, in push y load.

Considering the comparison between Ref case and the model case of Soft ground story – one sided overhanged (fig 109) results that, with small difference the reference case performs in better ductility. Currently with a displacement of 2.9% for reference frame, and 2.75% for SSHOSO.

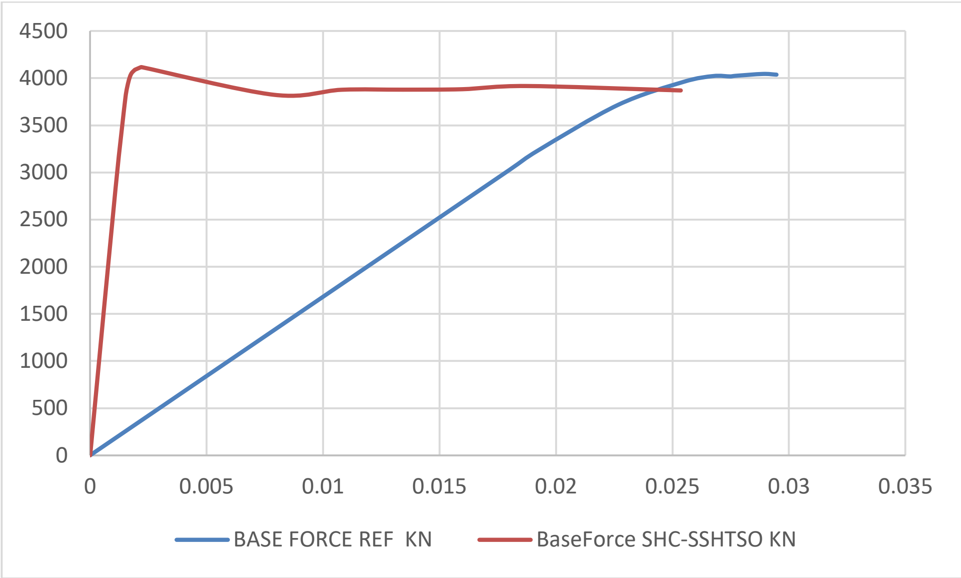


Fig 110. Fema 356 capacity curve for reference case vs shc-sshitso, in push y load.

Considering the graph above, from the comparison done with reference case and Soft story due to height infill-short column and two sided overhanged, results that the reference case, again provides the best ductility. Currently: 2.9% for reference building and 2.5% for SHC-SSHITSO.

- **Displayed values for all considered frames under Fema 356 pushover analysis, push y**

1	Table: Pushover Curve Demand - FEMA356									
2	LoadCase	Step	Displacement	BASE FORCE REF	Displacement	BASE FORCE SSWTSO	Displacement	BaseForce SSHOSO	Displacement	BaseForce SHC-SSHTSO
3			m	KN	m	KN	m	KN	m	KN
4	push y	0	0	0	0	0	0	0	0	0
5	push y	1	0.003	504.421	0.003042	1731.023	0.001323	3023.102	0.001087	2826.108
6	push y	2	0.006	1008.842	0.003278	1867.465	0.001762	3784.366	0.001517	3799.947
7	push y	3	0.009	1513.264	0.004026	2236.445	0.001941	3967.346	0.001638	3956.776
8	push y	4	0.012	2017.685	0.004202	2288.382	0.002268	4140.544	0.001754	4042.273
9	push y	5	0.015	2522.106	0.004387	2319.09	0.002325	4151.03	0.001898	4083.339
10	push y	6	0.018	3026.527	0.004434	2323.634	0.005315	3987.74	0.00194	4090.321
11	push y	7	0.01923	3233.247	0.004666	2337.741	0.008312	4070.098	0.002148	4112.307
12	push y	8	0.022664	3716.601	0.005049	2345.255	0.011305	4040.277	0.002208	4115.988
13	push y	9	0.025488	3961.732	0.00789	2326.114	0.014302	4124.466	0.007893	3822.837
14	push y	10	0.026749	4022.367	0.010894	2316.199	0.017293	4052.863	0.010807	3876.19
15	push y	11	0.027483	4018.649	0.013719	2317.218	0.02029	4137.48	0.013419	3877.11
16	push y	12	0.027692	4024.56	0.01663	2320.947	0.023282	4084.974	0.015988	3882.1
17	push y	13	0.028877	4045.212	0.019613	2314.969	0.026277	4147.086	0.018709	3915.945
18	push y	14	0.02947	4037.341	0.022694	2324.848	0.02927	4113.206	0.021363	3868.922
19	push y	15	0.029645	4042.268	0.025595	2306.107	0.032265	4155.716	0.026844	3952.961
20	push y	16	0.030326	4039.607	0.028446	2325.8	0.035258	4137.28	0.029506	3968.929
21	push y	17	0.030667	4049.458	0.031424	2319.283	0.038253	4169.158	0.032221	3974.175
22	push y	18	0.031173	4053.21	0.034412	2328.331	0.041247	4157.214	0.034059	4150.59
23	push y	19	0.031173	4053.263	0.037371	2325.626	0.044241	4185.579	0.035665	4117.494
24	push y	20	0.031353	4059.458	0.040187	2329.321	0.047235	4175.035	0.037134	4173.041
25	push y	21	0.032324	4027.554	0.043147	2331.096	0.050229	4203.254	0.037724	4167.914
26	push y	22	0.033283	4052.446	0.046147	2327.637	0.053223	4192.273	0.039922	4075.909
27	push y	23	0.033283	4052.453	0.049115	2335.715	0.056217	4219.751	0.04056	4112.456
28	push y	24	0.033423	4062.861	0.052035	2326.89	0.059211	4210.207	0.041302	4129.884
29	push y	25	0.034148	4052.76	0.054999	2335.496	0.062205	4235.002	0.044866	4017.948
30	push y	26	0.034748	4061.694	0.057886	2334.408	0.063117	4268.688	0.046244	4033.686
31	push y	27	0.035055	4068.771	0.060769	2342.297	0.065532	4261.289	0.047811	4009.16

Table 11. Displayed values for all considered frames under Fema 356 pushover analysis, push y.

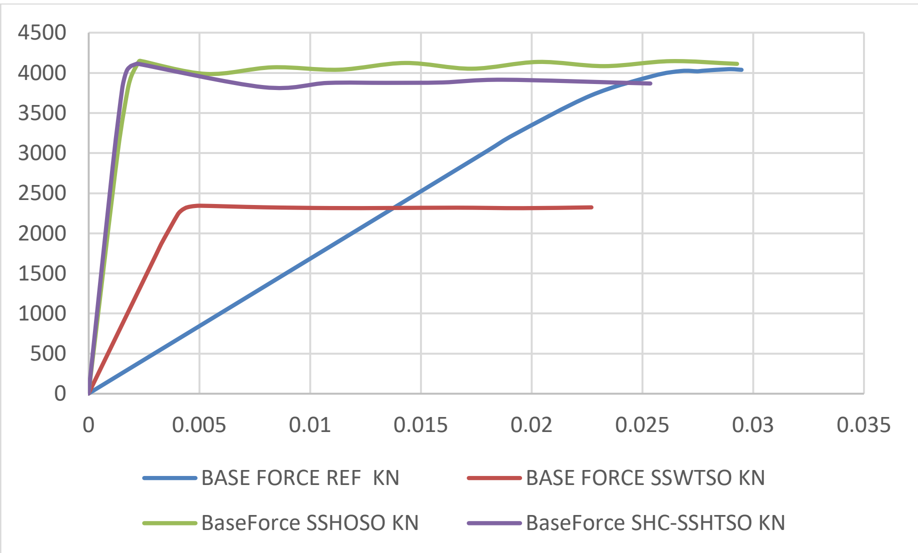


Fig 111. Displacement vs base force curves for 4 considered frames in pushy.

By displaying the pushover curve, Fema 356 in push Y load case (fig111), we were able to check how the buildings perform their elastic behaviour and range them from the good ductilty performance-medium ductility and poor ductility. It is obiously seen that the Reference case which is a regular building in plan and elevation, reaches a good post-yield deformability, comparing with three other irregular models, (REF; 2.9% displacement). After reference case the one which is less good than reference case is the building with one sided overhanged and soft ground story due to height difference (SSHOSO; 2.75% displacement). Considering the medium ductility performance, we can obtain that from the model of soft story due to height difference-two sided overhanged (balconies) and short column due to infill walls. (SHC-SSHITSO; 2.5%). The worst case, with the lowest ductility performance was that of Soft story due to irregularity in height- formation of midstory with absence of partition walls-two sided heavy overhangs due to structural addition in second and third floor. (SSWTSO; 2.2%)

- **COMPARISON BETWEEN REFERENCE CASE AND MODIFIED MODELS IN PUSH X BY FEMA 356- COEFFICIENT METHOD DISPLAYED CURVE.**

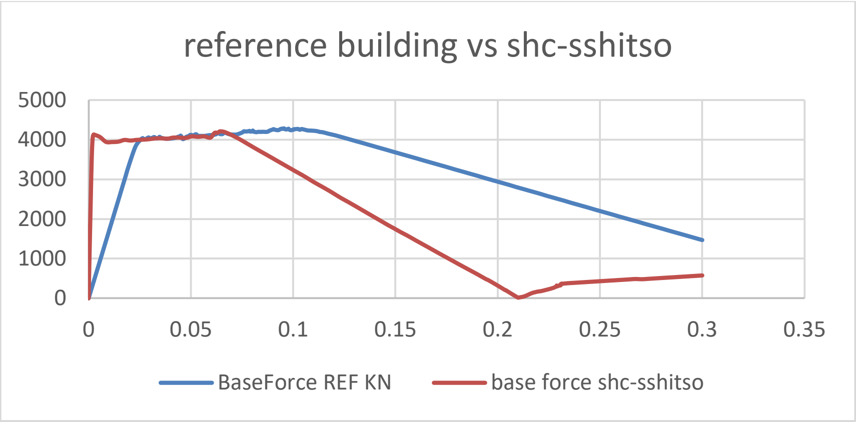


Fig 112. Fema 356 capacity curve for reference case vs shc-sshtso, in push x load.

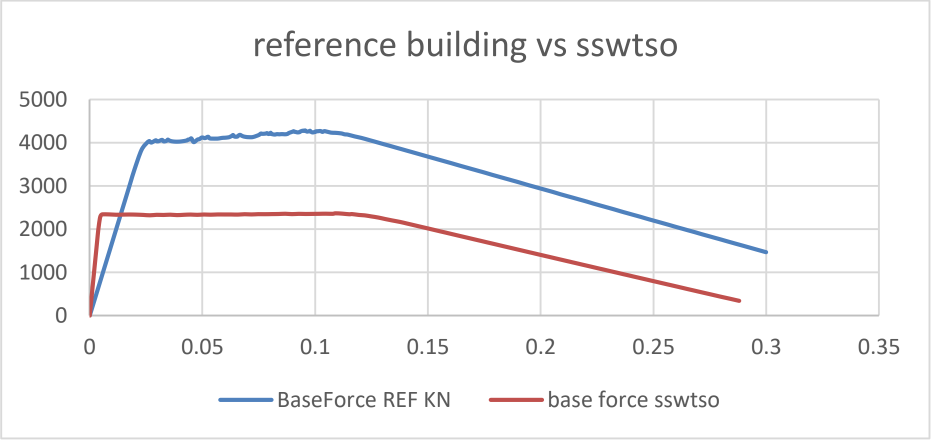


Fig 113. Fema 356 capacity curve for reference case vs sswtso, in push x load.

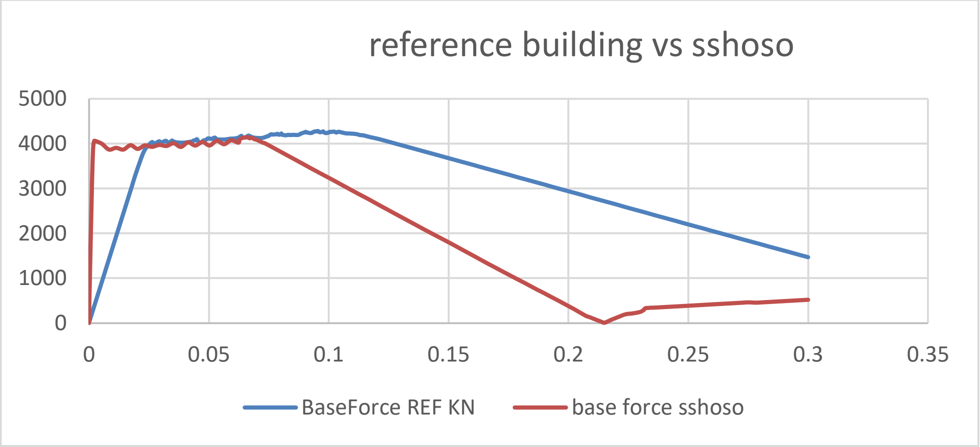


Fig 114. Fema 356 capacity curve for reference case vs sshoso, in push x load.

- **Displayed values for all considered frames under Fema 356 pushover analysis, push y**

1	TABLE: Pushover Curve Demand - FEMA356									
2	push x ref	Step	Displacement	BaseForce REF	Displacement	base force sshoso	displacement	base force shc-sshitso	displacement	ase force sswtso
3	load	Unitless	m	KN						
4	push x	0	0	0	0	0	0	0	0	0
5	push x	1	0.003	510.585	0.001308	3172.225	0.0013	3084.029	0.003019	1573.863
6	push x	2	0.006	1021.17	0.001685	3823.616	0.001696	3785.35	0.003801	1983.887
7	push x	3	0.009	1531.755	0.001811	3936.905	0.001963	4027.394	0.004539	2256.998
8	push x	4	0.012	2042.34	0.001942	4002.701	0.002088	4080.381	0.004868	2310.873
9	push x	5	0.015	2552.925	0.002053	4034.042	0.002163	4100.37	0.005048	2326.06
10	push x	6	0.018	3063.51	0.002136	4048.932	0.002257	4113.89	0.005289	2336.769
11	push x	7	0.019817	3372.631	0.002361	4066.666	0.002458	4130.662	0.005698	2344.171
12	push x	8	0.022982	3831.377	0.005361	4000.364	0.002521	4132.971	0.008698	2342.343
13	push x	9	0.025697	4013.777	0.008361	3869.612	0.005521	4070.283	0.011698	2334.468
14	push x	10	0.025952	4023.14	0.011361	3908.008	0.008521	3945.787	0.014698	2336.998
15	push x	11	0.026313	4031.735	0.014361	3869.234	0.011521	3941.9	0.017698	2339.708
16	push x	12	0.026468	4037.722	0.017361	3966.778	0.014521	3951.501	0.020698	2336.432
17	push x	13	0.027346	4002.488	0.020361	3884.266	0.017521	3994.352	0.023698	2330.452
18	push x	14	0.029282	4054.367	0.023361	3963.195	0.020521	3979.639	0.026698	2321.582
19	push x	15	0.029741	4031.571	0.026361	3930.255	0.023521	3996.895	0.029698	2332.23
20	push x	16	0.03056	4033.328	0.029361	3973.586	0.026521	4002.011	0.032698	2329.159
21	push x	17	0.032098	4064.392	0.032361	3949.885	0.029521	4013.144	0.035698	2335.176
22	push x	18	0.032807	4028.873	0.035361	4010.917	0.032521	4032.658	0.038698	2326.414
23	push x	19	0.03397	4039.419	0.038361	3928.749	0.035521	4037.243	0.041698	2332.721
24	push x	20	0.034553	4063.959	0.041361	4029.856	0.038521	4026.929	0.044698	2338.433
25	push x	21	0.034555	4064.311	0.044361	3959.541	0.041521	4054.9	0.047698	2332.047
26	push x	22	0.03468	4071.133	0.047361	4037.513	0.044521	4052.427	0.050698	2339.436
27	push x	23	0.03529	4050.954	0.050361	3961.638	0.047521	4045.411	0.053698	2335.225
28	push x	24	0.03634	4032.999	0.053361	4066.5	0.050521	4085.195	0.056698	2340.976
29	push x	25	0.037815	4023.771	0.056361	3988.989	0.053521	4073.079	0.059698	2340.367

Table 12. Displayed values for all considered frames under Fema 356 pushover analysis, push x.

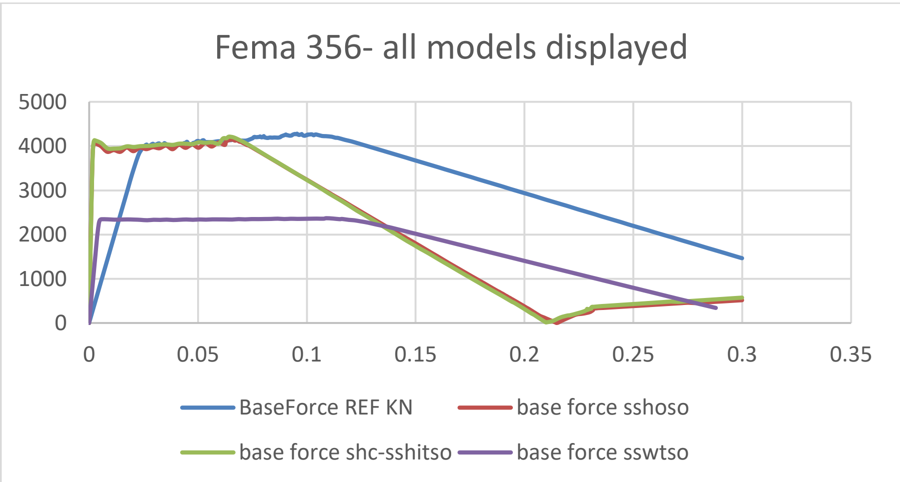


Fig 115. Displacement vs base force curves for 4 considered frames in pushx.

From pushover curve, FEMA 356 push -X we displayed the values and create the current curves based on Base shear-displacement parameters. We see that in X direction almost all cases perform better than in push Y load case. For instance, reference case performs a good ductility behaviour, reaching 3% displacement, so do even irregular cases, such as SSHOSO and SHC-SSHITSO. The only one performing less ductility is again SSWTSO, respectively 2.7% displacement.

It is obvious that all kind of irregularities effect the performance of a building. Some of them highly affect and others less. In our study we were tought that overhangs, elevation height difference, absence of masonry walls and short column formations gives poor ductility comparing to a regular building, dispate the number of stories.

Referring to our considered frames we can range them from best to worst with a respective percentage, considering the reference case as 100% good ductility.

Considered frames push-Y	**Displacement percentage**
1. Reference model	2.9% (100%)
2. SSHOSO	2.75% (70%)
3. SHC-SSHITSO	2.5% (50%)
4. SSWTSO	2.2% (20%)
Considered frames push-X	**Displacement percentage**
1. Reference model	3% (100%)
2. SSHOSO	2.9% (100%)
3. SHC-SSHITSO	2.95% (100%)
4. SSWTSO	2.7% (70%)

CHAPTER 6

CONCLUSIONS

6.1 Conclusions.

A typical residential R-C frame- building, 6 story height, was taken in evaluation through this study. This kind of building is mostly representative of lates -90 buildings typology due to their same construction method, period and quality. The structural design model introduced irregularities such as: soft story due to difference in height and soft story due to absence of masonry partition walls for open space commercial purposes, poor reinforcement details, structural additions represented as heavy overhangs, semi infilled frames and short column effects. The seismic performance assessment by analytical method consisting of pushover analysis was a simple way to explore the non-linear behaviour of building. The results we had from pushover curves, capacity spectrum and plastic hinges formation provide real understanding of structural behaviour.

What observed from analysis shows that every structural irregularity in RC building, affect the structure by decreasing its performance level in terms of lower load bearing stiffness and displacement capacity. When comparing the presence of irregularities some of the such as combination of soft story with height difference – absence of infill walls – short column due to semi infilled frames, - and overhangs, results in serios damages leading to total loss of stiffness in structure.

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