ANALYSIS OF SEISMIC BEHAVIOUR OF IRREGULAR STRUCTURES

A DISSERTATION
SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR THE AWARD OF DEGREE

OF

MASTER OF TECHNOLOGY

IN

STRUCTURAL ENGINEERING

SUBMITTED BY

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CANDIDATE'S DECLARATION

I, Vethivolu Nyekha (Roll No 2K20/STE/25), student of M.Tech (Structural Engineering), hereby declare that the project Dissertation titled "**Analysis of seismic behaviour of irregular structures "** which is submitted by me to the Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfilment of the requirement for the award of degree of Master of Technology, is original and not copied from any source without proper citation. This work has not previously formed the basis for the award of and Degree, Diploma Associateship, fellowship or other similar title or recognition.

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CERTIFICATE

I hereby certify that the project dissertation titled "Analysis of seismic behaviour of irregular structures," which is submitted by Vethivolu Nyekha, Roll no 2K20/STE/25, Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfilment of the requirement for the award of the degree of Master of Technology, is a record of the project work carried out by her under my supervision. To the best of my knowledge this work has not been submitted in part or full for any degree or diploma to this university or elsewhere.

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ABSTRACT

A structure can be categorised as irregular, if it contains uneven distributions of mass, stiffness and strength or due to irregular geometry. Most structures contain irregularity due to functional and aesthetic reasons. One of the significant reasons for a structure's collapse from past earthquakes, is the irregular configuration of the building either in plan or elevation as the performance of a building during a seismic event primarily depends on its configuration. Thus, irregular structures situated in highly seismic zones are a matter of concern.

Structures do not always contain single irregularity but may contain combination of structural irregularities. The choice of type, location and degree of irregularity is crucial in the design of structures. IS code 1893 (part1) 2016 recommends that all efforts should be made to remove irregularities by revising architectural planning and structural configurations. However, in structures the concept of "perfect regularity" is just an idealisation as real structures contain irregularity necessitated due to various needs and demands and also constitutes a large portion of modern urban infrastructure. Hence irregularities in a structure has become inevitable and the current study aims to incorporate irregularities in a structure without compromising the performance. A G+7 storey regular building RC frame is modified by incorporating single as well as combined irregularities involving mass, stiffness, and vertical geometric irregularities. All the models are subjected to earthquake loading and the responses are computed using CSI ETABS software. The objective will be accomplished by comparing the responses of the various models by two methods of seismic analysis namely Response Spectrum Analysis and Pushover Analysis.

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ABBREVIATIONS

R=Model with regular configuration

SI=Model with stiffness irregularity

VG=Model with vertical geometric irregularity

MI=Model with mass irregularity

CO-I=Model with combination of mass and stiffness irregularity

CO-II=Model with combination of stiffness and vertical geometric irregularity

RSA=Response Spectrum Analysis

PoA=Pushover Analysis

IO=Immediate occupancy

LS=Life safety

CP=Collapse Prevention

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Building components that resist seismic forces are known as lateral force resistance systems. The damage in a building usually starts at the weak planes present in a structural system. These vulnerabilities can further cause structural degradation and lead to structural collapse. The presence of the structural irregularities in a structure often gives rise to these weaknesses. Buildings with regular configuration and evenly distributed mass, stiffness and strength in plan and elevation suffer less damage as recorded from past seismic events than buildings with irregular geometry [6].

The structural irregularity can be plan or vertical irregularities depending on the presence of irregularity in distribution of mass, stiffness or strength in plan or elevation. As per IS 1893 (Part1): 2016, a storey in a building contains mass irregularity, if its seismic weight is more than 150 % of that of the storey below. When the lateral stiffness of a particular storey is less than that of the storey above, it contains stiffness irregularity (soft storey). And if the lateral strength of a storey is less than that of the storey above, it contains strength irregularity (weak storey).

In structures, the idea of perfect regularity is just an idealisation as real structures contain irregularities due to various reasons. [8]. Many existing structures contain irregularity for utility and aesthetic requirements. Some structures have been originally designed to be irregular to accomplish different functions. The difference in usage of a particular storey as compared to the adjacent storey can also result in irregularity [11]. In addition to this, many structures are inadvertently rendered irregular during construction stage due to several reasons like non-uniformity in construction practices and raw materials used.

From review of code limits, it can be deduced that majority of the seismic codes propose similar guidelines for the definition of irregularities based on magnitude ignoring the aspect of location of irregularity. However the type, location and degree of irregularities in the design of structures is significant. The right choice will aid in enhancing the utility and aesthetics of structures [6]. Although irregular buildings are preferred due to their functional and aesthetic considerations, the records of past earthquake display poor seismic performance of these structures [9]. But irregularities in a structure are inevitable, and as such, irregularities should be so chosen and located so that the structure's performance is not hampered.

1.2 TYPES OF IRREGULARITIES

The detailed classification of different structural irregularities as per IS 1893(Part 1): 2016 is presented in Table 1.1 and code limits have been shown in Table 1.2

Table 1.1 Types of irregularity as per IS 1893(part 1): 2016 [4]

TYPES OF IRREGULARITY	
PLAN IRREGULARITIES	VERTICAL IRREGULARITIES
1. Torsional irregularity 2. Re-entrant corner 3. Floor slabs having excessive cut outs or openings 4. Out-of-plane offsets in vertical elements 5. Non parallel lateral force system	1. Stiffness irregularity 2. Mass irregularity 3. Vertical geometric irregularity 4. In plane discontinuity in vertical elements resisting lateral force 5. Strength irregularity 6. Floating or stub column 7. Irregular modes of oscillation in two principal directions

Table 1.2: Irregularity limits as per IS 1893 (Part 1): 2016

Irregularity	**Type**	**Limits**
Mass	Vertical irregularity	$M_{i+1} > 1.5\ M_i$
Stiffness	Vertical irregularity	$S_i < S_{i+1}$
Torsion	Plan irregularity	$\Delta_{max}/\Delta_{avg}$=1.5 to 2.0 >2.0 extreme irregularity
Vertical Geometry	Vertical irregularity	A>0.1L

1.3 METHODS OF SEISMIC ANALYSIS

The seismic response of the building system shows that it is highly dependent on the seismic analysis method applied. In the past, the analysis methods were confined to linear static approach due to its ease in application, the simplicity in computation and interpretation. The methods yielded safe design but were found to be over conservative. With the development of advanced computers and analysis software, researchers have been able to stimulate actual earthquakes on the model to obtain more realistic seismic responses. These methods came to known as dynamic analysis.

Depending upon the force & deformation relationship of the structural members, both the static and dynamic analysis are further divided into linear and nonlinear methods.

Introducing structural irregularities affects the dynamic response by shifting the fundamental period and changing the mode shapes [11]

The methods are briefly discussed as under:

1.3.1 Equivalent Lateral Force Method

By taking the assumption that the lateral force is equal to the actual loading, the seismic analysis is carried out. As per IS 1893 (Part1): 2016, the linear static method is only applicable for regular buildings with height less than 15 m in seismic zone II and regular structures with approximate natural period T_a less than 0.4 s.

This method demands less computational work because the periods and shapes of higher modes are ignored. The base shear is calculated on the based of the structure's mass, its fundamental period and shape using the code prescribed formula. The base shear is then distributed along the height of the structure in terms of lateral forces.

1.3.2 Response Spectrum Analysis (RSA)

This method is recommended for structures where higher vibration modes have a significant effect on the response of the structure. This method is generally applicable to the analysis of dynamic responses of irregular structures or have discontinuous regions in the linear behaviour region. In particular, it can be applied to the analysis of forces and deformations of high-rise buildings exposed to medium intensity ground vibrations that cause moderately large but essentially linear responses to structures.

This method calculates the response of each natural vibration mode independently of the other modes for a particular damping mode and the modal responses can be combined to determine the overall response. As per IS 1893 - 2016 (Part 1) this method can be applied to all buildings other than regular buildings lower than 15 m in seismic zone II.

1.3.3 Pushover Analysis (PoA)

Pushover analysis is a non-linear static analysis as it allows inelastic behaviour of the structure. This method provides information on the strength, deformation, and ductility of the structure, as well as the distribution of demands. The method also predicts potential weak areas in the structure and identification of the vulnerable members that are most likely to reach their limit states. The identification of these critical members allows the engineer to revise the design and detailing process during the design stage.

For existing structures, this method can be used where seismic retrofitting is desired to meet the present demands or if the structure has deficiencies in seismic resisting capacity. But this method has limitations as it does not take into consideration the variation of loading patterns, the higher modes of vibration and resonance. Also the pushover analysis is not part of IS code.

Capacity Spectrum Method (ATC-40) and the Displacement Coefficient method (FEMA 356) are the two widely accepted procedures for Pushover Analysis (PoA) of buildings. In Pushover Analysis, the importance Factor as per table 8 of IS 1893 (Part 1): 2016 is not considered. The performance level of a building takes care of the criteria of Importance Factor.

1.3.4 Time History Method

This method is applicable to both elastic and inelastic analysis. Time history is a non-linear dynamic analysis method which is considered the most accurate method to describe the actual seismic behaviour of a structure. The structural response is calculated at a number of time intervals. This method however requires intensive computational effort for calculating the seismic response and expert interpretation skills. Hence, this method is recommended only for design of special structures.

1.4 OBJECTIVES OF THE STUDY

- To model regular as well as irregular frames incorporating single and combination of irregularities (plan & vertical) to generate different configurations
- To create these 3D models in ETABS software and perform Response Spectrum Analysis and Pushover Analysis
- To study and compare the various responses -storey displacement, storey drift and base shear.
- Comparison of various configurations of frames on the basis of seismic response
- To check the building performance level based on the hinge formation in pushover analysis.
- To study the critical combination of irregularities and combinations causing lesser seismic response if any.
- To enable incorporating irregularities in a structure without adversely affecting its seismic performance.

CHAPTER 2

LITERATURE REVIEW

2.1 LITERATURE REVIEW

Zabihullah et al (2020) analyzed the seismic response of a G+7 Storey RC building having various individual and combined complicated geometric irregularities in its horizontal and/or vertical planes. The horizontal irregular model was found to be the most vulnerable and the vertically irregular model was found to possess the best seismic performance. This study indicated that certain combinations of irregularities may not necessarily amplify the seismic response but may decline it.

Naveen et al (2019) focused on combination of irregularities for prediction of seismic response as structures usually contain multiple irregularities. For models with single irregularity, the model with stiffness irregularity was found to have maximum effect on the response. For combination of irregularity model, the combination of mass, stiffness and vertical geometric irregularities was found to be most vulnerable. The focus of the study was to enable incorporating irregularities in a structure without adversely affecting its performance.

Guevera et al (1992) analysed H shaped and L shaped buildings through a dynamic analysis to corroborate the hypothesis that geometric irregularity in floor plans is one of the most significant factor causing torsional effects in a building. Building damage cannot be entirely attributed to floor plan irregularities but it recognised that these type of buildings behave inadequately under action of seismic force. The result of the study was even buildings belonging to the same family did not possess same degree of vulnerability due to various reasons.

Gokdemir et al (2013) studied the torsional effect on structures. Building models having different floor areas and number of floors are modelled and analyzed by SAP 2000. The analysis showed that eccentricity caused torsion in structures. The magnitude of this moment is the function of eccentricity ratio. The paper concluded that separation of building sections into regular configurations with appropriate separation distances and increasing the lateral rigidity of the weaker direction of the structure can decrease the torsion.

Sanyogita and Babita Saini (2019) analysed and studied the response of three frames regular, I shaped and stepped for G+2, G+4 & G+11 buildings using equivalent static method by ETABS. It was observed that the stiffness value of structure containing geometric irregularity was more than that of regular structure which caused corresponding decrease in the displacement values. As the vertical irregularity increased, it was observed that the corresponding base shear decreased due to the reduction in mass. The increase in setback increased both the shear force and displacement value.

Ozmen et al (2014) conducted a parametric study on six groups of structures with varying shear wall positions, different number of storey and axis numbers to determine the cause of excessive torsional irregularity and also discuss the validity of existing code provisions for the same. It was found that maximum torsional irregularity coefficient occur for single-storey structures and this value decreased as the storey number increased. When the shear walls at asymmetrical locations are placed nearer to the centers of mass, the coefficient was found to possess maximum value. On the other hand, the results obtained for floor rotations were contradictory. Floor rotations increased with increase in storey number. Hence, a provisional new definition for torsional irregularity coefficient based on floor rotations was suggested.

Cotipalli et al (2020) conducted a study to understand the seismic effects on a building when one of the parameters is varied while the others remain constant. The seismic zone V is kept constant and the loading is applied to all the structures (regular and irregular) with varying heights in all the three soil types. The base shear is determined as per equivalent static lateral force method. The pattern was observed to be similar in both the configurations.

Rofooei and Mirjalili (2018) cited that the responses of a plan asymmetric building cannot be adequately approximated by static pushover analysis. Hence a dynamic based pushover analysis (DPPA) is proposed. The aim was to appropriately consider the torsional effects as well as higher modes in the applied lateral load pattern. The paper concluded that the proposed procedure estimated the seismic demand more accurately than other existing pushover procedures especially for buildings with dominant behaviour as shear.

Raheem et al (2018) discussed that damage assessments after past earthquake show that plan irregular buildings suffer adverse damage due to extreme torsion and stress concentrations. In this paper a constructive research for the class of L shaped buildings to investigate the structural seismic response demand is done using ETABS. The paper concluded that the more irregular the structure is, larger will be the number of modes needed to compute the response.

Mario De Stefano Barbara Pintucchi (2008) presented a summary of the development in research in three areas concerning seismic response of plan and vertical irregular buildings. Firstly, the effects of plan irregularity by use of both single and multi-storey models were studied. Secondly it discussed passive control devices as a measure to mitigate torsional effects. And thirdly it discussed vertically irregular structures and setback buildings. The study concluded that discontinuities of mass, strength and stiffness along the height do not necessarily result in actual increase in plastic demands or poor seismic behaviour.

Penelis and Kappos (2002) cited that pushover analysis do not properly account the torsional effects in a structure. They presented a methodology to model inelastic torsional response of buildings. The approach was verified with the use of single storey mono symmetric buildings.

Shakeri et al (2014) proposed a pushover procedure with a load pattern based on the height-wise distribution of the combined modal story shear for plan irregular buildings. Unlike the conventional procedure, here the higher modes are taken into consideration and the torsional response are included in the proposed methodology. The proposed method is a one-time run, which permits to trace the non-linear response of the structure during analysis. This avoids the need to conduct several pushover analysis.

2.2 LITERATURE GAP

- The shortage of experimental studies characterises the state of research activities in the area of irregular structures.

- Real Structures are usually a combination of both types of irregularities. Most literatures and seismic codes separate between the irregularities in plan and elevation.

- Recent research papers cited that the existence of irregularities in a structure does not necessarily amplify the response but some combinations of irregularities decrease the structural response. Fewer literature is available for such cases.

- The non-linear static pushover analysis is yet to be incorporated in the Indian standard codes.

CHAPTER 3

METHODOLOGY

3.1 DEFINITION OF BUILDING MODELS

The present study incorporates a regular G+7 building model with different types of irregularities. The responses of these models are compared to that of regular model which is considered the base model. The various building models have been briefly described below:

3.1.1 Regular model (R)

The regular building model is represented by the building model with no irregularity in mass, stiffness and strength distribution. The different types of irregularities have been incorporated in the regular building model to generate irregular building models. These irregularities have been introduced by varying mass, stiffness and geometry. This model is considered the base model and all comparisons and results are discussed with respect to the base model

3.1.2 Mass irregularity Model (MI)

Mass irregularity was introduced by increasing the mass of one storey and keeping the other storey masses constant. Mass irregularity usually arises due to the difference in usage of one store as compared to the other storey. In this study, a parking floor is considered in the third floor. As per IS 875 (PART 2): 1987, the uniformly distributed load to be considered for garage floors for vehicles not exceeding 4.0 tonnes gross weight (including access ways and ramps) should not be less than 5.0 kN/m^2[5]. The mass of third floor is increased to 1.5 times the mass of other floor.

3.1.3 Stiffness irregularity model (SI)

Stiffness irregularity is introduced by increasing the length of columns to 4.5 m from 3m in the bottom storey as this is considered the most severe case and yet practical case [11]. The calculation shows that the storey stiffness in the bottom storey calculated is only 57.0567% with respect to the storey above categorising it as a soft Storey.

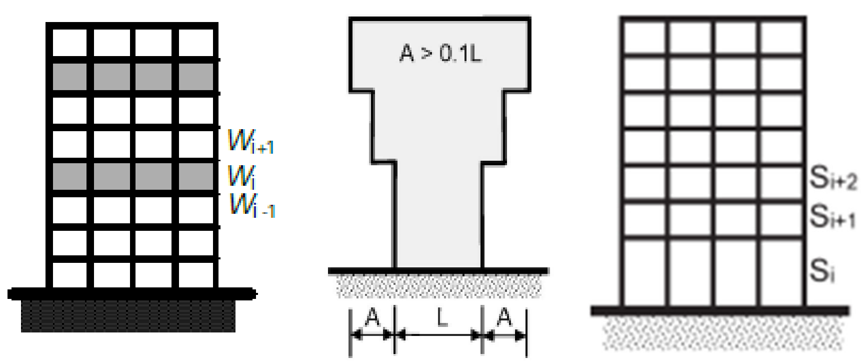
3.1.4 Vertical geometric irregularity Model (VG)

A stepped building is considered as this is commonly done in practice to meet architectural requirements. As per IS 1893 2016 part 1 A building is considered to possess geometric irregularity if A>0.1L as per fig 3.1 (b)

3.1.5 Combination of irregularity models (CO-I & CO-II)

Two models having combination of irregularities are analysed

1. CO-I=combination of mass irregularity (MI) and stiffness irregularity (SI)
2. CO-II=Combination of vertical geometric irregularity (VG) and stiffness irregularity (SI)



(a)Wi>Wi-1 (b) A>0.1L (c) Si<Si+1

(a) Mass irregularity (b) vertical geometric irregularity (c) stiffness irregularity

Fig 3.1 Descriptions of irregular buildings as per IS 1893 (Part 1) 2016 [4]

3.2 CODES AND STANDARDS

The relevant codes and software used in the modelling are listed below:

- The modelling and analysis is carried out in CSI software ETABS 2018
- The structure properties are designed and detailed as per IS 456:2000 and IS 800:2007
- The loads considered and load combinations are according to IS 875 (part 2): 1987
- Seismic analysis and seismic loading conform to IS 1893 (part 1): 2016
- The performance levels checks are in accordance with ATC 40

3.3 MODELLING

The modelling is done in CSI, INC Structural and Earthquake Engineering software ETABS 2018 which has inbuilt functions for both Response Spectrum Analysis and Pushover Analysis. The part of the procedure which is common to both are defining the geometry, materials, section, restraints for support and creating the load patterns and assigning the loads. The subsequent steps for PoA consists of defining the properties for the non-linear hinges for frames and shell section and defining the gravity nonlinear static load case and creating the nonlinear Pushover load case. The load application is displacement controlled by putting a default value of 0.04 times the building height. After running the analysis, the capacity curve and performance point is displayed in the result. While the subsequent steps for RSA consists in defining the response spectrum function and Response spectrum load case. The analysis process is linear dynamic for Response Spectrum Analysis and non-linear static for Pushover Analysis. The analysis output in both cases remain similar like base shear, drift and maximum storey displacement which have been discussed in detail in the later part of this study.

3.4 INPUT PARAMETERS OF THE MODELS

The table 3.1 below consist of the detailed Specifications and input parameters of the model that was used in the analysis process.

Table 3.1 Input parameters of the models

Seismic parameters as per IS 1893 (part 1): 2016	
Type of Building	Office
Type of Frame	SMRF
Seismic zone and Zone Factor	Zone V and 0.36
Type of soil	Medium (Type II)
Response Reduction factor (R)	5
Importance Factor (I)	1.5
Type of support	Fixed
Time period	Program calculated
Method of seismic analysis	Response Spectrum analysis Pushover analysis
Geometric parameters	
Storey height	3m for all storeys 4.5 m for bottom storey for SI model and 3m for remaining storeys
Overall height of the building	24 m for regular model 25.5 m for SI model
Overall dimensions of plan in X direction	28 m (7 bays, 4 m each) for all models 12 m (3 bays, 4m each) for VG and CO-II model
Overall dimensions of plan in Y direction	17.5 m (5 bays ,3.5 m) 10.5 m (3 bays, 3.5 m each) for VG and CO-II model
Dimensions of structural members	
Cross section of column (mm)	450 x 450
Cross section of beam (mm)	400 x 450

Depth of Slab (mm)	150
Thickness of partition wall (mm)	125
Thickness of main wall (mm)	250
Thickness of parapet wall (mm)	120
Height of parapet wall (mm)	1000
Properties of Grade of concrete and steel	
Grade of concrete	M 25
Grade of steel	Fe 415
Density of Reinforced concrete	25 kN/m^2
Density of brick	19 kN/m^2
Loads on frame	
Floor finish load	1.5 kN/m^2
Live load	4 kN/m^2
Dead load of main wall	14.25 kN/m^2
Dead load of partition wall	7.125 kN/m^2
Dead load of parapet wall	2.8 kN/m^2

CHAPTER 4

ANALYSIS AND RESULTS

4.1 RESPONSE SPECTRUM ANALYSIS

All the various models are seismically analysed as per Response Spectrum analysis as this is the recommended method as per IS code for irregular structures. The responses of the model as per RSA are detailed and discussed below:

4.1.1 Regular model (R)

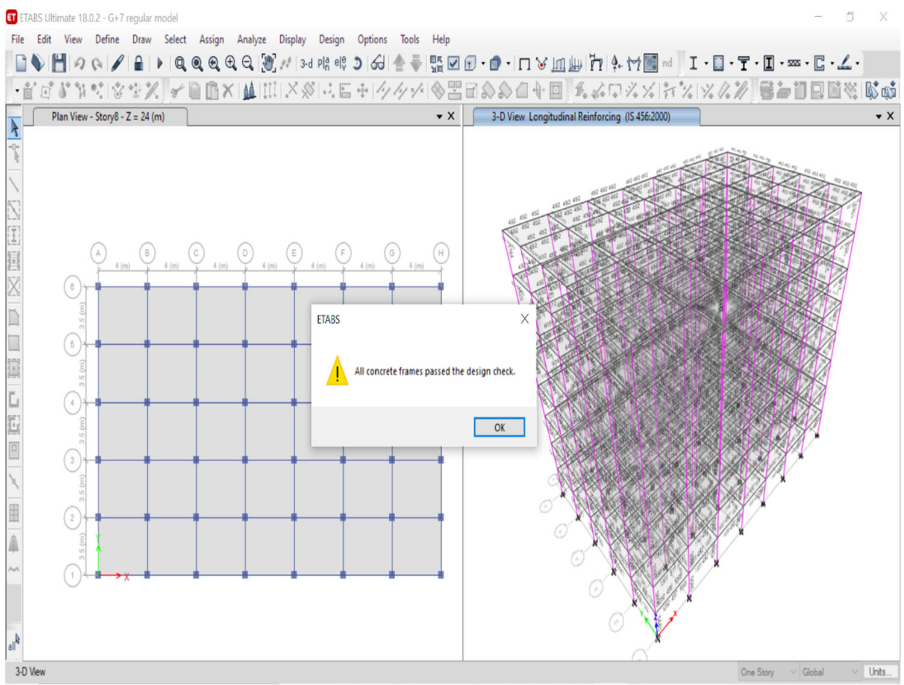


Fig 4.1 Plan and 3D view of regular model (R)

The design of the model was found to be adequate as per concrete frame design check according to IS 456: 2000. The seismic response parameters and the corresponding values categorising it as a regular building are listed in table 4.1. This model is considered the base model and all comparisons are discussed with respect to the base case.

Table 4.1 Response of R model as per RSA

Seismic response parameter	**Value**	**Limit**
Maximum storey displacement	36.509 mm (storey 8)	96 mm
Maximum Storey drift	0.00206 (storey 3)	0.004
Maximum stiffness	1040186 kN/m	
Torsional Irregularity ratio	1.1	1.5
Remark	Model is considered regular (R)	

4.1.2 Stiffness Irregularity model (SI)

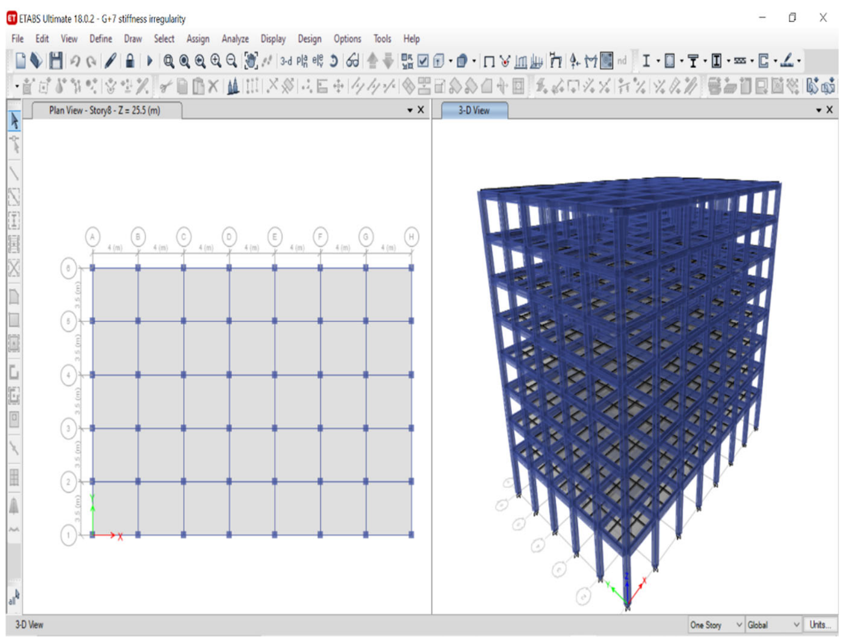


Fig 4.2 Plan and 3D view of stiffness irregularity model (SI)

Table 4.2 Response of SI model as per RSA

Seismic response parameter	**Value**	**Limit**
Maximum storey displacement	38.01mm (storey 8)	102 mm
Maximum Storey drift	0.002272 (store 1)	0.004
Maximum stiffness (kN/m)	692213 (storey 3)	

Stiffness of storey 1 (K1)	380667.721 kN/m	
Stiffness of storey 2 (K2)	667053.231 kN/m	
Stiffness ratio K1/K2	57.0567%	100%
Remark	Model has stiffness irregularity (SI) as per IS 1893 (part 1) 2016 table 6	

The bottom storey height is increased to 4.5 m and it was calculated that the stiffness of storey 1 was found to be 57.06% of storey 2 in the SI model as shown in table 4.2. The stiffness decrease in storey 1 of SI model was found to be 63.4% of the R model. The corresponding drift was increased by 57.7% and the top storey displacement increased by 4%.

4.1.3 Vertical Geometric irregularity Model (VG)

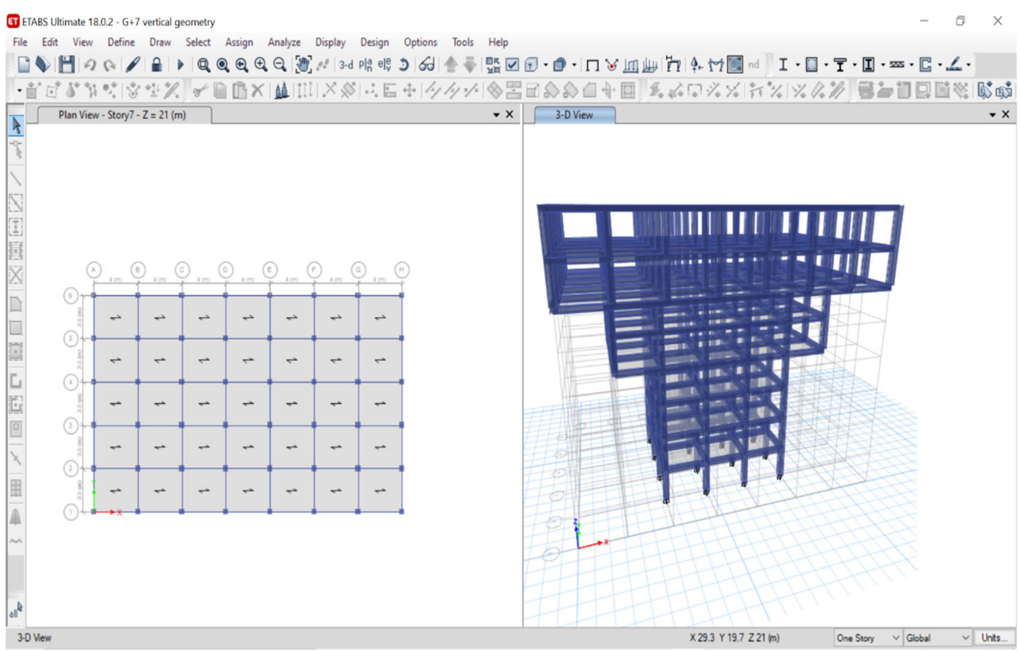


Fig 4.3 Plan (top view) and 3D view of vertical geometric irregularity model (VG)

Table 4.3 Response of VG model as per RSA

Seismic response parameter	**Value**	**Limit**
Maximum storey displacement	25.75 mm (storey 8)	96 mm
Maximum Storey drift	0.001447(storey 3)	0.004
Maximum stiffness	272118 kN/m (storey 1)	
VG limit	A=8m L=16m	A>0.1L
Remark	Model has vertical geometric irregularity as per IS 1893 (part 1) 2016 table 6	

4.1.4 Mass irregularity model (MI)

The mass of third storey was increased by 1.5 times causing uneven distribution of mass in the model.

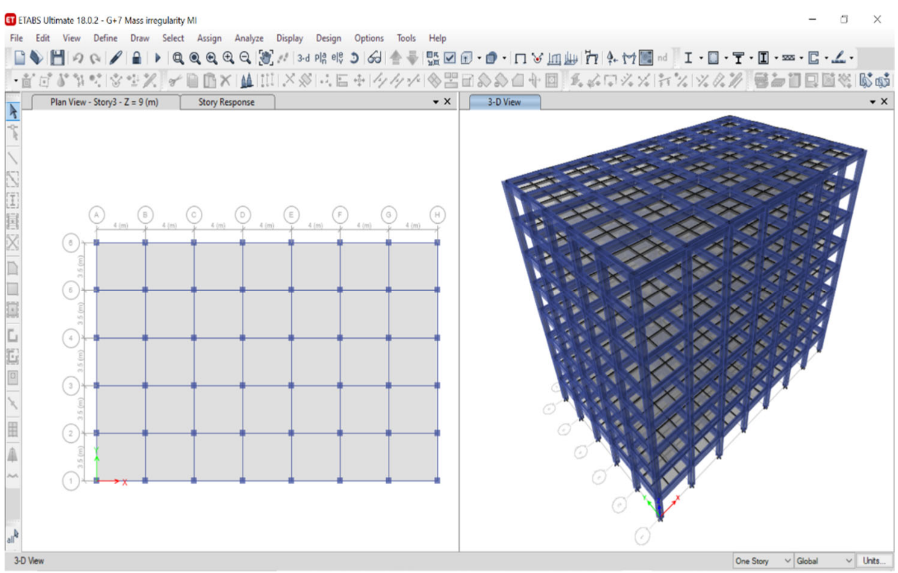


Fig 4.4 Plan and 3D view of mass irregularity model (MI)

Table 4.4 Response of MI model as per RSA

Seismic response parameter	**Value**	**Limit**
Maximum storey displacement	42.36 mm (Storey 8)	96mm
Maximum Storey drift	.002551 (storey 2)	0.004

Maximum stiffness	1040760 kN/m (storey 1)	
Mass of storey 3 (M3)	773896.06 kg	
Mass of storey 2 (M2)	515737.92 Kg	
Ratio M3/M2	1.5	1
Remark	Building has mass irregularity as per IS 1893 (part 1) 2016 table 6	

4.1.5 Combination of mass and stiffness irregularity model (CO-I)

Table 4.5 Response of CO-I model as per RSA

Seismic response parameter	**Value**	**Limit**
Maximum storey displacement	43.425 mm (storey 8)	102 mm
Maximum Storey drift	0.00277 (Storey 1)	0.004
Maximum stiffness	710747 kN/m (storey3)	

4.1.6 Combination of stiffness and vertical geometric model (CO-II)

Table 4.6 Response of CO-II model as per RSA

Seismic response parameter	**Value**	**Limit**
Maximum storey displacement	60.282 mm (storey 8)	102 mm
Maximum Storey drift	0.003016 (Storey 1)	0.004
Maximum stiffness	215539 (storey 7)	

4.2 COMPARISON OF THE RESPONSES OF ALL MODELS AS DERIVED FROM RESPONSE SPECTRUM ANALYSIS (RSA)

The responses of all the models are computed and recorded in tabular or graphical form and investigation is carried out in x direction.

4.2.1 Base shear and seismic weight

Table 4.7 Seismic weight and base shear as per RSA

Building model	R	SI	VG	MI	CO-I	CO-II
Base Shear (kN)	4226.58	3636.90	620.51	5250.82	1544.08	1288.42
seismic weight (kN)	61133.12	61315.32	33796.86	83918.12	84100.31	36947.30

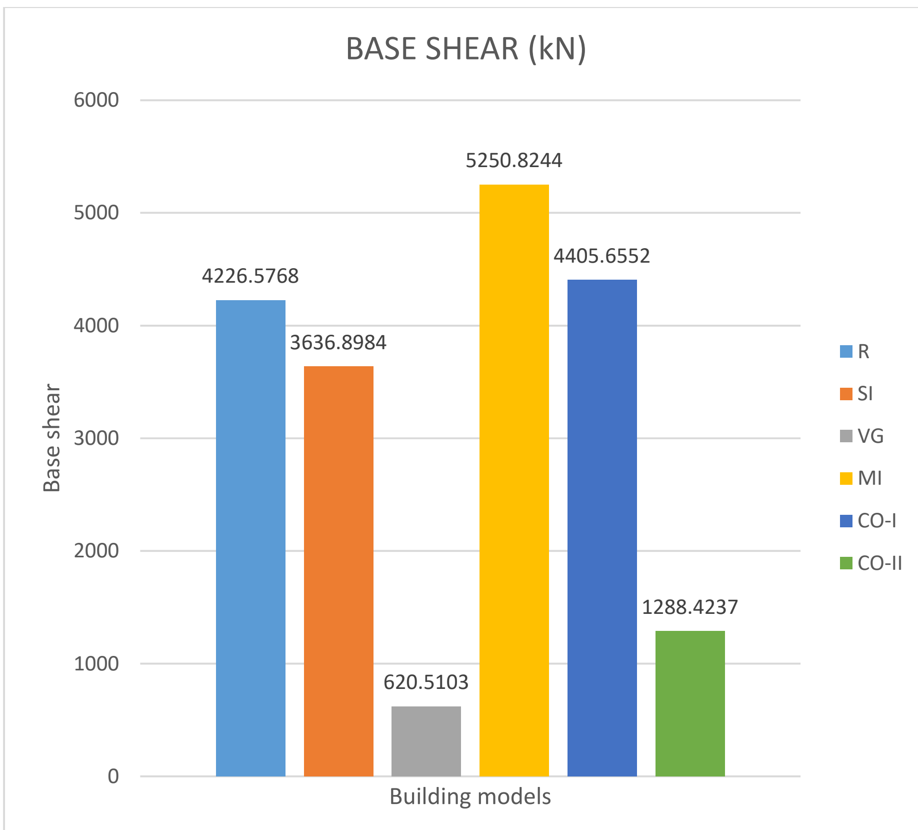


Fig 4.5 Base shear of models in x direction

The seismic weight of all models vary due to change in degree and location of irregularity, which has a direct impact on the base shear. The base shear is found to have maximum value for Mass irregularity (MI) Model while it has minimum value for the vertical geometric irregularity (VG) model. The base shear value of MI and CO-I model are found to be higher than the regular model and it implies that these two models can resist more seimic forces than the regular model.

4.2.2 Maximum storey displacement (mm)

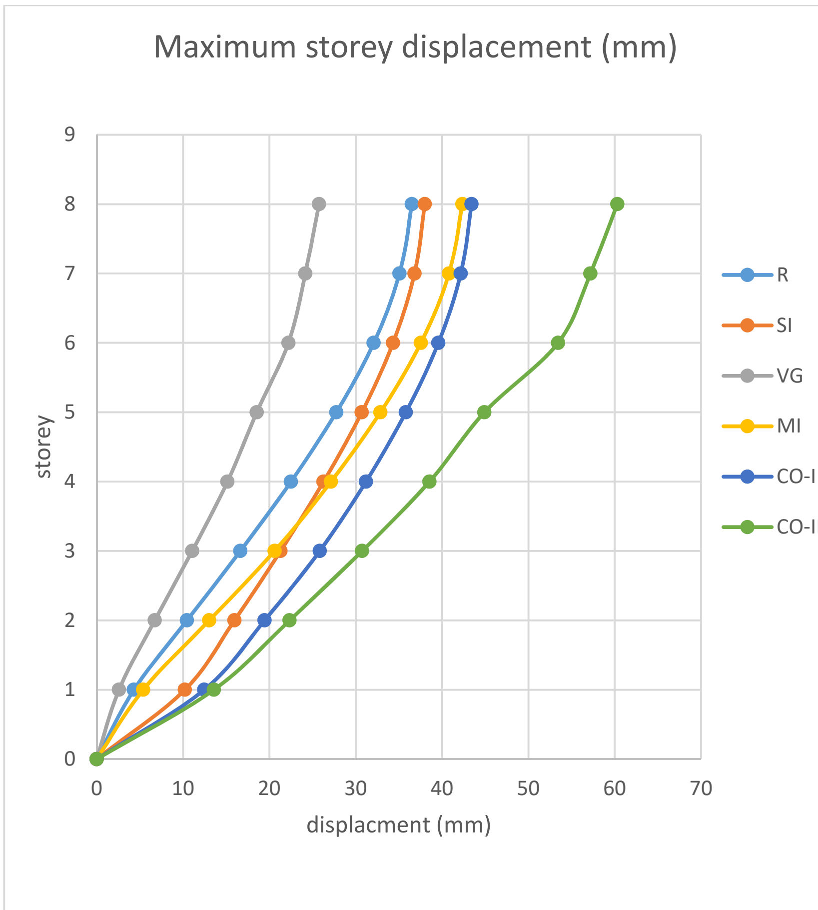


Fig 4.6 Maximum storey displacement in x direction

As per fig 4.6, the VG model displays superior performance with an overall top storey displacement of 25.75 mm while the combination model CO-II displays the worst performance with top storey displacement of 60.282 mm. However, the maximum displacement limit of 0.004 times the height of storey (i.e., 96 mm for all models and 102 mm for models containing SI) is satisfied by all models.

4.2.3 Storey drift

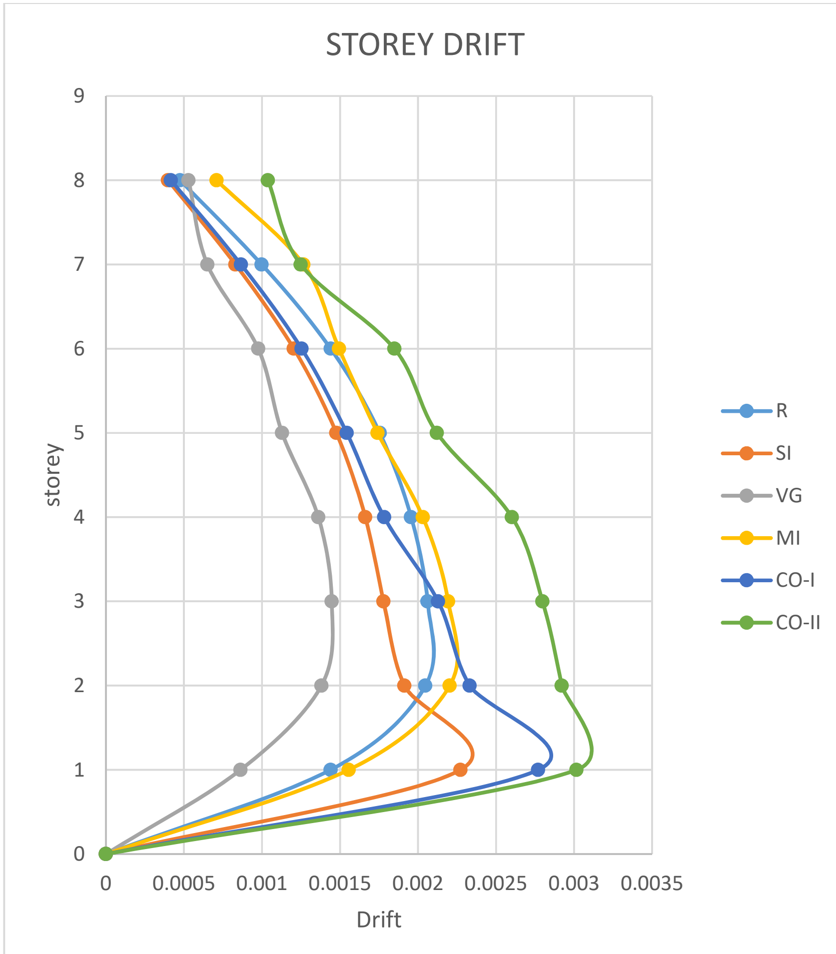


Fig 4.7 Storey Drift in x direction

As per fig 4.7, the VG model has overall lowest drift values while the CO-II model has the highest overall drift value. The models R,SI and VG have maimum drift in storey 3, model MI in storey 2 and models CO-I and CO-II have maximum drift in storey 1.The storey drift criteria of 0.004 times the storey height as per IS 1893 (part 1): 2016 clause 7.11.1.1 is satified by all models. In ETABS, the generated value of drift is Storey ratio which is the ratio of storey drift to the storey height and it is unitless.

4.2.4 Storey Stiffness

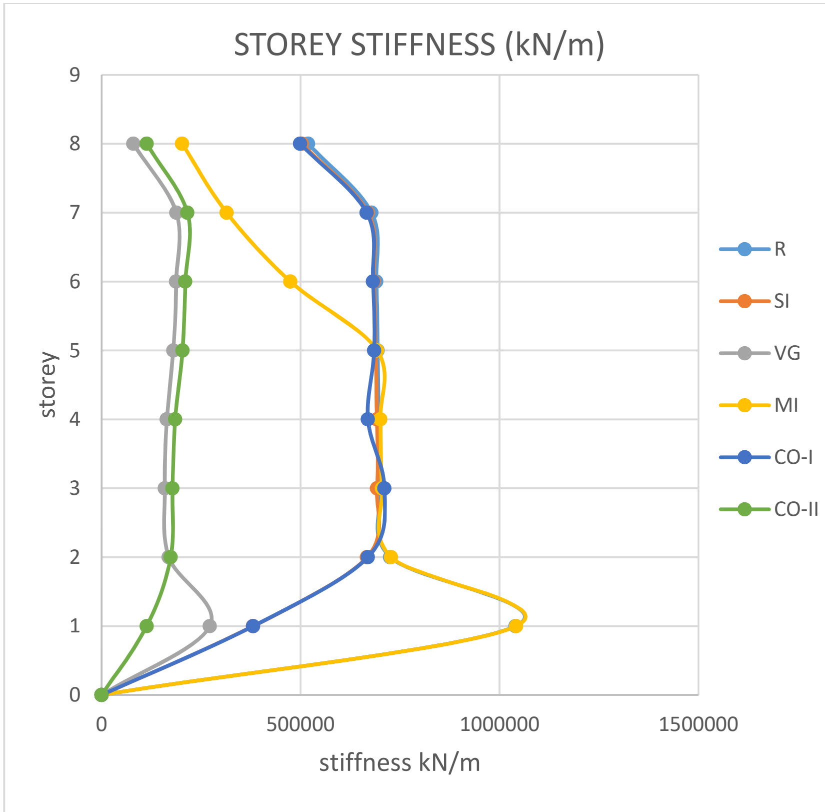


Fig 4.8 Storey stiffness in x direction

The storey stiffness of irregular models is visibly lower as compared to the regular model. The VG model has the lowest stiffness in the upper storeys while the combination model CO-II has the lowest stiffness in the bottom storey. The model CO-II however seems to have a stable stiffness through out the building's height. The mass irregularity shows a drastic decrease in the stiffness in the upper storeys compared to the lower storeys.

4.3 Pushover Analysis (PoA)

To study the inelastic behaviour, pushover analysis is carried out. The lateral deformations at the performance point displacement are to be checked against the deformation limits as per ATC 40 given in table 4.8 below.

Table 4.8 Deformation limits as per ATC 40

	Performance Level			
Interstorey drift limit	Immediate occupancy	Damage control	Life safety	Structural stability
Maximum total drift	0.01	0.01-0.02	0.02	$0.33V_i/P_i$

Where h is the height of the building, V_i represents the total calculated lateral shear force in storey i, and P_i represents the total gravity load (inclusive of dead load plus likely live load) at storey i.

4.3.1 Regular model (R)

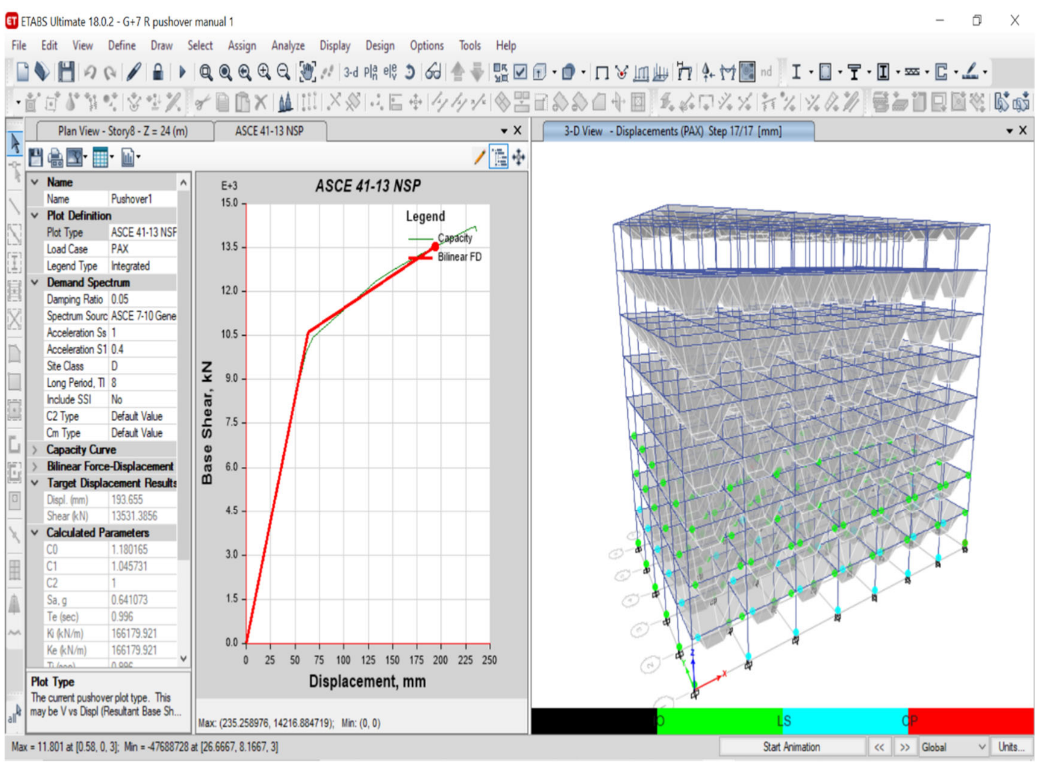


Fig 4.9 Pushover curve & hinge results of R model

Table 4.9 Response of Regular model as per PoA

Target displacement (mm)	Demand base Shear (kN)	Maximum displacement (mm)	Drift
193.655 mm	13531.3856	51.143	0.003594 (Storey 2)

Table 4.10 Hinge steps in each step of pushover analysis for R model

Step	Δ roof top	Vb	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN					
0	0	0	2848	0	0	0	2848
1	51.12	8495.85	2848	0	0	0	2848
2	61.44	9904.01	2848	0	0	0	2848
3	68.77	10446.38	2848	0	0	0	2848
4	132.40	12333.82	2669	176	1	2	2848
5	132.40	12333.83	2669	176	1	2	2848
6	147.84	12669.22	2668	177	1	2	2848
7	147.85	12669.43	2668	177	1	2	2848
8	179.56	13287.70	2584	257	5	2	2848
9	179.56	13285.11	2584	257	5	2	2848
10	179.57	13286.05	2584	257	5	2	2848
11	**230.90**	**14180.29**	**2572**	**248**	**18**	**10**	**2848**
12	235.22	14215.59	2564	249	23	12	2848
13	235.26	14216.88	2564	249	23	12	2848
14	235.26	14216.81	2564	249	23	12	2848
15	235.41	14202.54	2564	239	33	12	2848
16	236.02	14074.61	2564	226	46	12	2848
17	236.05	14077.69	2564	225	47	12	2848

From fig 4.9, it is seen that most hinges are formed on the beams. Table 4.10 shows the development of hinges at each step of analysis. The performance point taken at step 11 (actual between 10 & 11), 99% of hinges are within LS limits and 90% within IO performance level. A $\Delta_{\text{roof top}}$ of 193.655 mm, and height of building 24m, gives a $\Delta_{\text{roof top}}$ to h ratio of 0.0081 which lies within performance level of IO as per table 4.8 of this section

4.3.2 Stiffness irregularity Model

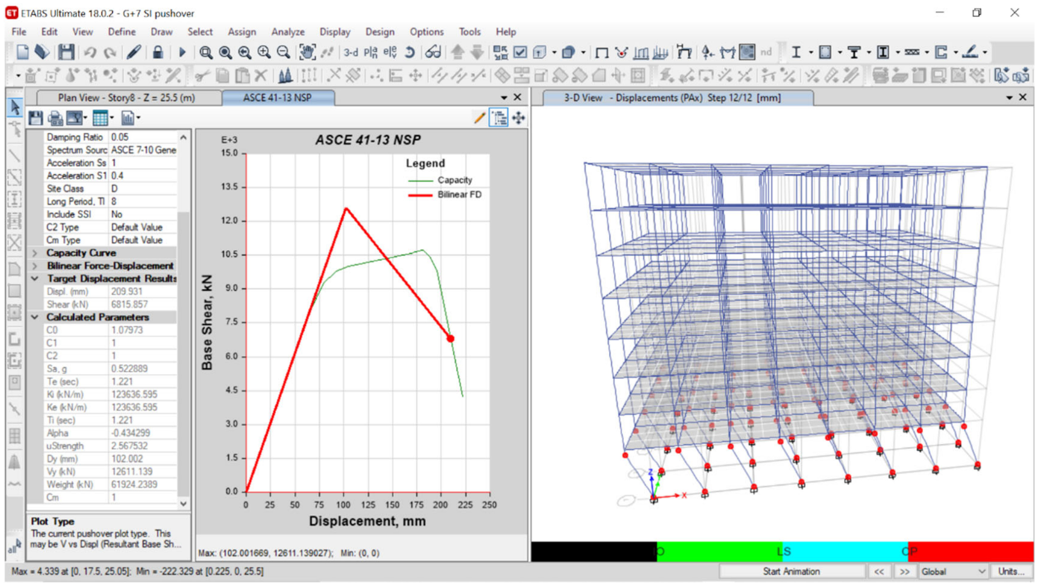


Fig 4.10 Pushover curve & hinge results of SI model

From fig 4.10, it is seen most hinges are formed on the columns. The deformed shape of the SI model is a clear demonstration of soft storey phenomenon in the first floor which will eventually lead to the structure's collapse in the event of a seismic activity.

Table 4.11 Response of SI model as per PoA

Target displacement (mm)	Demand base Shear (kN)	Maximum displacement (mm)	Drift
209.931 mm	6815.857	63.282	0.004926 (Storey 1)

Table 4.12 Hinge steps in each step of pushover analysis for SI model

Step	Δ roof top mm	Vb kN	A-IO	IO-LS	LS-CP	>CP	Total
0	0	0	4160	0	0	0	4160
1	63.28	7824.05	4160	0	0	0	4160
2	78.87	9257.46	4160	0	0	0	4160
3	92.27	9790.37	4160	0	0	0	4160
4	104.57	10011.36	4160	0	0	0	4160

5	168.79	10577.53	3968	104	56	32	4160
6	170.80	10620.40	3968	82	36	74	4160
7	179.53	10722.69	3968	46	56	90	4160
8	180.58	10723.29	3968	46	54	92	4160
9	181.39	10713.36	3968	40	54	98	4160
10	188.34	10428.23	3968	6	66	120	4160
11	195.35	9840.24	3968	0	36	156	4160
12	**222.33**	**4244.01**	**3968**	**0**	**0**	**192**	**4160**

From the table 4.12, performance point taken at step 12 (actual between 11 & 12), 95.38 % of hinges are within IO limits and approx. 5% have crossed the Collapse Prevention performance level. The fig 4.11 above shows development of hinges beyond CP in columns of storey 1 which clearly is due to the soft storey effect. Hence retrofitting is required in storey 1 to stiffen the columns. A $\Delta_{\text{roof top}}$ of 209.931 mm, with height of building 25.5 m, gives a $\Delta_{\text{roof top}}$ to h ratio of 0.008233 which lies within performance level of IO as per table 4.8 of this section

4.3.3 Mass irregularity model

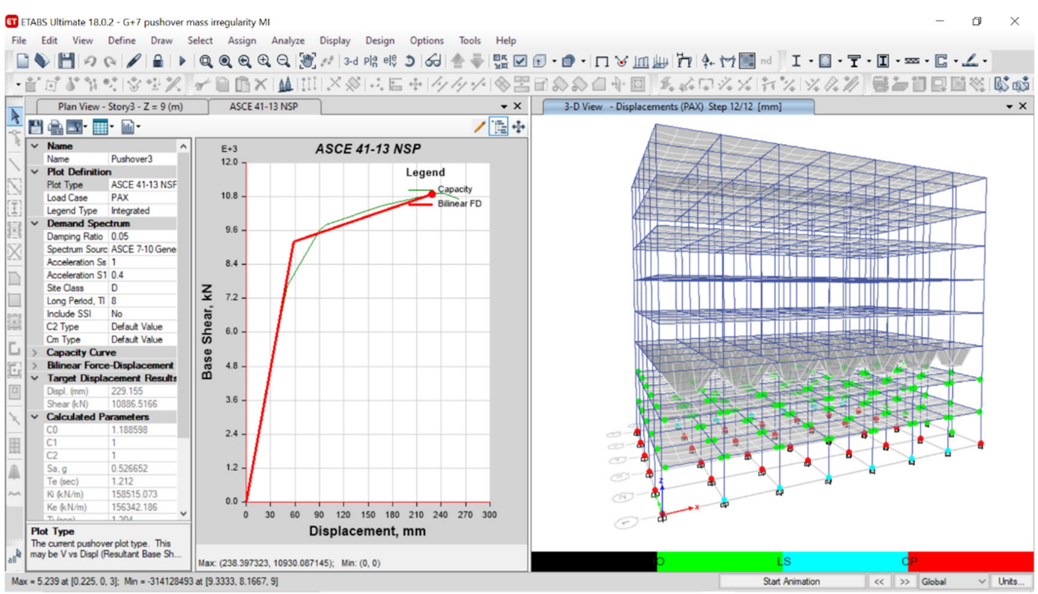


Fig 4.11 Pushover curve & hinge results of MI model

Table 4.13 Response of MI model as per PoA

Target displacement (mm)	Demand base Shear (kN)	Maximum displacement (mm)	Drift
229.155 mm	10886.5166	30.69	0.002454(storey 2)

Table 4.14 Hinge steps in each step of pushover analysis for MI model

Step	**Δ roof top**	**Vb**	**A-IO**	**IO-LS**	**LS-CP**	**>CP**	**Total**
	mm	kN					
0	0.00	0.00	4160	0	0	0	4160
1	30.69	4865.53	4160	0	0	0	4160
2	50.41	7648.50	4160	0	0	0	4160
3	90.47	9607.01	4160	0	0	0	4160
4	98.06	9797.49	4158	0	0	2	4160
5	168.22	10497.98	4098	60	0	2	4160
6	**233.76**	**10915.88**	**3896**	**206**	**54**	**4**	**4160**
7	238.40	10930.09	3884	216	40	20	4160
8	239.85	10928.82	3868	232	26	34	4160
9	242.10	10921.16	3828	270	20	42	4160
10	243.18	10915.13	3820	278	14	48	4160
11	261.13	10722.18	3728	346	24	62	4160
12	261.65	10715.49	3728	346	24	62	4160

In fig 4.11, it is visible most hinges are formed on the beams. From the table 4.14, performance point taken at step 6 (actual between 5 & 6), 98.6 % of hinges are within LS limits and 93.6 % is within IO performance level. A $\Delta_{\text{roof top}}$ of 229.155 mm, with height of building 24 m, gives a $\Delta_{\text{roof top}}$ to h ratio of 0.0095 which lies within performance level of IO as per table 4.8 of this section

4.3.4 Vertical geometric irregularity model

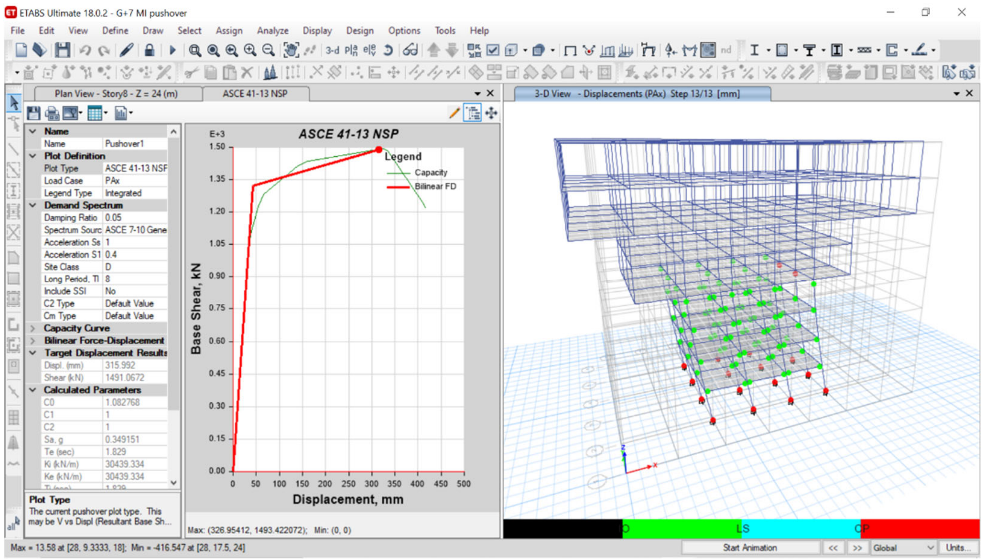


Fig 4.12 Pushover curve & hinge results of VG model

Table 4.15 Response of VG model as per PoA

Target displacement (mm)	Demand Base Shear (kN)	Maximum displacement (mm)	Drift
315.992 mm	1491.0672	31.566	0.001937(storey 2)

Table 4.16 Hinge steps in each step of pushover analysis for VG model

Step	Δ roof top	Vb	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN					
0	0.00	0.00	2408	0	0	0	2408
1	31.57	960.85	2408	0	0	0	2408
2	35.22	1054.34	2408	0	0	0	2408
3	39.15	1112.61	2408	0	0	0	2408
4	53.92	1226.42	2408	0	0	0	2408
5	66.76	1282.40	2408	0	0	0	2408
6	141.02	1415.30	2408	0	0	0	2408
7	161.65	1435.19	2384	24	0	0	2408

8	290.25	1481.73	2200	184	24	0	2408
9	**321.81**	**1493.18**	**2200**	**176**	**32**	**0**	**2408**
10	326.95	1493.42	2200	176	24	8	2408
11	334.55	1483.45	2200	176	8	24	2408
12	413.20	1237.64	2200	172	0	36	2408
13	416.54	1220.50	2200	172	0	36	2408

In fig 4.12, it is seen that most hinges are formed in the columns in the lower storeys. From table 4.16 above, performance point taken at step 9 (actual between 8 & 9), 91% of hinges are within IO limits and 98.7% under LS performance level. A $\Delta_{\text{roof top}}$ of 315.992 mm, with height of building 24 m, gives a $\Delta_{\text{roof top}}$ to h ratio of 0.0132 which lies within performance level of Damage control (between IO and LS) as per table 4.8 of this section.

4.3.5 Combination of stiffness and mass irregularity model CO-I

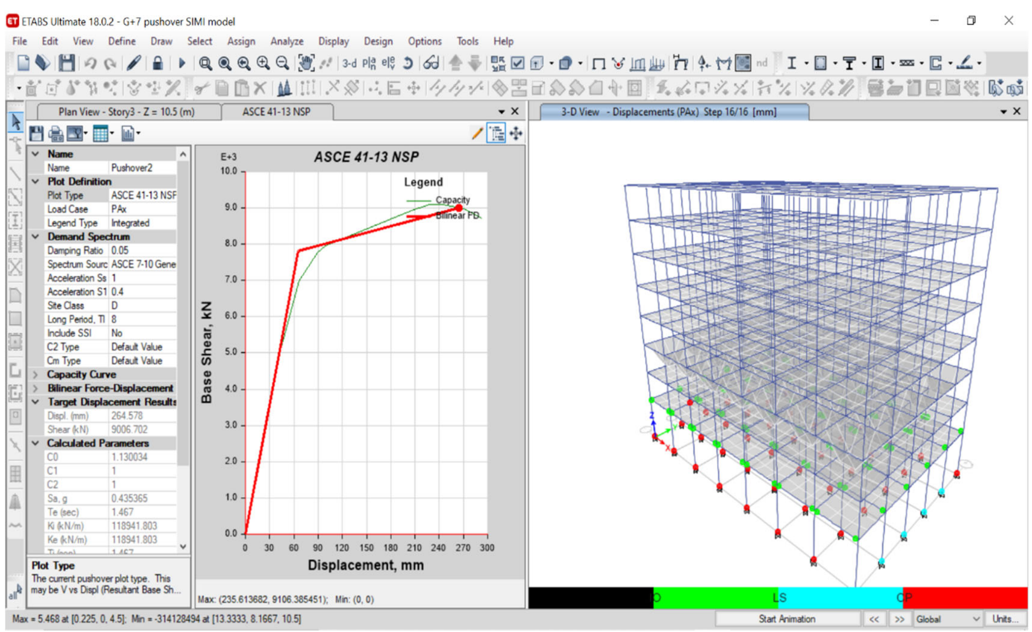


Fig 4.13 Pushover curve & hinge results of CO-I model

Table 4.17 Response of CO-I model as per PoA

Target displacement (mm)	Demand base Shear (kN)	Maximum displacement (mm)	Drift
264.578 mm	9006.702	40.669	0.003239(Storey 1)

Table 4.18 Hinge steps in each step of pushover analysis for CO-I model

Step	Δ roof top	Vb	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN					
0	0	0	4160	0	0	0	4160
1	40.67	4837.28	4160	0	0	0	4160
2	66.68	6979.44	4160	0	0	0	4160
3	89.57	7777.63	4160	0	0	0	4160
4	99.48	7957.37	4160	0	0	0	4160
5	205.45	8936.51	4064	54	42	0	4160
6	227.36	9088.31	3912	192	40	16	4160
7	230.83	9102.43	3896	208	30	26	4160
8	233.00	9105.59	3896	208	20	36	4160
9	235.61	9106.39	3896	204	18	42	4160
10	242.34	9097.70	3896	204	12	48	4160
11	244.61	9092.94	3896	200	12	52	4160
12	**269.86**	**8983.90**	**3896**	**176**	**28**	**60**	**4160**
13	288.27	8775.69	3896	168	12	84	4160
14	288.28	8775.55	3896	168	12	84	4160
15	292.04	8729.09	3894	168	9	89	4160
16	292.04	8728.88	3894	168	9	89	4160

From table 4.18 above, performance point taken at step 12 of the analysis (actual between 11 & 12), 93.7% of hinges are within IO limits and 97.9% under LS performance level. While 1.4% of hinges presumably in storey 1 have crossed CP level. A $\Delta_{\text{roof top}}$ of 264.578 mm, with height of building 25.5 m, gives a $\Delta_{\text{roof top}}$ to h ratio of 0.0104 which lies within performance level of Damage control as per table 4.8 of this section.

4.3.6 Combination of stiffness and vertical geometric irregularity model (CO-II)

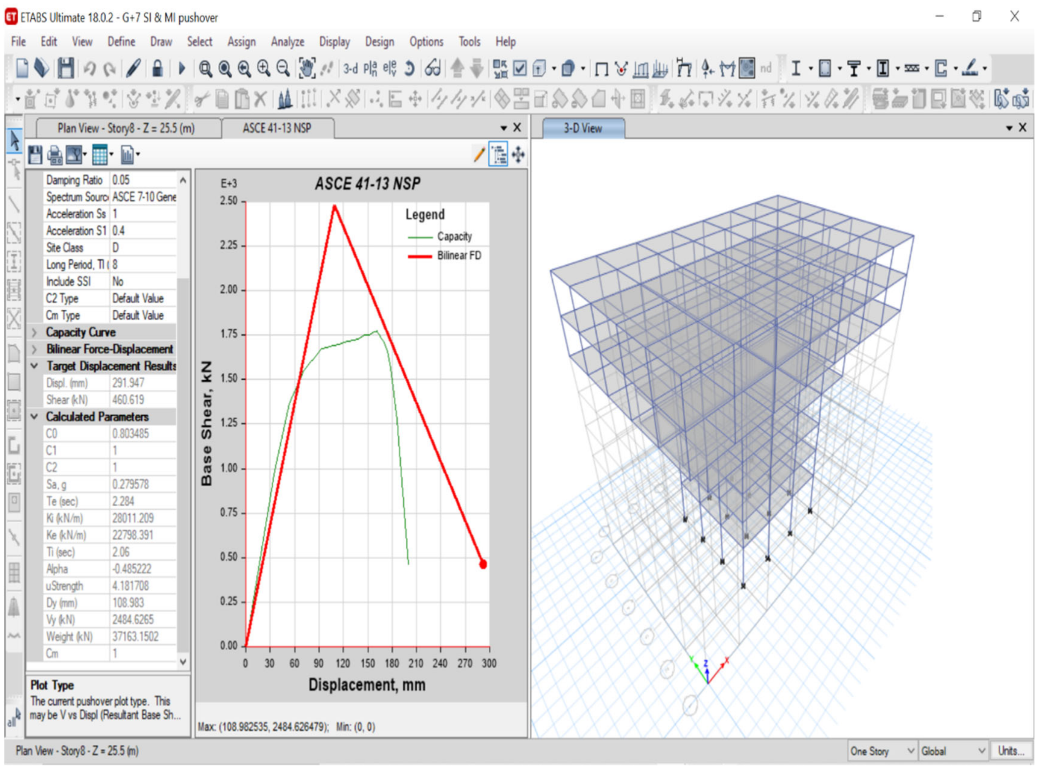


Fig 4.14 Pushover curve of CO-II model

As per Fig 4.14 above, the CO-II model is found to be seismically weak denying the structure of a performance point. Therefore, hinge results also could not be generated.

Table 4.19 Response of CO-II model as per PoA

Target displacement (mm)	Demand base Shear (kN)	Maximum displacement (mm)	Drift
291.947 mm	460.619	35.42	0.002111 (Storey 1)

Table 4.20 Hinge steps in each step of pushover analysis for CO-I model

Step	Δ roof top	Vb	A-IO	IO-LS	LS-CP	>CP	Total
	mm	kN					
0	0	0	2408	0	0	0	2408
1	35.42	992.17	2408	0	0	0	2408
2	53.91	1363.11	2408	0	0	0	2408
3	70.86	1548.98	2406	0	0	2	2408
4	93.52	1671.77	2406	0	0	2	2408
5	99.84	1677.78	2398	8	0	2	2408
6	102.65	1683.86	2378	24	0	6	2408
7	107.80	1690.29	2372	30	0	6	2408
8	115.30	1696.01	2372	30	0	6	2408
9	120.59	1712.29	2356	46	0	6	2408
10	137.33	1728.68	2348	54	0	6	2408
11	145.71	1748.93	2344	34	24	6	2408
12	151.00	1754.37	2344	34	24	6	2408
13	158.58	1771.43	2344	28	30	6	2408
14	159.89	1771.89	2344	28	24	12	2408
15	160.94	1772.92	2344	16	34	14	2408
16	161.20	1772.89	2344	16	34	14	2408
17	161.73	1771.00	2344	16	28	20	2408
18	170.71	1712.34	2344	4	24	36	2408
19	174.60	1658.15	2344	4	16	44	2408
20	178.56	1569.56	2344	2	6	56	2408
21	186.24	1271.90	2344	0	4	60	2408
22	199.68	460.62	2344	0	0	64	2408

As seen in fig 4.14 and table 4.20, the analysis result could not yield the target displacement of 291.947 mm presumably due to weak structure of the model. However, in the final step of the analysis corresponding to a displacement value of

199.68 mm, it is observed that 97.34% of hinges are in IO level and 2.66% of hinges have crossed CP performance level. The results show poor seismic performance of the model.

4.4 COMPARISON OF RESPONSES AS DERIVED FROM PUSHOVER ANALYSIS

4.4.1 Normalised base shear-roof top displacement relationship

Table 4.21 Target displacement and Demand base shear

Model	Target displacement (mm)	% w.r.t to height	Demand base shear (kN)	Capacity base shear (kN)
R	193.655	0.81	13531.3856	14077.69
SI	209.931	0.82	6815.857	4244.01
VG	315.992	1.32	1491.0672	1220.50
MI	229.155	0.95	10886.5166	10715.49
CO-I	264.578	1.04	9006.702	8728.88
CO-II	291.947	1.22	460.619	460.62

The target displacement values of all the models calculated from Pushover Analysis lies within 0.8 to 1.5% of the structure storey height. The target displacement value of the R model is found to be the lowest while the VG model had the highest target displacement value. From the pushover curves of all models, the demand base shear is shown in red line and the capacity is shown in green line. From table 4.21, it is seen that regular model has the highest capacity in terms of base shear as compared to the other models.

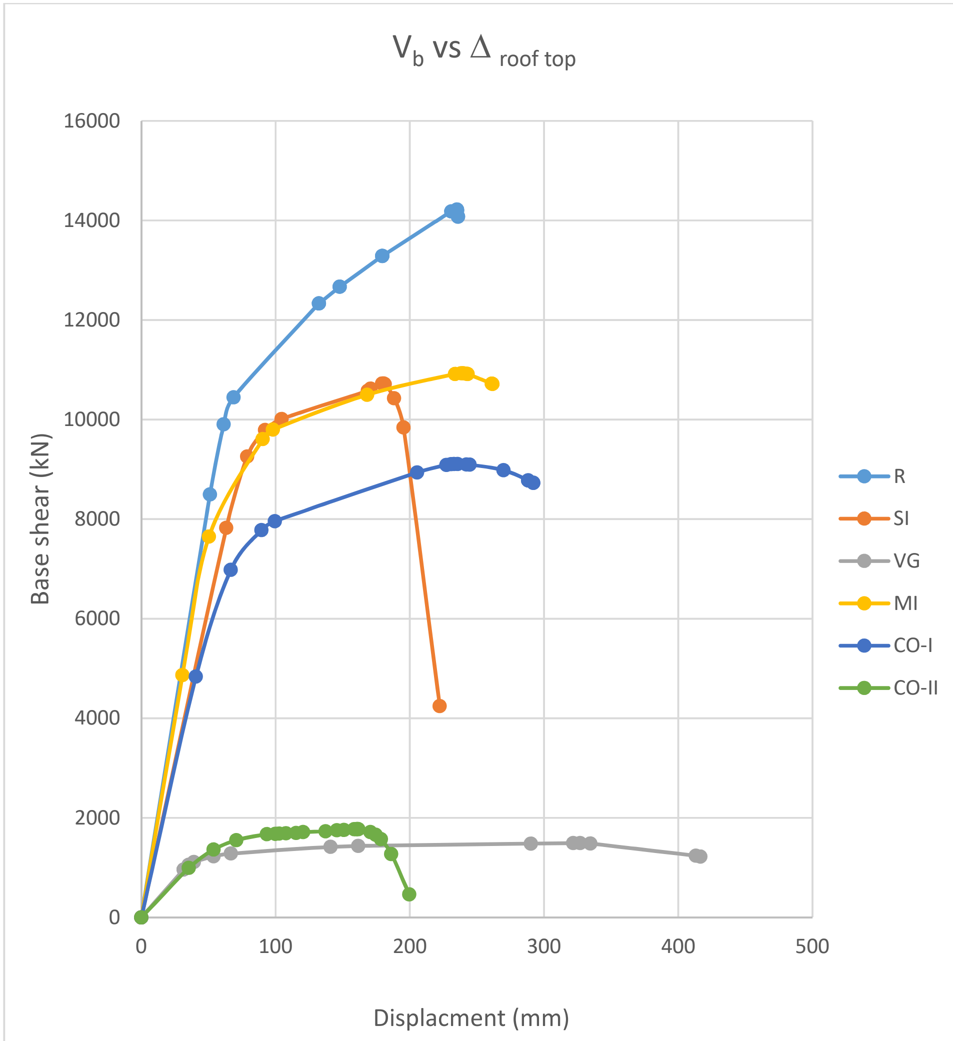


Fig 4.15 Normalised base shear roof top displacement relationship for models

The normalised base shear top displacement relationships obtained by pushover analysis for all models is presented in fig 4.15. It is observed that the VG model had good displacement capacity but less lateral strength. The CO-I model exhibited a more rigid behaviour. The regular model has the highest capacity in terms of base shear as compared to the other models.

4.4.2 Maximum storey displacement

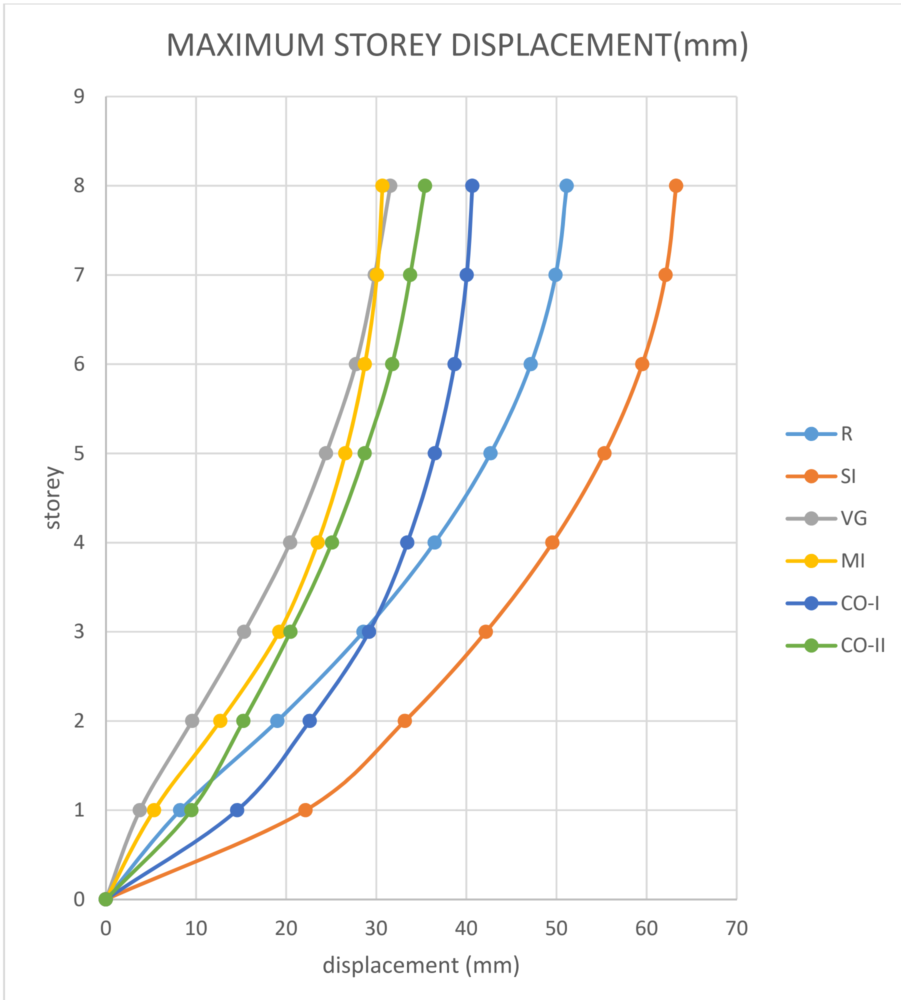


Fig 4.16 Maximum storey displacement as per PoA

The model VG shows the best performance in terms of overall lateral displacement while the model MI has the minimum top storey displacement of 30.69 mm. the model SI depicts the worst performance with top storey displacement of 63.28 mm. The displacement values derived from PoA showed higher values for models R, SI, VG and lower values for MI, CO-I and CO-II than response spectrum analysis.

4.4.3 Storey drift

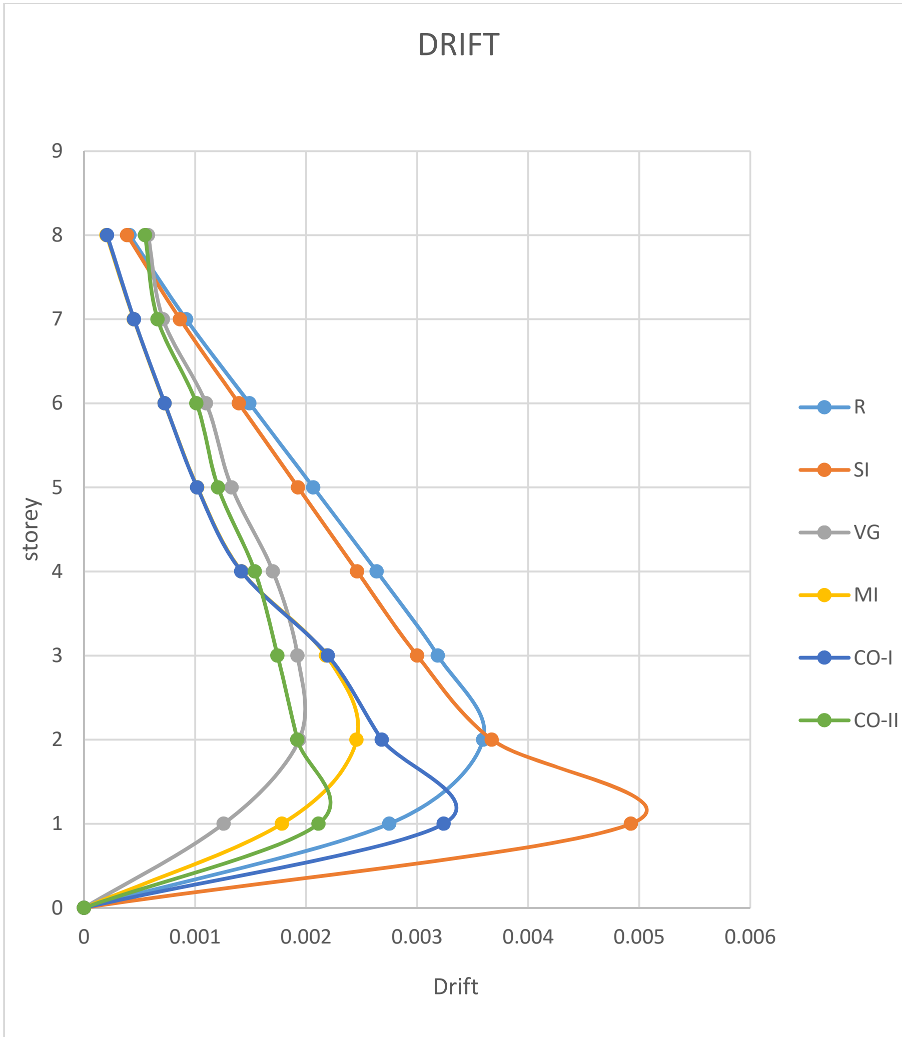


Fig 4.17 Storey Drift as per PoA

The SI model showed a drastic decrease in the drift value in the upper storey as compared to storey 1. The CO-I model showed less drift in the upper storeys and VG model showed less drift in lower storeys. The drift values are found to be within acceptable limits as per table 4.8 of this section.

4.4.4 Sesimic weight and base shear

Table 4.22 Seismic weight and corresponding base shear as per PoA

Building model	R	SI	VG	MI	CO-I	CO-II
Base Shear (kN)	1122.404	3454.887	620.5103	1540.737	1544.082	678.3524
seismic weight (kN)	61133.12	61315.31	33796.86	83918.12	84100.31	36947.3

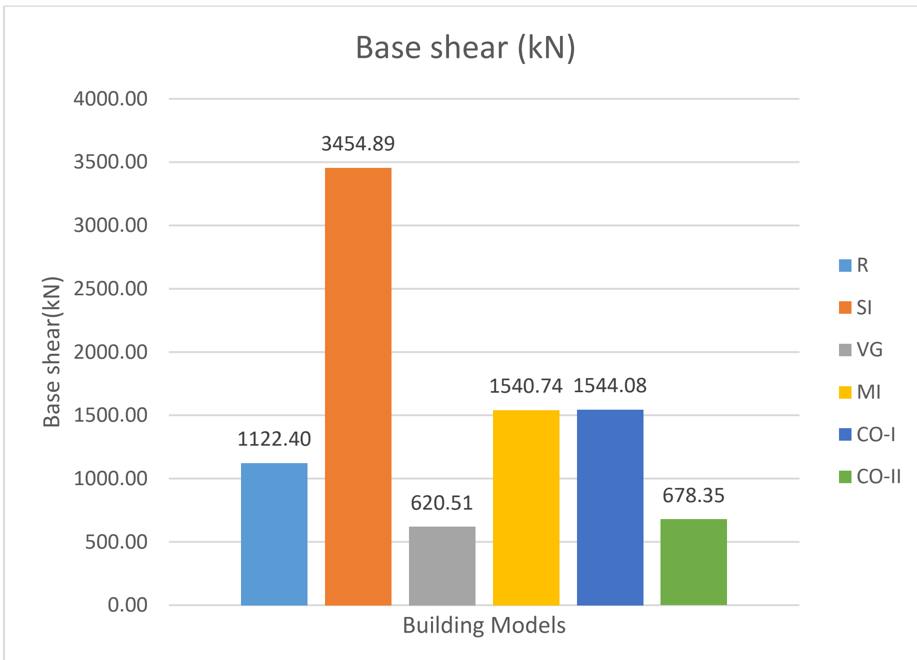


Fig 4.18 Base shear as per PoA in x direction

The SI model has the maximum base shear value while the VG model has the lowest base shear value. The values of base shear as derived from PoA is lower than RSA for the same seismic weight.

CHAPTER 5

CONCLUSIONS

- The Pushover analysis represents non linear behavior of structures and is accurate but does not take into consideration the effects of higher modes hence this method is recommeded only for structures where the higher modes are insignificant.
- The IS Code measures to quantify irregularity is inadequate as they have classified irregularity based on type and magnitude and ignored the location. However, review of literature (Siva Naveen et al 2019) show that the location of irregularity has a significant influence on the seismic response. So, the type, magnitude and location of irregularity should be taken into consideration.
- The code proposed empirical formula for calculating the fundamental time period ($Ta=0.09h/d^{1/2}$) is the same for all models except VG and CO-II model as it depends on the base dimensions and height. However, the time period generated by ETABS is varying for each model which indicated that time period is dependent upon the irregularity.
- The seismic weight of all models vary due to change in degree and location of irregularity, which has a direct impact on the value of base shear. For the same seismic weight, the base shear values generated from PoA had lower values than RSA.
- The drift limit of 0.004 H as described in IS 1893(Part1): 2016 is more relaxed than the drift limit of ATC 40 (table 4.8). The drift values of all models were within the prescribed limits.

- The model CO-II which is a combination of Stiffness and vertical geometric irregularity is found to be seismically weak hence denying the structure of a performance point in Pushover analysis. Hence, it can be inferred that the model CO-II is the most critical and vulnerable of all models.
- The presence of irregularities in a structure considerably affects the seismic behavior which can significantly alter the performance of a building.

FUTURE SCOPE OF WORK

Many literature have proposed improved pushover analysis procedures due to limitations of the conventional pushover procedure. For the future work, the improved pushover procedures can be explored. The study can also be extended to include other structural configurations.

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