EFFECT OF DIAPHRAGM DISCONTINUITY IN THE SEISMIC RESPONSE OF A REGULAR RC FRAME MULTI-STOREYED BUILDING

Dissertation submitted in the partial fulfilment of the requirement for the award of

MASTER OF TECHNOLOGY
(STRUCTURAL ENGINEERING)

Submitted by
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CERTIFICATE

This is to certify that the thesis entitled **"Effect of Diaphragm Discontinuity in the Seismic Response of Regular RC Frame Multi-Storeyed Building"** submitted by **Mr. Sidhartha Kumar** in partial fulfilment of the requirements for the award of Master of Technology Degree in Civil Engineering with specialization in Structural Engineering at the Delhi Technological University is an authentic work carried out by him under my supervision. To the best of my knowledge, the matter embodied in the thesis has not been submitted to any Other University/Institute for the award of any degree or diploma.

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This is to certify that the above statement laid by the candidate is correct to the best of our knowledge.

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SIDHARTHA KUMAR

ABSTRACT

Earthquakes are natural calamity which causes severe damage or collapse of buildings. Now a day's many structures have irregular configurations both in plan and elevation. Damage due to earthquake are more severe at the point of discontinuity in the structure. Openings in the floors are common for many reasons like staircases, lighting, architectural and etc. these openings develop stresses at discontinuities. Discontinuous diaphragms are designed without stress calculations and are thought-about to be adequate ignoring any gap effects. In multi-storeyed framed building, damages from earthquake generally initiates at locations of structural weaknesses present in the lateral load resisting frames Diaphragms with abrupt discontinuities or variations in stiffness, which includes those having cut-out or open areas greater than 50 percent of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50 percent from one storey to the next. In structural engineering, a diaphragm is a structural system used to transfer lateral loads to shear walls or frames primarily through in-plane shear stress. Lateral loads are usually wind and earthquake loads. This paper focuses the general effects of diaphragm discontinuity on seismic response of multi-storeyed building on various structural parameters.

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Earthquake usually initiates in multi storeyed framed structures at locations of structural weaknesses gift within the lateral load resisting frames. This behaviour of multi-storey framed buildings throughout vigorous earthquake motions depends on the distribution of mass, stiffness, strength in each the horizontal and vertical planes of buildings. In few cases, these weaknesses is also created by discontinuities in stiffness, strength or mass on the diaphragm.

Such discontinuities between diaphragms are usually related to sharp variations within the frame pure mathematics on the length of the building. Structural engineers have developed confidence in the style of buildings during which the distributions of stiffness, mass and strength are a lot of or less uniform. There is a less confidence concerning the look of structures having irregular geometrical configurations or diaphragm discontinuities.

In this present thesis, the result of diaphragm separation on the seismic response of a particular multi construction building is studied.

1.1.1Diaphragm Discontinuity

As per IS-1893:2002: Diaphragms with unexpected discontinuities or variations in stiffness, which includes those having cut-out or open areas greater than 50 % of the gross surrounded diaphragm area, or changes in effective diaphragm stiffness of more than 50 % from one storey to the next.

In structural engineering, a diaphragm may be a structural system used to transfer lateral loads to frames or shear wall primarily through in-plane shear stress. Lateral loads are generally wind and earthquake loads. Two primary kind of diaphragm are rigid and flexible. Flexible diaphragms resist horizontal forces depending on the area, regardless of the flexibility of the members that they are transmitting force to. Rigid diaphragms transfer load to shear walls or frame depending on their flexibility and their location within the structure. Flexibility of a diaphragm impinges the distribution of lateral forces to the vertical parts of the lateral force resisting parts in a very structure.

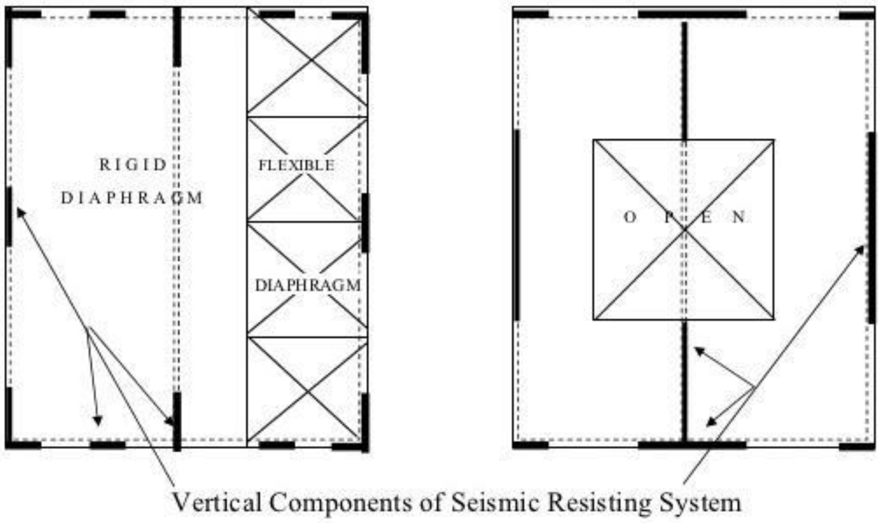


Fig 1: Rigid and flexible diaphragms

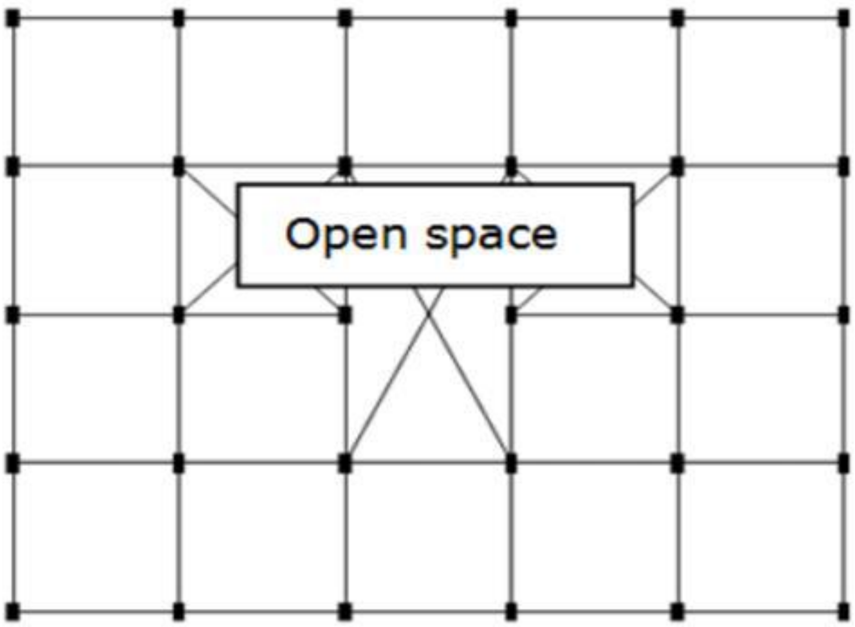


Fig 2: Frame with Diaphragm Discontinuity

1.2 OBJECTIVES

A detailed literature review is distributed to outline the objectives of the thesis. The literature review is mentioned well in Chapter two and shortly summarized as follows:

i) International Building Code (IBC) recommends that for buildings with diaphragm separation, the code recommends an increase of twenty five percent inside the design forces found for the connections of diaphragms.

ii) American Concrete Institute Building Code, I 318-08 does not address the results of a spot on the ground.

iii) ASCE 7-05, Section 12.3.1.2, permits diaphragms of RCC slabs or concrete crammed metal decks with span-to-depth ratios of (3:1) or less.

iv) Nakashima et al. Analysed a multi structure RC building victimisation non-linear analysis last that the enclosure of diaphragm flexibility did not significantly modification the actual quantity of the structure and so the foremost total base shear.

Based on the literature review, the relevant objective of the present study have been identified as follows:

1. To analyse the seismic performance of a multi-story building with different diaphragms i.e., model-1 to model-7 through a close case study.
2. To see effect of diaphragm discontinuity on 7 different models.

1.3 SCOPE OF THE PRESENT STUDY

In the present study, a typical G+17 multi storey building is analysed using commercial software's ETABS and SAP2000.

ETABS in this project is used to perform linear static analysis and response spectrum analysis and SAP2000 for nonlinear static (pushover) analysis. All the analyses have been carried out allowing for and ignoring the diaphragm discontinuity and also the results therefore obtained were compared. This study is accomplished for RC framed multi storeyed building having fixed support conditions. The results of this report relies on one case-study.

1.4METHODOLOGY

a) A radical literature review to grasp the seismic analysis of building structures and application of pushover analysis and time.

b) Choose associate existing building with diaphragm separation.

c) Style the building as per prevailing Indian normal for dead load, live load, wind load and earthquake load.

d) Analyse the building victimization linear/nonlinear static/dynamic analysis strategies.

e) Analyse the results and gain conclusions.

1.5 ORGANIZATION OF THE THESIS

The thesis is prepared as per detail given below:
Chapter 1: Introduces to the subject of thesis in brief. Diaphragm discontinuity is defined and codal provisions are mentioned. Seismic effects on various types of structures are also discussed briefly

Chapter 2: Deliberates the literature review i.e. the work done by varied researchers within the area of diaphragm discontinuity of building. Results and conclusions of different research work is mentioned which helps to determine the aim of this thesis.

Chapter 3: Modelling of the building has been mentioned during this chapter.

Chapter 4: During this chapter pushover analysis has been studied intimately. The idea and procedure of pushover analysis mentioned in short.

Chapter 5: In this chapter theory and procedure related to Response spectrum analysis has been discussed in brief.

Chapter 6: The results from nonlinear static push over analysis and response spectrum analysis were studied.

Comparison between the 7 different models are done and conclusion is given followed by references.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

To provide an in depth review of the literature regarding diaphragm Discontinuity in its completeness would be tough to deal with here. A short review on diaphragm discontinuity of previous studies is given here. This literature review focuses on recent contributions regarding diaphragm and past efforts most closely regarding the wants of this work.

2.2 LITERATURE REVIEW

International Building Code-2006, desires the diaphragm with unanticipated discontinuities or variations in stiffness, also those having cut-out or open areas greater than 50 % of the gross bounded diaphragm area, or change in effective diaphragm stiffness of over 50 % from one story to subsequent, to be considered as irregular in plan. For structures with this diaphragm discontinuity, the code prescribes an increase of 25% within the design forces determined for connections of diaphragms to vertical components. The code does not attribute any criteria relating the diaphragm vogue itself.

In the space of concrete style, American Concrete Institute code ACI 318-08, addresses the impact of a niche on slabs in native terms. It restricts gap size in column strips and limits the allowable maximum openings size in middle strips. The interrupted reinforcement by a niche ought to be placed at common fraction on either side of the gap. ACI 318-08 does not address the final impact of a niche on the ground. This reinforcement replacement criterion has no restriction on the gap size as long as a result of it's among the prescribed column and middle strips demand.

ASCE 7-05, the Guide to arranging the design the of Diaphragms permits diaphragms of concrete slabs or concrete stuffed metal decks with span-to-depth magnitude relation of 3:1 in structures that have no horizontal plan irregularities to be perfect as rigid, otherwise, the structural analysis shall expressly embody thought of the stiffness of the diaphragm whereas not explaining but in the field of concrete beams with web openings, Nasser et. al. (1993), Mansur et. al. (1999) and Abdalla and Kennedy (1988) shed light-weight on but a spot in rectangular RC pre-stressed beams affects stress distributions and capability of a concrete beam. Sadly, the speculation provided was mark against accessible experimental results with no proof that it's extended to include various configurations.

Kato et. al. (1991), Taylor et. al. (1992) and Daisuke et. al. (1959), investigated the look of RC shear walls with one gap. Again, the results were exclusively applicable to the pertinent cases. Alternative studies were conducted at intervals the world of concrete panels, notably at intervals the world of buckling.

Swartz and Rosebraugh (1974), Aghayere and malefactor (1971), and Park and Kim (1992) self-addressed buckling of concrete plates to a lower place combined in-plane and cross loads.

Since concrete diaphragms is thought-about as concrete plates with beams as net stiffeners, this buckling approach does not address openings.

Button et. al. (1984) investigated the influence of floor diaphragm flexibility on three wholly completely different buildings, huge prepare ratio, three-winged (Y-shaped) and separate towered. Notwithstanding the insight given into but lateral force distribution differs from rigid to flexible diaphragms, openings weren't thought-about.

Basu (2004), Jain (1984) and Tao (2008) had analysed different types of structures ranging from fashioned, Y-shaped to long and slender buildings. Though these studies proven to be conductive to understanding the dynamics of such type of structures, they did not address the results of diaphragm openings.

Kunnath et. al. (1991) developed a modelling theme for the dead response of floor diaphragms, and Reinhorn et. al. (1992) and Panahshahi et. al. (1988) verified it, mistreatment shake table testing for single-story RC, 1:6 scaled model structures, gap effects weren't incorporated at intervals the model and additionally the projected model's ability to account for in-plane diaphragm deformations, confirmed the prospect of building collapse, as a results of diaphragm yielding for low rise (one-, two-, and three-story) rectangular buildings with end shear walls and building arrange ratio larger than 3:1. Nakashima et. al. (1984) analysed a seven story RC building exploitation linear and non-linear analysis final that the inclusion of diaphragm flexibility did not significantly change the actual quantity of the structure and additionally the foremost total base shear. Effects of diaphragm openings weren't a region of that analysis.

Anderson et. al. (2005) developed analytical models mistreatment industrial laptop programs, SAP 2000 and ETABS to evaluate the seismic performance of walk-up buildings with concrete walls and versatile diaphragms. Again, openings weren't a region of the models devised.

Barron and Hueste (2004) evaluated the impact of diaphragm flexibility on the structural response of four buildings having 2:1 and 3:1 discovered set up ratios and were 3 and five stories tall, severally. The building diaphragms did not yield and additionally the buildings in question did not have diaphragm openings.

Hueste and Bai (2004) analysed a model five-story RC frame building designed for the mid-1980s code desires inside the Central United States. Recommending Associate in nursing addition of shear walls and RC columns jackets LED to decrease inside the probability of surpassing the life safety (LS) limit state. Sadly, retrofitting recommendations were specific to this structure entirely and no diaphragm gap effects were looked into.

Kunnath et al. (1987) developed associate analytical modelling theme to assess the damageability of RC buildings experiencing non resilient behaviour beneath earthquake forces. The results of the response analysis, expressed as harm indices, failed to give any reference to diaphragm openings.

Jeong and Elnashai (2004) projected a three-dimensional unstable assessment methodology for plan-irregular buildings. The analysis showed that plan-irregular structures suffer high levels of earthquake harm owing to torsional effects. The analysis in addition verified that normal harm observation approaches would be inaccurate and even non-conservative. However, the assessment failed to account for diaphragm openings.

Ju & Lin (1999) and Moeini (2011) investigated the excellence between rigid floor and versatile floor analyses of buildings, victimization the finite component technique to analyse buildings with and whereas not shear walls. a blunder formula was generated to estimate the error in column forces for buildings with set up regular arrangement of shear walls at a lower place the rigid floor assumption. Though 520 models were generated, none forbidden diaphragm openings.

Kim and White (2004) planned a linear static methodology applicable entirely to buildings with versatile diaphragms. The procedure relies on the concept that diaphragm stiffness is little compared to the stiffness of the walls, that versatile diaphragms inside a building structure tend to retort severally of 1 another. Though the planned approach gave insight into the restrictions of current building codes, it failed to influence diaphragm gap effects. Other connected analysis addresses the consequence of assuming a rigid floor on lateral force distribution.

Roper and Iding (1984) in short examined the appropriateness of assuming that floor diaphragms area unit completely rigid in their plane. 2 models were used, the first was for a cruciform-shape building and additionally the second was for an oblong building. Each models showed discrepancy between rigid and versatile floor diaphragm lateral force distribution. Specially, once shear walls exhibit associate abrupt change in stiffness. Still, effects of openings on lateral force distribution weren't explored.

Tokoro et al. (2004) replicated associate existing instrumented three story building victimization in ETABS and compared the model's diaphragm drift to the code allowable drift and judged the structure to be among the code's given drift limit; whereas not considering any diaphragm gap effects.

Saffarini and Qudaimat (1992) analytically investigated xxxvii buildings, with diaphragm lateral deflection and inter-story shears as a comparison criterion between rigid and versatile diaphragms assumptions. The analysis showed wide distinction inside the diaphragms' deflections and shears. The investigation in short addressed gap effects as a locality of various parameters being studied. It completely was terminated that a niche completely ablated the ground stiffness, and thence raised the inadequacy of the rigid floor assumption.

Easterling associated Porter (1992) given the results of an experimental analysis program throughout that 32 life-size composite (steel-deck and concrete floor slabs) diaphragms were loaded to failure. The foremost vital analysis contribution was the event of the next vogue approach for composite floor systems and stressing the importance of malformed bars reinforcing to spice up plasticity and management cracking relating to concrete failure around headed studs. The recommendations were entirely pertinent to the cantilevered diaphragms tested and no gap effects were examined.

Lastly, inside the world of formed concrete and parking structures,Rodriguez et. Al. (2007) compared ASCE 7-05 unstable forces to get shake table forces for a selected systems in question whereas not investigation openings.

Lee and Kuchma (2008) and Wan et. al. (2005) looked into formed concrete diaphragm structures accounting for the ramp cavity and diaphragm connections but ignoring block out-of-plane property and its effects.

The analysis assumes a ply board diaphragm with openings behaves sort of a Vierendeel Truss. Chord parts between shear webs of the Vierendeel Truss square measure assumed to have points of contra flexure at their mid-lengths. Diaphragm segments outside the openings square measure analysed, then segments around the openings analysed second assumptive no openings square measure gift. The procedure is carried-out all over again with the openings thought-about. Finally web amendment in chord forces thanks to openings is achieved by superimposing every results. This technique can satisfy equilibrium conditions, is not altogether reliable.

Kamiya and Itani (1998) investigated the APA technique by horizontally test-loading three plywood-sheathed floor diaphragms designed to the same load. The tests conducted yielded diaphragm shear and deflection equations instead of the long APA technique for those three diaphragms; there was no indication on but their effort is extended to include various configurations.

Philips et. al. (2006) studied but walls transversal to the loading direction in wood-framed buildings share lateral masses. The study shows that such interaction between transversal walls and plywood-sheathed diaphragms can go up as high as twenty five p.c.

Gebremedhin and value (1999) examined but plywood diaphragms distributed lateral masses to frames. Gap effects were tested terribly very custom entirely to state that for walls with openings, the stiffness decrease is not linear with the gap size. For a twenty five % loss in frame space, the wall stiffness attenuated by seventeen % and for a fifty % loss in frame area the stiffness of identical wall attenuated by sixty four %.

Carney (1975) provided a listing on wood and laminate diaphragms analysis going back as way because the 1920's and nearly none self-addressed diaphragm openings. Peralta et al. through an experiment investigated in-plane behaviour of existing wood floor and roof diaphragms in un-reinforced masonry buildings according to components and association details typical for pre-1950 construction. The tip result was style curves shaping the link between the applied lateral force and additionally the diaphragm mid-span displacement. Gap effects on diaphragm stiffness weren't self-addressed either.

Itani and Cheung (1984) introduced a finite part model to analysis the non-linear load deflection behaviour of cased wood diaphragms. The model is general and is in wise agreement with experimental measurements. Withal it's won't subsume openings and but to increase the developed model to account for them.

Pudd and Fonseca (2005) developed a replacement progressive analytical model for sheeting-to-frame connections in wood shear walls and diaphragms. Although the new model isn't like previous analytical models, being acceptable for every monotonic and cyclic analysis, it did not account for the implications of openings on neither shear walls nor diaphragms.

Degenkolb (1959) investigated pitched and flexuous timber diaphragms emphasizing that boundary stresses exist at any break inside the protection plane and will be provided inside the look of a cost-effective diaphragm - no gap effects were thought-about.
Bower written laminate deflection formulas below lateral loading, stating they will that they will| that they're going to be modified to use to any diaphragm type or loading pattern whereas not giving examples.

Westphal and Panahshahi (2002) used three-dimensional finite part models to urge in-plane deformations of wood roof diaphragms and story drift due to seismic load for buildings with set up quantitative relation ranging from one.2 to 1.6. The results obtained show that the anticipated diaphragm deflections by the International codification (IBC) square measure conservative. However, effects of openings on this conclusion weren't investigated.

As for the realm of sunshine gauge steel deck (or metal decks), Nilson (1960) set the benchmark for all future experimental add metal diaphragms. Although the entire tests were intensive, with stress on shear strengths and diaphragm deflections, openings effects were never self-addressed.

Bryan and El-Dakhakhni (1968) have developed Nilson’s work to further general theory for crucial stiffness and strength of sunshine gage metal deck. Still the idea developed did not account for diaphragm openings.

Easley (1975) centred on the buckling facet of furrowed metal shear diaphragms. It had been complete that for many applications, buckling happens once the amount of fasteners is lots therefore localized failure at the fasteners does not occur. However, gap effects on diaphragm buckling weren't looked into.

Davies (1976) developed the way to interchange a metal deck diaphragm by a series of frame components connected by springs. This technique can also be extended to account for openings. A significant disadvantage of this technique is that results obtained square measure strictly linear.

Atrek and Nilson (1980) established a non-linear analysis methodology for lightweight gage steel decks. Results resembled closely out there experimental info, all the same openings weren't self-addressed and no insight was given on the thanks to extend this technique to hide diaphragms excluding the tested ones

Luttrell (1996) recommended a method to urge shear stress distribution around a gap in metal deck diaphragms. The strategy developed would quantitative relation the shear distribution round the gap by the proportion of diaphragm length lost parallel to the loading direction. Hysteretic behaviour has been discovered and studied extensively in picket shear walls.

Fischer et. al. (2001) conducted a complete check structure laboratory experiment and used a nonlinear dynamic response spectrum analysis program Ruaumoko (Carr, 1998) and wood shear walls program. Cashew (Folz and Filiatrault 2000) to make numerical models. Several physical phenomenon models are developed to predict the seismic response of wood-frame structures. Some hysteretic models have created comparatively smart results, however the information collected have sometimes been supported by displacement histories. Records from associate instrumented web site, love California’s sturdy motion stations, solely have acceleration time histories. Extraction of physical phenomenon parameters becomes tougher within the absence of displacement time histories.

R.G. Herrera and C.G. Soberon (2008) showed associate analytical description of the damages caused by completely different arrange irregularities, throughout seismic events of various magnitudes. All told the studied systems, effects of various irregularities square measure analysed supported the variation of displacements, with reference to regular systems

2.3 CONCLUSION

Here a matter arises that what's going to the result if constant building is intended with diaphragm discontinuity and while not diaphragm discontinuity. It's studied during this project. Also the effect of different types of diaphragm discontinuity should be analysed to see the change in the pattern of different parameters like time period, natural frequency, storey drift, stiffness, modal mass participation factor, seismic weight and forces etc.

CHAPTER - 3

MODELING OF BUILDINGS

3.1 INTRODUCTION

In this project, 7 different multi-storeyed buildings with Diaphragm Discontinuity were studied. The structure is modelled and analysed in ETABS from which reinforcing details were appropriated. Further the structure is modelled in SAP 2000 and pushover and response spectrum analysis has been performed from the above appropriated details.

3.2 Details of Structure

Structures are analysed in ETABS software and pushover analysis is performed in SAP-2000.
Design parameters considered for the structure are
Dead Load: - 1.5 KN/m^2
Live Load: - 3.0 KN/m^2
Bricks of density 20 KN/m^3 are used for walls.
Number of stories: - 17
Floor to Floor height: - 3.6 m
Rigid diaphragms are used for slabs.
Wind load is considered as per IS: 875 (Part III)
As per IS: 1893-2002 Earthquake loads are considered
(Special Moment Resisting frame with response reduction factor of 5, Zone IV and 5% damping is provided)
Equivalent static method is used to calculate base shear of the structures manually
Models used in ETABS and SAP 2000 are shown in figure below:

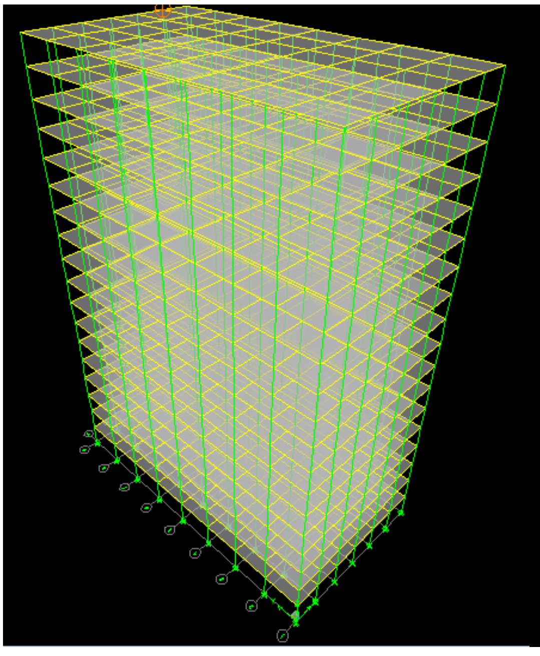


Fig: 3 MODEL 1 (3D VIEW)

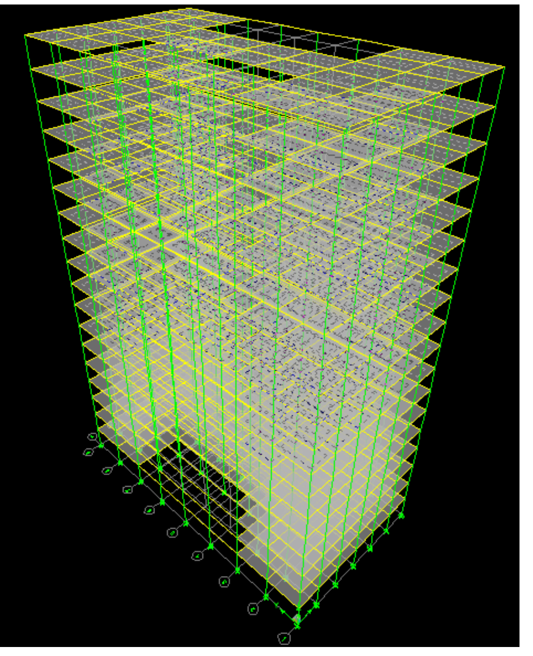


Fig: 4 MODEL 2 (3D VIEW)

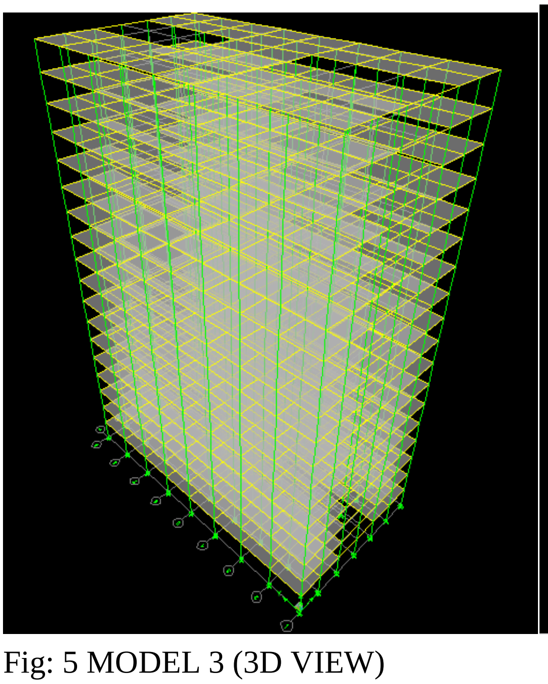


Fig: 5 MODEL 3 (3D VIEW)

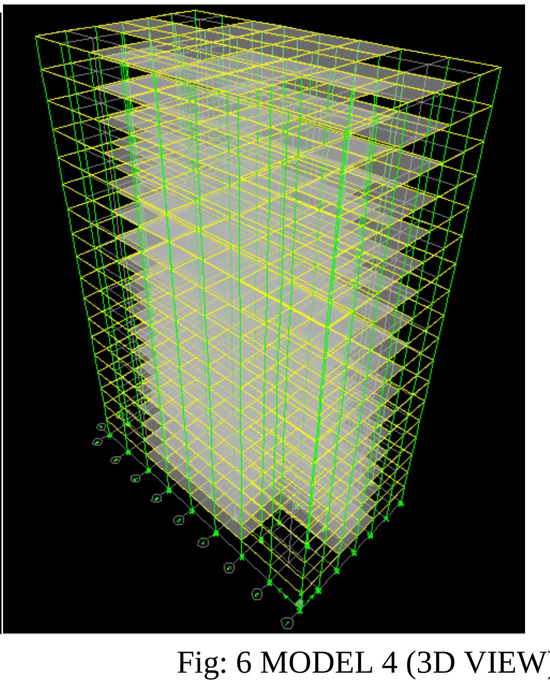


Fig: 6 MODEL 4 (3D VIEW)

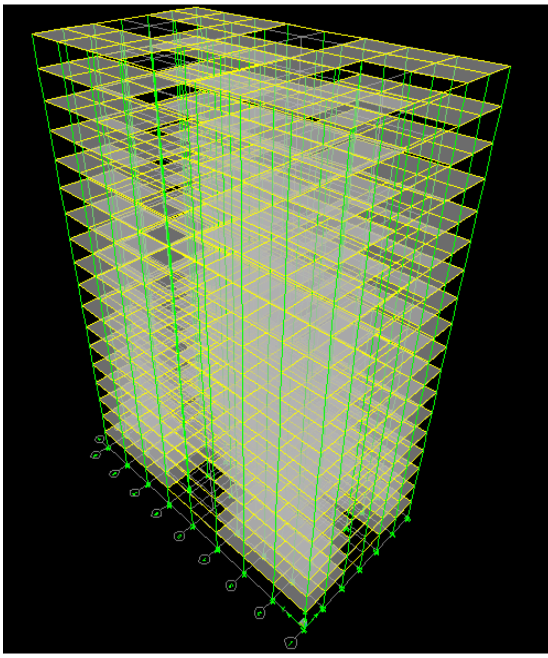


Fig: 7 MODEL 5 (3D VIEW)

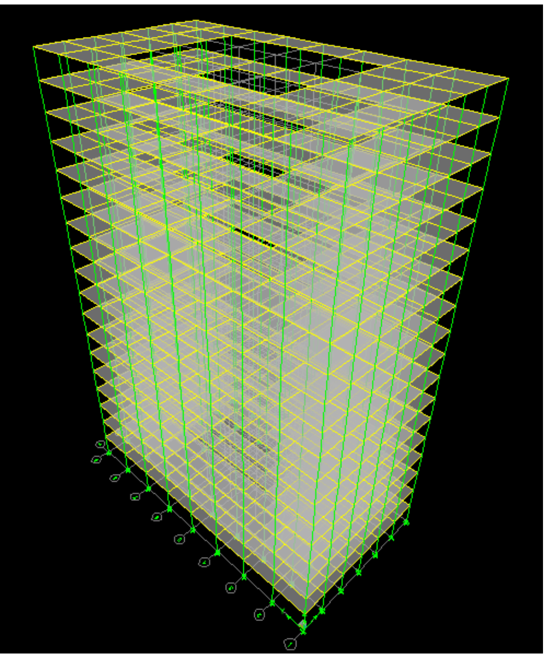


Fig: 8 MODEL 6 (3D VIEW)

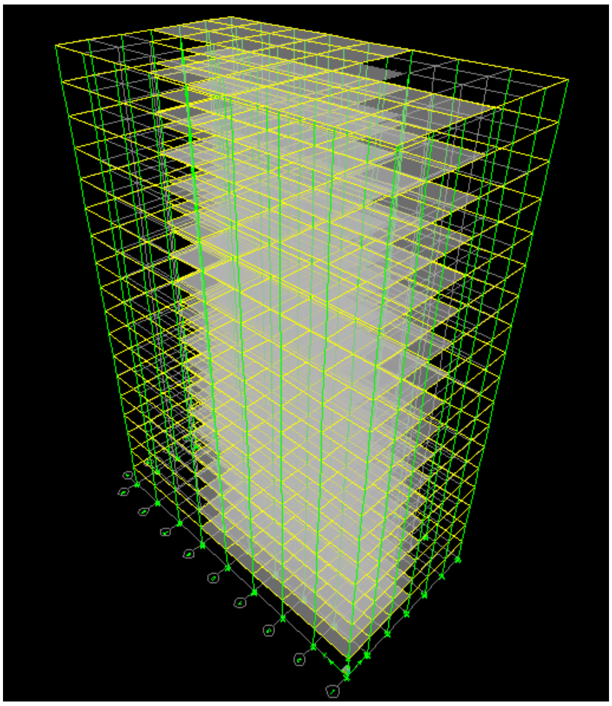


Fig: 9 MODEL 7 (3D VIEW)

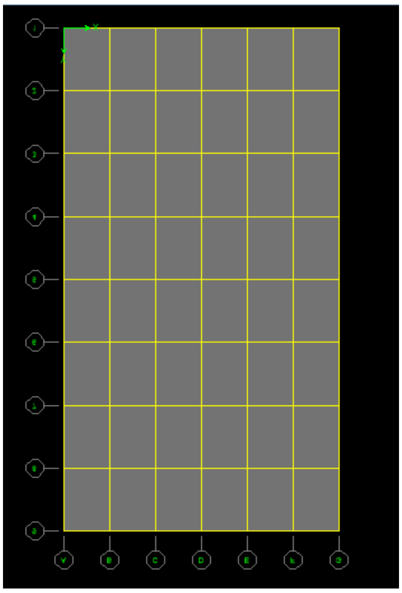
Wall of 230 mm thickness is provided also 12 mm plaster is assumed on both sides of the wall. Slab is taken as 150 x 150 mm in dimension.

Dead load is calculated on the basis of material unit weight, sizes of different elements in the model like slabs, columns, beams, brick masonry etc.

In Etabs software we designed different models to check whether these models are functional or not. Appropriate reinforcements were provided for the design purpose.

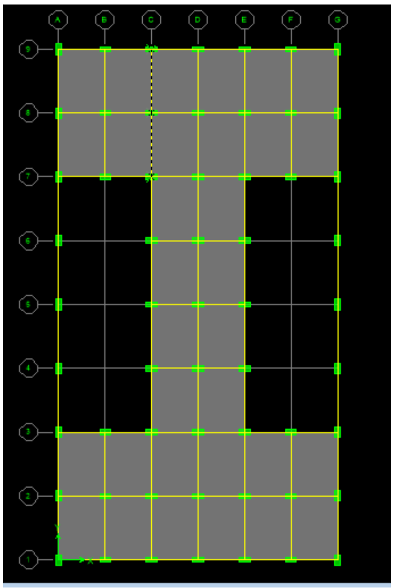
Due to plan irregularity some of the members fail in shear and torsion. This problem was solved by increasing the size of members at required places. Further the pushover analysis was performed in SAP 2000 where performance point and target displacements were obtained.

Plans of different model are shown below



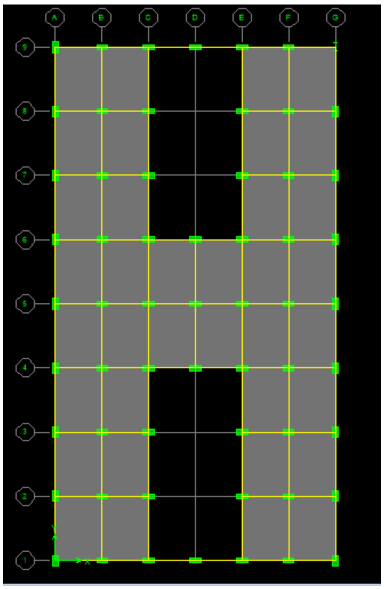
PLAN OF MODEL1

Fig: 10



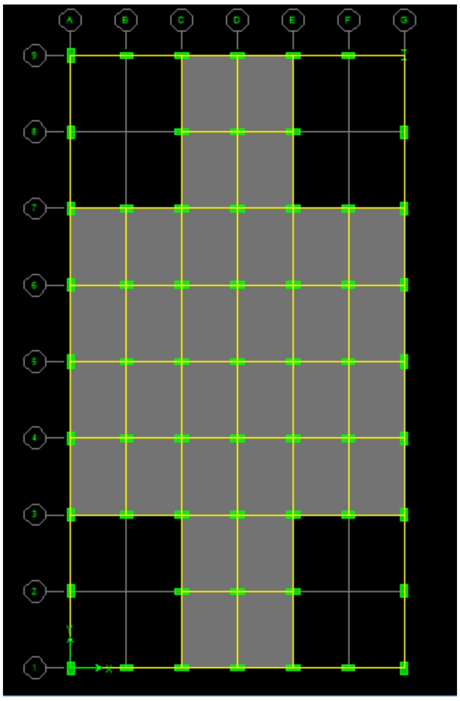
PLAN OF MODEL 2

Fig: 11



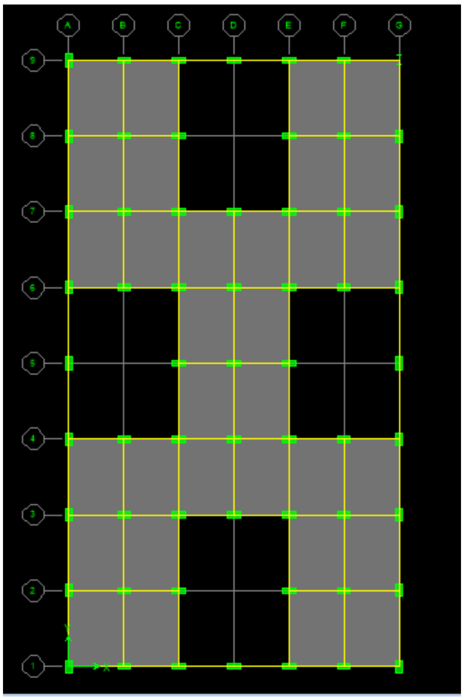
PLAN OF MODEL 3

Fig: 12



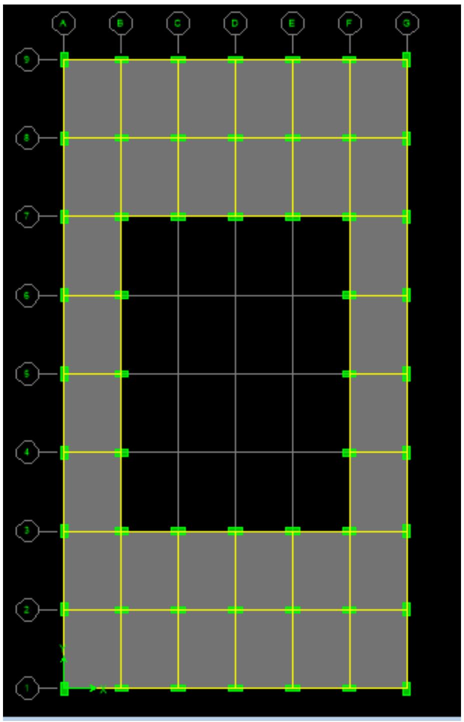
PLAN OF MODEL 4

Fig: 13



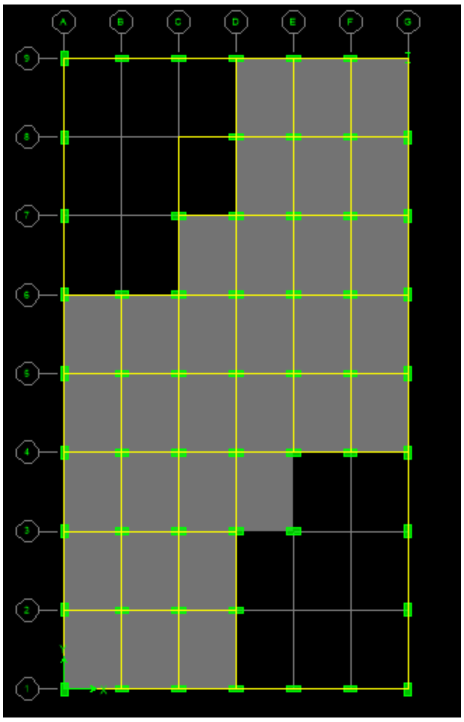
PLAN OF MODEL 5

Fig: 14



PLAN OF MODEL 6

Fig: 15



PLAN OF MODEL 7

Fig: 16

TABLE 1: DETAILS OF MODEL 1

Plan type	Bare frame 24x44 m
Storey height	3.6 m
Total height	64.8 m
Grade of concretes used	M25 for beams and slabs M35 and M40 for columns
Dimension of Beam	600x350 mm
Dimension of Columns	900x450 mm for M35 and M40 grade concrete
Thickness of slab	150 mm
Support conditions	Fixed
Number of beams	1980
Number of 900x450 mm size columns	882
Number of 900x500 mm size columns	252
Number of slabs	864

TABLE 2: DETAIL OF MODEL 2

Plan type	I Section type 24x44 m
Storey height	3.6 m
Total height	64.8 m
Grade of concretes used	M25 for beams and slabs M35 and M40for columns
Dimension of Beam	600x350 mm
Dimension of Columns	900x450 mm for M35 and M40 grade
Thickness of slab	150 mm
Support conditions	Fixed
Number of beams	1620
Number of 900x450 mm size columns	1026
Number of slabs	576

TABLE 3: DETAILS OF MODEL 3

Plan type	H- type 24x44 m
Storey height	3.6 m
Total height	64.8 m
Grade of concretes used	M25 for beams and slabs M35 and M40 for columns
Dimension of Beam	600x350 mm
Dimension of Columns	900x450 mm for M35 and M40 grade
Thickness of slab	150 mm
Support conditions	Fixed
Number of beams	1728
Number of 900x450 mm size columns(M35 grade)	912
Number of 900x450 mm size columns(M40 grade)	114
Number of slabs	756

TABLE 4: DETAILS OF MODEL 4

Plan type	+ Shaped 24x44 m
Storey height	3.6 m
Total height	64.8 m
Grade of concretes used	M25 for beams and slabs M35 and M40 for columns
Dimension of Beam	600x350 mm
Dimension of Columns	900x450 mm for M35 and M40 grade
Thickness of slab	150 mm
Support conditions	Fixed
Number of beams	1476
Number of 900x450 mm size columns(M35 grade)	770
Number of 900x450 mm size columns(M40 grade)	220
Number of slabs	432

TABLE 5: DETAILS OF MODEL 5

Plan type	24x44 m
Storey height	3.6 m
Total height	64.8 m
Grade of concretes used	M25 for beams and slabs M35 and M40 for columns
Dimension of Beam	600x350 mm
Dimension of Columns	900x450 mm for M35 and M40 grade
Thickness of slab	150 mm
Support conditions	Fixed
Number of beams	1692
Number of 900x450 mm size columns(M35 grade)	944
Number of 900x450 mm size columns(M40 grade)	118
Number of slabs	576

TABLE 6: DETAILS OF MODEL 6

Plan type	Hollow at core, 24x44 m
Storey height	3.6 m
Total height	64.8 m
Grade of concretes used	M25 for beams and slabs M35 and M40 for columns
Dimension of Beam	600x350 mm
Dimension of Columns	900x450 mm for M35 and M40 grade
Thickness of slab	150 mm
Support conditions	Fixed
Number of beams	1548
Number of 900x450 mm size columns(M35 grade)	864
Number of 900x450 mm size columns(M40 grade)	108
Number of slabs	576

TABLE 7: DETAILS OF MODEL 7

Plan type	24x44 m
Storey height	3.6 m
Total height	64.8 m
Grade of concretes used	M25 for beams and slabs M35 and M40 for columns
Dimension of Beam	600x350 mm
Dimension of Columns	900x450 mm for M35 and M40 grade
Thickness of slab	150 mm
Support conditions	Fixed
Number of beams	1620
Number of 900x450 mm size columns(M35 grade)	798
Number of 900x450 mm size columns(M40 grade)	228
Number of slabs	576

CHAPTER 4

PUSHOVER ANALYSIS

4.1 INTRODUCTION

The pushover analysis is nonlinear static method in which lateral loads are increased gradually. Total base shear v/s top displacement is obtained in any structure which indicate a premature failure or feebleness. All the beams and columns that reach yield or have tough crushing and even fracture are known. Total base shear v/s inter - story drift plot is additionally obtained. A pushover analysis is performed by subjecting a structure to a gradually increasing pattern of lateral hundreds that shows the mechanical phenomenon forces which might be tough by the structure once subjected to ground motion.

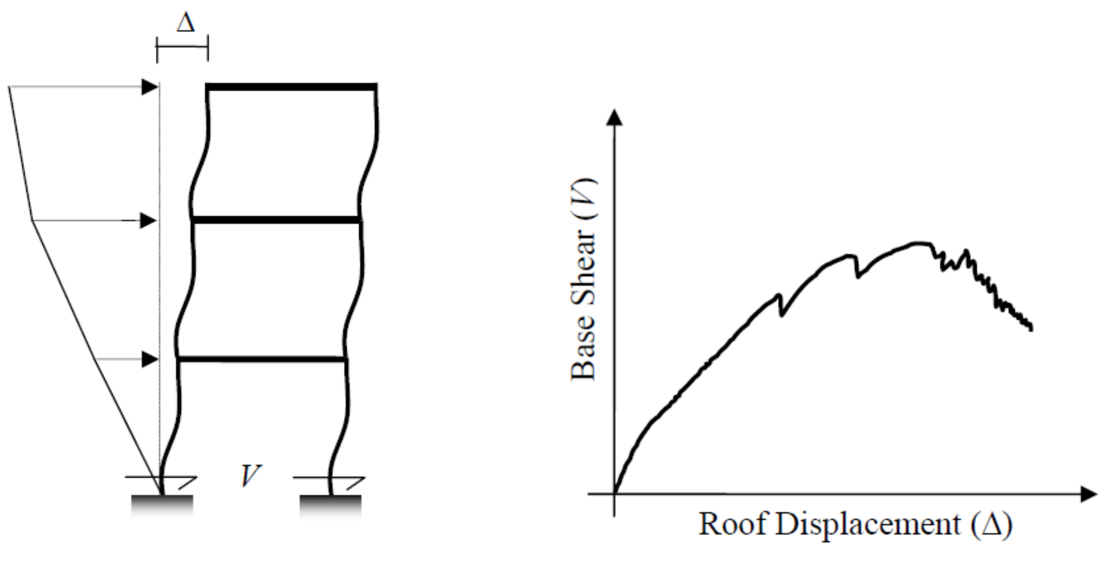
Underneath incrementally increasing loads several structural parts might yield consecutive. Therefore, at every event, the structure experiences a decrease in the stiffness. Using a nonlinear static pushover analysis, a representative non-linear force displacement relationship is obtained.

Nonlinear static pushover analysis, has been advanced over the past twenty years and has currently become the foremost most well-liked analysis technique for style and seismic performance functions as this system is relatively straightforward and considers post elastic performance. However, this system includes sure approximations and simplifications thanks to that some extent of variation is usually probable to exist within the seismic demand prediction of nonlinear static pushover analysis

Though, pushover analysis is used to capture very important structural response characteristics once the structure is under earthquake excitation, but the responsibility and also the accuracy of pushover analysis in estimating international and native seismic demands for all of the structures are a subject of dialogue and increased in pushover procedures are recommended to beat sure limitations of traditional nonlinear static pushover techniques. However, the improved techniques are largely computationally exhausting and in theory advanced thus use of such techniques are impractical in engineering professions and codes. As traditional nonlinear static pushover analysis is employed wide for the planning and seismic performance estimation functions, thus its weaknesses, limitations and predictions accuracy in routine application should be known by finding out all the factors that the pushover prediction. That is, the pertinence of pushover analysis for predicting seismic demands should be investigated for low-rise , mid-rise and high-rise structures by recognizing sure problems like modelling nonlinear member performance, process theme of the technique, effectiveness of invariant lateral load patterns in demonstrating higher mode effects, variations within the estimations of various lateral load patterns employed in traditional nonlinear static pushover analysis and precise estimation of target displacement wherever seismic demand prediction of pushover technique is accomplished.

4.2 Pushover Analysis Procedure

Pushover analysis is a static nonlinear procedure in which the magnitude of the lateral load is increased monotonically maintaining a predefined distribution pattern along the height of the building (Fig. 17a). Building is displaced till the 'control node' reaches 'target displacement' or building collapses. The sequence of cracking, plastic hinging and failure of the structural components throughout the procedure is observed. The relation between base shear and control node displacement is plotted for all the pushover analysis (Fig. 17b). Generation of base shear – control node displacement curve is single most important part of pushover analysis. This curve is conventionally called as pushover curve or capacity curve. The capacity curve is the basis of 'target displacement' estimation as explained in Section 4.4. So the pushover analysis may be carried out twice: (a) first time till the collapse of the building to estimate target displacement and (b) next time till the target displacement to estimate the seismic demand. The seismic demands for the selected earthquake (storey drifts, storey forces, and component deformation and forces) are calculated at the target displacement level. The seismic demand is then compared with the corresponding structural capacity or predefined performance limit state to know what performance the structure will exhibit. Independent analysis along each of the two orthogonal principal axes of the building is permitted unless concurrent evaluation of bidirectional effects is required.



a) Building model b) Pushover curve

Fig17: Schematic representation of pushover analysis procedure

The analysis results are sensitive to the selection of the control node and selection of lateral load pattern. In general, the centre of mass location at the roof of the building is considered as control node. For selecting lateral load pattern in pushover analysis, a set of guidelines as per FEMA 356 is explained in Section 4.3. The lateral load generally applied in both positive and negative directions in combination with gravity load (dead load and a portion of live load) to study the actual behaviour.

4.3 Lateral Load Profile

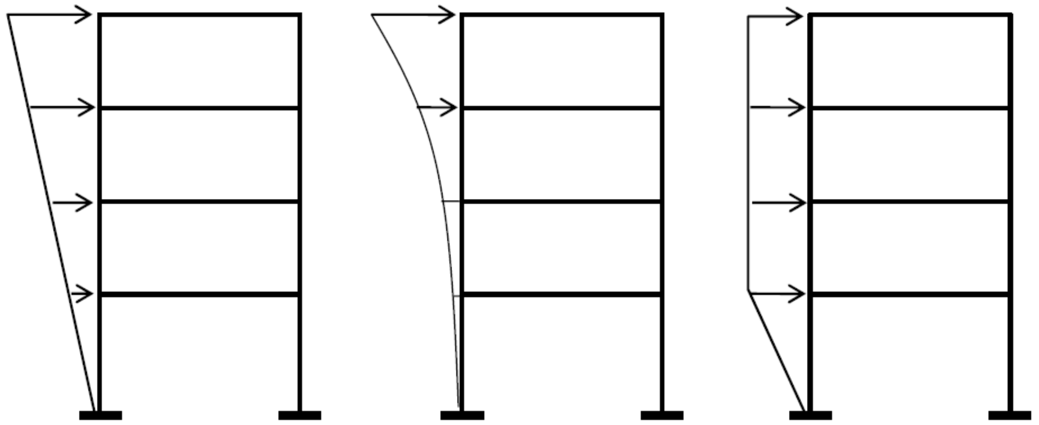
In pushover analysis the building is pushed with a specific load distribution pattern along the height of the building. The magnitude of the total force is increased but the pattern of the loading remains same till the end of the process. Pushover analysis results (*i.e.*, pushover curve, sequence of member yielding, building capacity and seismic demand) are very sensitive to the load pattern. The lateral load patterns should approximate the inertial forces expected in the building during an earthquake. The distribution of lateral inertial forces determines relative Base Shear (V) Roof Displacement (Δ) magnitudes of shears, moments, and deformations within the structure. The distribution of these forces will vary continuously during earthquake response as the members yield and stiffness characteristics change. It also depends on the type and magnitude of earthquake ground motion. Although the inertia force distributions vary with the severity of the earthquake and with time, FEMA 356 recommends primarily invariant load pattern for pushover analysis of framed buildings.
Several investigations (Mwafy and Elnashai, 2000; Gupta and Kunnath, 2000) have found that a triangular or trapezoidal shape of lateral load provide a better fit to dynamic analysis results at the elastic range but at large deformations the dynamic envelopes are closer to the uniformly distributed force pattern. Since the constant distribution methods are incapable of capturing such variations in characteristics of the structural behaviour under earthquake loading,
FEMA 356 suggests the use of at least two different patterns for all pushover analysis. Use of two lateral load patterns is intended to bind the range that may occur during actual dynamic response. FEMA 356 recommends selecting one load pattern from each of the following two groups:

1. Group – I:

i) Code-based vertical distribution of lateral forces used in equivalent static analysis (permitted only when more than 75% of the total mass participates in the fundamental mode in the direction under consideration).
ii) A vertical distribution proportional to the shape of the fundamental mode in the direction under consideration (permitted only when more than 75% of the total mass participates in this mode).
iii) A vertical distribution proportional to the story shear distribution calculated by combining modal responses from a response spectrum analysis of the building (sufficient number of modes to capture at least 90% of the total building mass required to be considered). This distribution shall be used when the period of the fundamental mode exceeds 1.0 second.

2. Group – II:

i) A uniform distribution consisting of lateral forces at each level proportional to the total mass at each level.
ii) An adaptive load distribution that changes as the structure is displaced. The adaptive load distribution shall be modified from the original load distribution using a procedure that considers the properties of the yielded structure.
Instead of using the uniform distribution to bind the solution, FEMA 356 also allows adaptive lateral load patterns to be used but it does not elaborate the procedure. Although adaptive procedure may yield results that are more consistent with the characteristics of the building under consideration it requires considerably more analysis effort. Fig18 shows the common lateral load pattern used in pushover analysis.



(a) Triangular (b) IS Code Based (c) Uniform

Fig18: Lateral load pattern for pushover analysis as per FEMA 356 (considering uniform mass distribution)

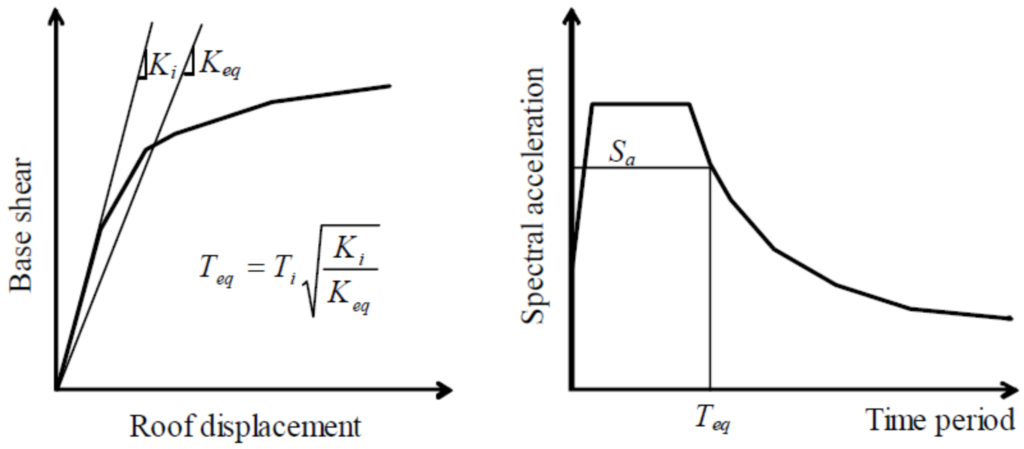
4.4 Target Displacement

Target displacement is the displacement demand for the building at the control node subjected to the ground motion under consideration. This is a very important parameter in pushover analysis because the global and component responses (forces and displacement) of the building at the target displacement are compared with the desired performance limit state to know the building performance. So the success of a pushover analysis largely depends on the accuracy of target displacement. There are two approaches to calculate target displacement:
(a) Displacement Coefficient Method (DCM) of FEMA 356 and
(b) Capacity Spectrum Method (CSM) of ATC 40.
Both of these approaches use pushover curve to calculate global displacement demand on the building from the response of an equivalent single-degree-of-freedom (SDOF) system. The only difference in these two methods is the technique used.

A. Displacement Coefficient Method (FEMA 356)

This method primarily estimates the elastic displacement of an equivalent SDOF system assuming initial linear properties and damping for the ground motion excitation under consideration. Then it estimates the total maximum inelastic displacement response for the building at roof by multiplying with a set of displacement coefficients.
The process begins with the base shear versus roof displacement curve (pushover curve) as shown in Fig19a. An equivalent period (T_{eq}) is generated from initial period (T_i) by graphical procedure. This equivalent period represents the linear stiffness of the equivalent SDOF system.
The peak elastic spectral displacement corresponding to this period is calculated directly from the response spectrum representing the seismic ground motion under consideration (Fig19b).

$$S_d = \frac{T_{eq}^2}{4\pi^2} S_a$$



(a) Pushover Curve (b) Elastic Response Spectrum

Fig19: Schematic representation of Displacement Coefficient Method (FEMA 356)

Now, the expected maximum roof displacement of the building (target displacement) under the selected seismic ground motion can be expressed as:

$$\delta_t = C_0 C_1 C_2 C_3 S_d = C_0 C_1 C_2 C_3 \frac{T_{eq}^2}{4\pi^2} S_a$$

C_0 = a shape factor (often taken as the first mode participation factor) to convert the spectral displacement of equivalent SDOF system to the displacement at the roof of the building.
C_1 = the ratio of expected displacement (elastic plus inelastic) for an inelastic system to the displacement of a linear system.
C_2 = a factor that accounts for the effect of pinching in load deformation relationship due to strength and stiffness degradation
C_3 = a factor to adjust geometric nonlinearity (P-Δ) effects

These coefficients are derived empirically from statistical studies of the nonlinear response history analyses of SDOF systems of varying periods and strengths and given in FEMA 356.
From the above definitions of the coefficients, it is clear that the change in building geometry will affect C_0 significantly whereas it is likely to have very little influence on the other factors. As per FEMA 356, the values of C_0 factor for shear buildings depend only on the number of storeys and the lateral load pattern used in the pushover analysis. Table 8 presents the values of C_0 provided by the FEMA 356 for shear buildings. In practice, Setback buildings have 5 or more storeys and the C_0 factor, as per FEMA 356, is constant for buildings with 5 or more storeys (Table 8).

Table 8: Values of C_0 factor for shear building as per FEMA 356

Number of storeys	Triangular Load Pattern	Uniform Load Pattern
1	1.0	1.00
2	1.2	1.15
3	1.2	1.20
5	1.3	1.20
10+	1.3	1.20

B. Capacity Spectrum Method (ATC 40)

The basic assumption in Capacity Spectrum Method is also the same as the previous one. That is, the maximum inelastic deformation of a nonlinear SDOF system can be approximated from the maximum deformation of a linear elastic SDOF system with an equivalent period and damping.
This procedure uses the estimates of ductility to calculate effective period and damping. This procedure uses the pushover curve in an acceleration-displacement response spectrum (ADRS) format. This can be obtained through simple conversion using the dynamic properties of the system. The pushover curve in an ADRS format is termed a ‘capacity spectrum’ for the structure.
The seismic ground motion is represented by a response spectrum in the same ADRS format and it is termed as demand spectrum (Fig. 2.4).

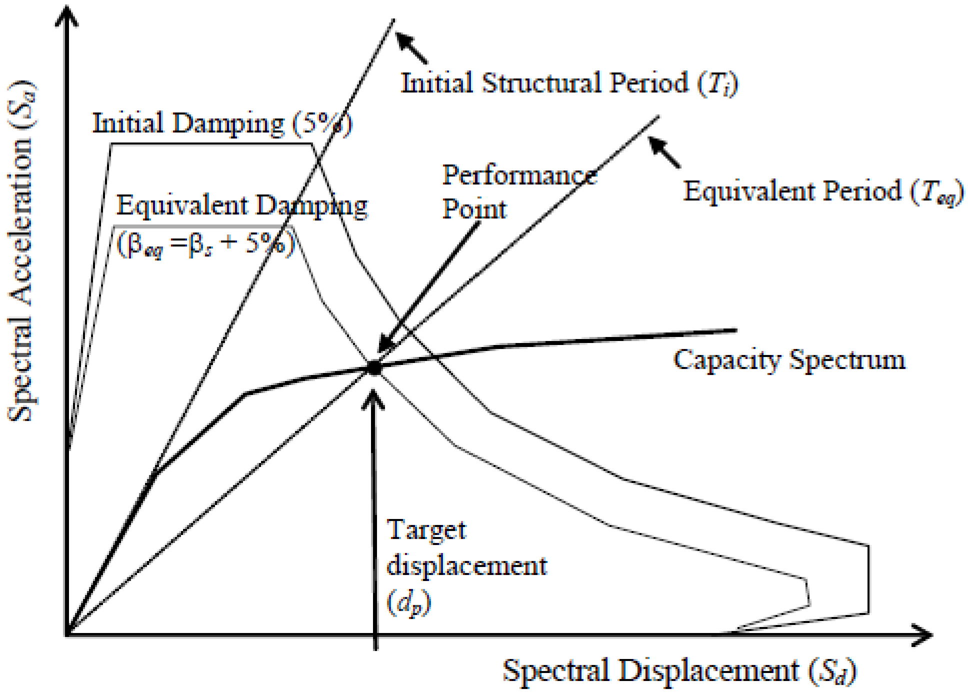


Fig20: Schematic representation of Capacity Spectrum Method (ATC 40)

The equivalent period (T_{eq}) is computed from the initial period of vibration (T_i) of the nonlinear system and displacement ductility ratio (μ). Similarly, the equivalent damping ratio (β_{eq}) is computed from initial damping ratio (ATC 40 suggests an initial elastic viscous damping ratio of 0.05 for reinforced concrete building) and the displacement ductility ratio (μ). ATC 40 provides the following equations to calculate equivalent time period (T_{eq}) and equivalent damping (β_{eq}).

$$T_{eq} = T_i \sqrt{\frac{\mu}{1 + \alpha\mu - \alpha}}$$

$$\beta_{eq} = \beta_i + \kappa \frac{2}{\pi} \frac{(\mu - 1)(1 - \alpha)}{\mu(1 + \alpha\mu - \alpha)} = 0.05 + \kappa \frac{2}{\pi} \frac{(\mu - 1)(1 - \alpha)}{\mu(1 + \alpha\mu - \alpha)}$$

Where α is the post-yield stiffness ratio and κ is an adjustment factor to approximately account for changes in hysteretic behaviour in reinforced concrete structures.
ATC 40 relates effective damping to the hysteresis curve (Fig. 2.5) and proposes three hysteretic behaviour types that alter the equivalent damping level. Type A hysteretic behaviour is meant for new structures with reasonably full hysteretic loops, and the corresponding equivalent damping ratios take the maximum values. Type C hysteretic behaviour represents severely degraded hysteretic loops, resulting in the smallest equivalent damping ratios. Type B hysteretic behaviour is an intermediate hysteretic behaviour between types A and C. The value of κ decreases for degrading systems (hysteretic behaviour types B and C).

4.5 SHORTCOMINGS OF THE PUSHOVER ANALYSIS

Pushover analysis is a very effective alternative to nonlinear dynamic analysis, but it is an approximate method. Major approximations lie in the choice of the lateral load pattern and in the calculation of target displacement. FEMA 356 guideline for load pattern does not cover all possible cases. It is applicable only to those cases where the fundamental mode participation is predominant. Both the methods to calculate target displacement (given in FEMA 356 and ATC 40) do not consider the higher mode participation. Also, it has been assumed that the response of a MDOF system is directly proportional to that of a SDOF system. This approximation is likely to yield adequate predictions of the element deformation demands for low to medium-rise buildings, where the behaviour is dominated by a single mode. However, pushover analysis can be grossly inaccurate for buildings with irregularity, where the contributions from higher modes are significant. Many publications (Aschheim, *et. al*., 1998; Chopra and Chintanapakdee, 2001; Chopra and Goel, 1999; Chopra and Goel, 2000; Chopra, *et. al*., 2003; Dinh and Ichinose, 2005; Fajfar, 2000; Goel and Chopra, 2004; Gupta and Krawinkler, 2000; Kalkan and Kunnath, 2007; Moghadam and Hajirasouliha, 2006; Mwafy and Elnashai, 2000; Mwafy and Elnashai, 2001; Krawinkler and Seneviratna, 1998) have demonstrated that traditional pushover analysis can be an extremely useful tool, if used with caution and acute engineering judgment, but it also exhibits significant shortcomings and limitations, which are summarized below:

a) One important assumption behind pushover analysis is that the response of a MDOF structure is directly related to an equivalent SDOF system. Although in several cases the response is dominated by the fundamental mode, this cannot be generalised. Moreover, the shape of the fundamental mode itself may vary significantly in nonlinear structures depending on the level of inelasticity and the location of damages.

b) Target displacement estimated from pushover analysis may be inaccurate for structures where higher mode effects are significant. The method, as prescribed in FEMA 356, ignores the contribution of the higher modes to the total response.

c) It is difficult to model three-dimensional and torsional effects. Pushover analysis is very well established and has been extensively used with 2-D models. However, little work has been carried out for problems that apply specifically to asymmetric 3-D systems, with stiffness or mass irregularities. It is not clear how to derive the load distributions and how to calculate the target displacement for the different frames of an asymmetric building. Moreover, there is no consensus regarding the application of the lateral force in one or both horizontal directions for such buildings.

d) The progressive stiffness degradation that occurs during the cyclic nonlinear earthquake loading of the structure is not considered in the present procedure. This degradation leads to changes in the periods and the modal characteristics of the structure that affect the loading attracted during earthquake ground motion.

e) Only horizontal earthquake load is considered in the current procedure. The vertical component of the earthquake loading is ignored; this can be of importance in some cases. There is no clear idea on how to combine pushover analysis with actions at every nonlinear step that account for the vertical ground motion.

f) Structural capacity and seismic demand are considered independent in the current method. This is incorrect, as the inelastic structural response is load-path dependent and the structural capacity is always associated with the seismic demand.

4.6 Limitations

Although pushover analysis has sure blessings as compared to elastic analysis techniques, underlying numerous assumptions, the accuracy of pushover predictions and also the restrictions of current pushover procedures should be recognized. The estimation of target displacement, choice of the lateral load patterns and identification of failure mechanism thanks to higher modes of vibration are very important problems that have an impact on the accuracy of pushover result. Target displacement is a global displacement probably during severe earthquake.

In pushover analysis, target displacement for a multi degree of freedom system is mostly calculable kind of like the displacement demand for corresponding equivalent single degree of freedom system. The elemental properties of identical SDOF system are gotten from a shape vector that represents the deflected form of MDOF system.
Most researchers advocate mistreatment the normalized displacement profile at target displacement level as that represents the deflected shape of MDOF system. Most researchers recommend using the normalized displacement profile at target displacement level as a shape vector, but since this displacement is not known beforehand, an iteration is needed. Therefore, by most of the approaches, a fixed shape vector, elastic first mode, is utilized for simplicity without regarding higher modes. The target displacement is found by the roof displacement at mass centre of the structure.
The correct estimation of the target displacement related to explicit performance objective, has an impact on accuracy of the unstable demand predictions of pushover analysis. Furthermore, hysteretic characteristics of MDOF should be incorporated into the equivalent SDOF model, just in case displacement demand is affected from stiffness degradation or pinching, strength deterioration, P-Δ effects. Foundation uplift, torsional effects further as semi-rigid diaphragms may additionally have an effect on target displacement.

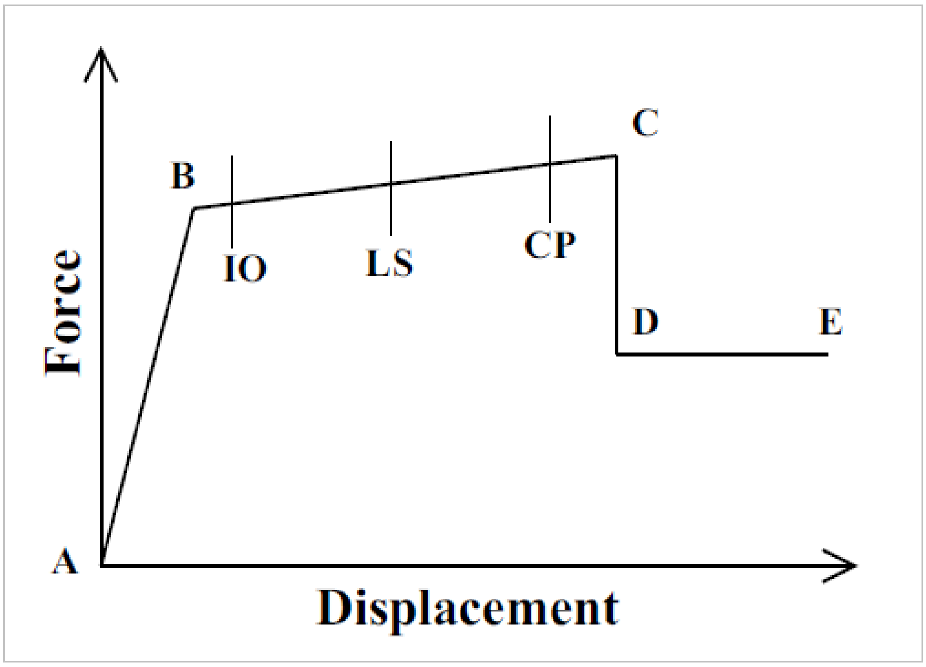


Fig21: Force Deformation in pushover hinge

However, in pushover analysis, typically associate invariant lateral load pattern is employed that the distribution of the inertia forces is assumed to be not dynamic throughout earthquake and unshapely configuration of the structure underneath the action of invariant lateral load pattern is probably going to be kind of like that that is tough within the style earthquake.
As response of the structure, thus the capability curve is extremely sensitive to the lateral load distribution elect selection of lateral load pattern is a lot of vital as compared to the correct estimation of the target displacement.
In order to get performance points additionally because the location of hinges in numerous stages, we will use the pushover curve. During this curve, the vary AB the elastic vary, B to IO the vary of instant occupancy, IO to LS the vary of life safety and LS to CP the vary of collapse interference.

When a hinge touches purpose C on its force-displacement curve then that hinge should begin to drop load. The style within which the load is discharged from a hinge that has reached purpose C is that the pushover force or the bottom shear is reduced until the force therein hinge is steady with the force at purpose D.

As the force is discharged, all the weather unload and additionally displacement is attenuated. When they yielded hinge touches the purpose D force level, the magnitude of pushover force is once more amplified and therefore the displacement starts to extend once more. If all the hinges are inside the given CP limit then the structure is meant to be safe. Though, the hinge when IO vary may additionally be needed to be retrofitted looking on the importance of the structure.

a) Immediate Occupancy – Achieves elastic behaviour by limiting structural injury (e.g. yielding of steel, vital cracking of concrete, and non-structural injury.)

b) Life Safety - Limit injury of structural and non-structural parts to attenuate the chance of injury or casualties and to stay essential circulation routes accessible.

c) Collapse interference – guarantee a tiny low risk of partial or complete building collapse by limiting structural deformations and forces to the onset of strength and stiffness degradation.

CHAPTER - 5

RESPONSE SPECTRUM ANALYSIS

5.1 INTRODUCTION

The basis of engineering geophysical science is that they have to be compelled to quantify however to a given structure which can answer complex ground movements. The structure's response is decided by its mass and stiffness distributions. For example, stiff buildings can expertise low accelerations relative to the bottom. Tall buildings tend to accelerate far from ground motions, leading to low absolute accelerations, wherever absolute acceleration is that the add of the building's movement relative to the bottom and therefore the ground acceleration. Inhomogeneity at intervals the building might cause twisting (de Sylva 2005).

Italian engineers began braving the problem of earthquake response in structural style once the 1908 Reggio Calabria earthquake, which, in conjunction with a tidal wave, claimed nearly a 100000 casualties. They used a static approach and Danusso, in 1909, prompt that buildings got to "follow docile the shaking action, not opposing it stiffly." He used acoustic theory to represent building response because the superposition of n pendulums. His work, however wasn't widespread (Trifunac and Todorovska, 2008).

The response spectrum methodology (RSM) was introduced in 1932 within the doctorial treatise of Maurice Anthony Biot at Caltech. it\'s AN approach to finding earthquake response of structures exploitation waves or vibrational mode shapes. The mathematical principles of oscillations in n-degree-of-freedom systems were taken for the most part from the theories of acoustics developed by Rayleigh. Biot declared "… [a] building…has a certain variety of therefore referred to as traditional modes of vibration, and to every of them corresponds a particular frequency." Biot used the Fourier amplitude spectrum to search out the utmost amplitude of motion of a system: The addition of amplitudes for every separate mode of oscillation (Trifunac and Todorovska, 2008).

The idea of the "response spectrum" was applied in style needs within the middle 20th century, for example in building codes within the state of CA (Hudson, 1956; Trifunac and Todorovska, 2008). It came into widespread use because the primary theoretical tool in earthquake engineering within the Nineteen Seventies once strong-motion accelerograph information became wide accessible (Trifunac and Todorovska, 2008).

Using a mathematical model of a building, as an instance with given lots, stiffness values, and dimensions for every level, earthquake acceleration records may be applied to gauge, however the given structure behaves (Clough, 1962). System response is described because the linear superposition of single degree-of-freedom systems for numerous mode shapes and corresponding natural frequencies (Trifunac and Todorovska, 2008).

5.2 Response Spectrum Analysis Procedure

There are a unit two main ways that to try to earthquake analysis of linear systems.

Particularly Response History Analysis (RHA) and Response spectrometry Analysis (RSA). RHA may be an additional labour intensive analysis as a result of which it provides a structural response r(t) as an operate of your time over the period of a shaking event. For a SDOF Response Spectrum Analysis can give an equivalent result, however that's not the case for a MDOF situation.

However it will give an honest estimate. For an N-story building with a concept trigonal concerning 2 axes, you'll reason the height response as follows from Chopra [3]:

1. Outline the structural properties

(a) confirm the mass matrix m and also the lateral stiffness matrix k

(b) Estimate the modal damping ratios ζ_n

2. Confirm the natural frequencies ω_n and natural modes φ_n of vibration

3. Reason the height response within the ordinal mode

(a) Resembling natural amount Volunteer State and damping magnitude relation ζ_n, scan D_n and A_n from the response or style spectrum

(b) Reason the ground displacement and story drifts with $u_{jn} = \Gamma_n \varphi j_n D_n$ and

$$\Delta_{jn} = \Gamma_n(\varphi_{jn} - \varphi_{j-1,n})D_n$$

(c) Reason equivalent static forces f_n from $f_{jn} = \Gamma_n m_j \varphi_{jn} A_n$

(d) Reason the story forces, shear and overturning moment, and component forces, bending moments and shear, by static analysis of the structure subjected to lateral forces fn.

4. Confirm an estimate for the height worth r of any response amount by combining the peak modal values r_n in line with SRSS or CQC reckoning on the spacing of the modal frequencies.
When exploitation this methodology it's vital to remember that it's wrong to reason the combined peak worth of a response amount from the combined peak values of alternative response quantities. The proper procedure is to mix the height modal values, so hard the combined peak of this.

CHAPTER – 6

RESULTS AND DISCUSSION

6.1 BASE SHEAR COMPARISON OF DIFFERENT MODELS

Table 9: Base shear variation of different models

MODEL	1	2	3	4	5	6	7
EQX(KN)	6909.68	5377.88	5795.15	4729.76	5506.28	5158.27	5304.66
SPECX(KN)	3984.362	2830.761	3211.661	2575.701	3177.128	2921.043	2953.597
PUSHX(KN)	8829.613	5923.742	8916.547	8373.73	6988.89	6719.05	7047.93

Base shear EQX depends upon the seismic weight of the structure. Since seismic weight of model 4 is least the base shear and spectral base shear are least among the others.
In case where diaphragm is continuous we observe maximum base shear.

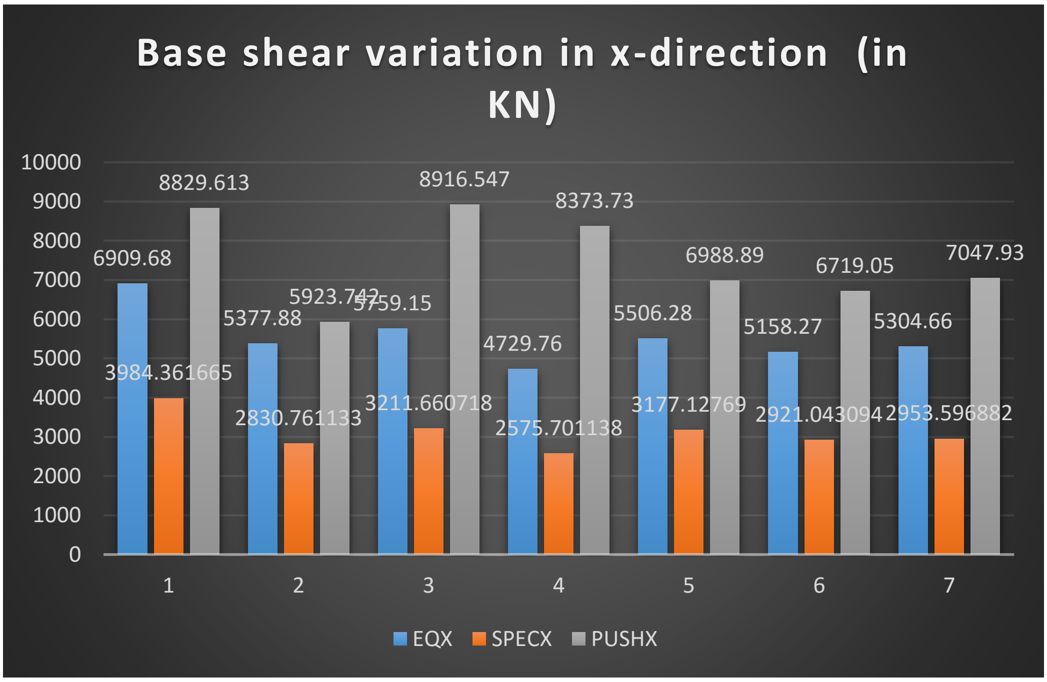


Fig22: Base shear variation in x-direction (in KN)

PUSHX base shear is larger than SPECX and EQX in every case this is because of the fact that pushover analysis is a nonlinear analysis. Displacements are also larger in case of push over analysis.

TABLE 10: BASE SHEAR COMPARISION IN Y-DIRECTION

MODEL	EQY(KN)	SPECX(KN)	PUSHY(KN)
1	6909.68	3984.362	11023.63
2	5377.88	2830.761	8330.66
3	5759.15	3211.661	10616.66
4	4729.76	2575.701	8588.78
5	5506.28	3177.128	7578.02
6	5158.27	2921.043	7655.14
7	5304.66	2953.597	7047.934

Base shear EQY depends upon the seismic weight of the structure. Since seismic weight of model 4 is least the base shear and spectral base shear are least among the others.
In case where diaphragm is continuous we observe maximum base shear.

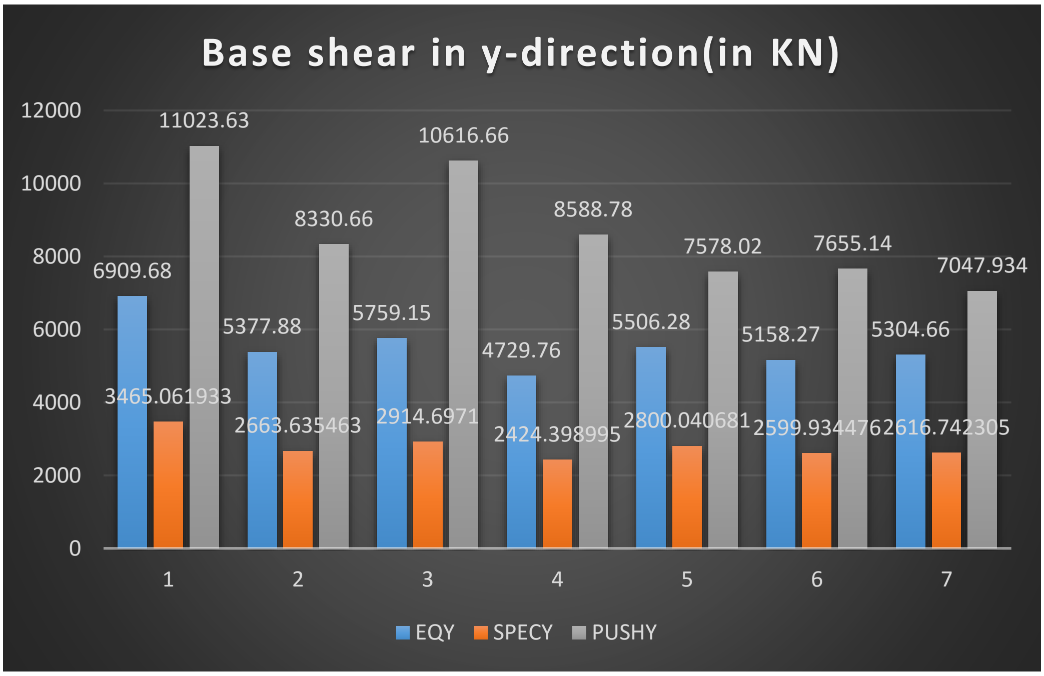


Fig23: Base shear in y-direction (in KN)

PUSHY base shear is larger than SPECY and EQY in every case this is because of the fact that the pushover analysis is a nonlinear analysis. Displacements are also larger in case of push over analysis. Here EQY is static shear, SPECY is dynamic shear and PUSHY is nonlinear static shear. It is clear from the above data that shear in case of dynamic analysis is least and shear in case of nonlinear static analysis is maximum.

6.2 TIME PERIOD AND FREQUENCY VARIATION FOR MODE 1

TABLE 11: Time period and frequency of models corresponding to mode1

MODEL	TIME PERIOD(Sec)	FREQUENCY(Hz)
1	3.072	0.325521
2	2.999917	0.333343
3	3.052063	0.327647
4	2.91	0.343643
5	3.036	0.329381
6	3.069574	0.325778
7	3.027077	0.330352

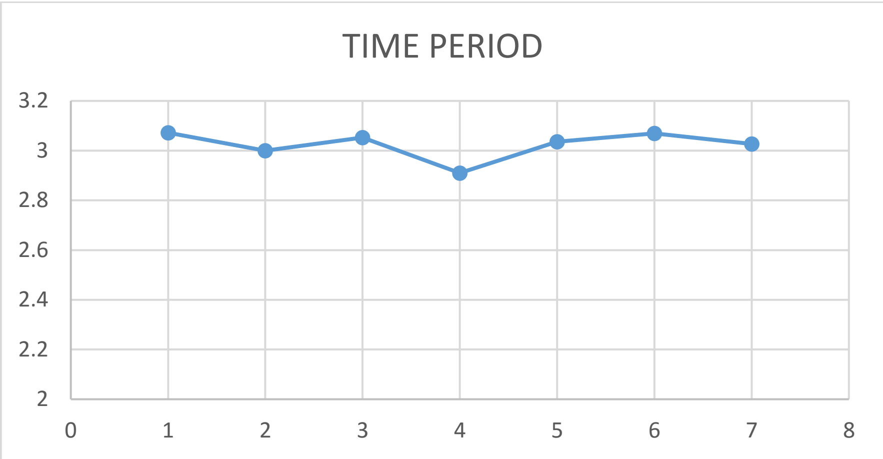


Fig24: Time period corresponding to 1st mode

Time period in case of model 4 is less which emphasizes that stiffness of this model is maximum as time period is inversely proportional to the stiffness of the structure.

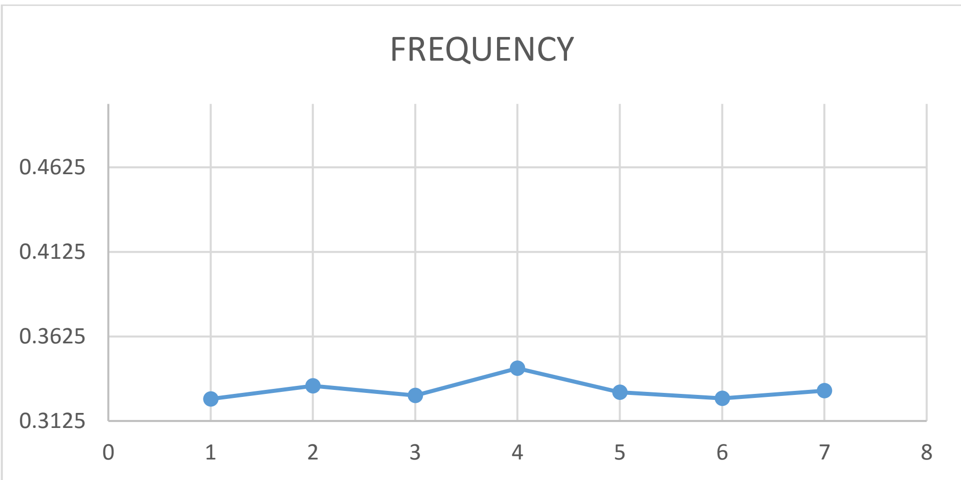


Fig25: Frequency variation corresponding to 1st mode

Frequency in case of model 4 is maximum which means that the stiffness of this structure is maximum as frequency is directly proportional to the stiffness of building.

6.3 WIND FORCE AT BASE

TABLE 12: Wind force variation

MODEL	WINDX(KN)	WINDY(KN)
1	4708	3001
2	4708	3001
3	4708	3001
4	4708	300
5	4708	3001
6	4708	3001
7	4708	3001

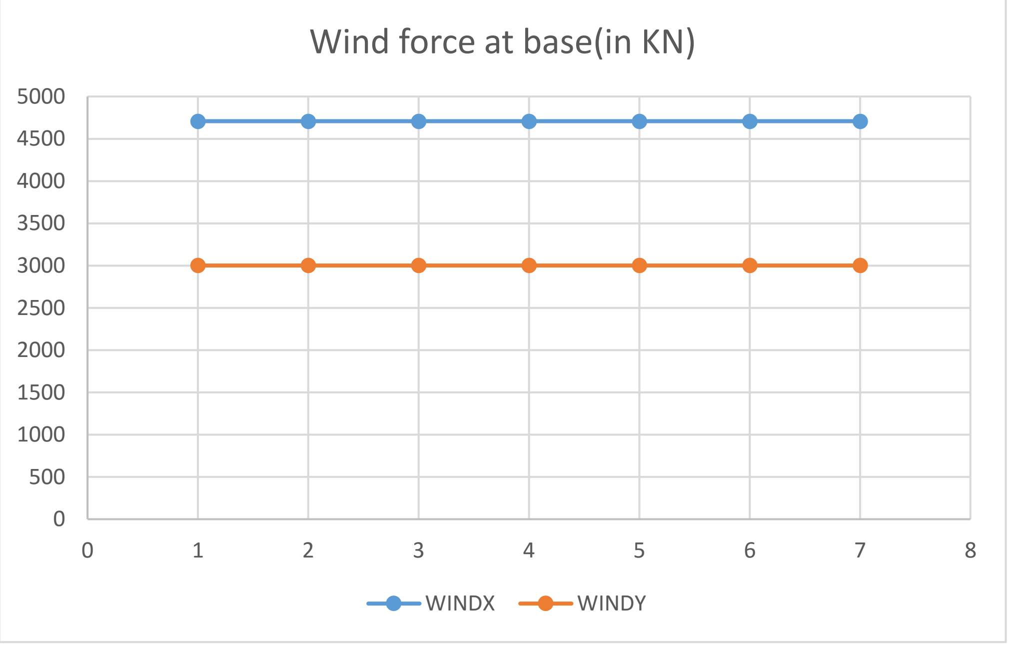


Fig26: Wind force variation at base of different models

Wind loading is constant as it acts to the structure uniformly in a certain zone and it also depends on height. Since the height of the structure is same in all cases and all the wind parameters are same, the wind forces at base are same in each models

Wind parameters taken as per IS 875:2015

i) Wind speed = 47 m/s
ii) Terrain category = 3
iii) Probability factor(K_1) = 1
iv) Topography factor(K_3) = 1
v) Importance factor(K_4) = 1

6.4 BASE SHEAR VARIATION IN X and Y DIRECTION

TABLE 13: Base shear variation in x - y direction

MODEL	EQX(KN)	EQY(KN)
1	6909.68	6909.68
2	5377.88	5377.88
3	5759.15	5759.15
4	4729.76	4729.76
5	5506.28	5506.28
6	5158.27	5158.27
7	5304.66	5304.66

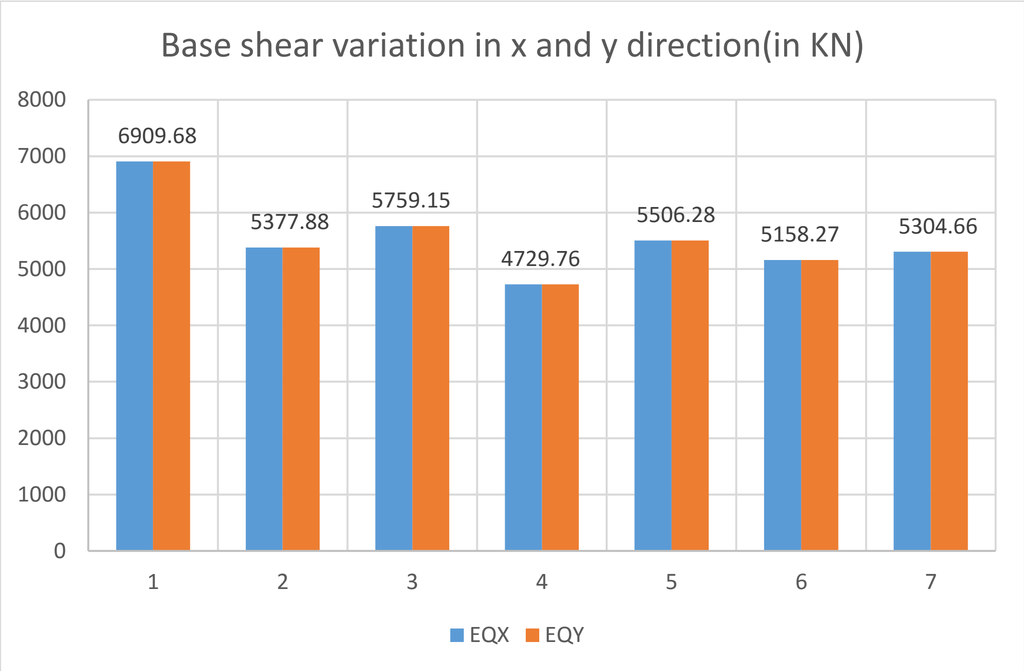


Fig27: Base shear variation in x and y direction (in KN)

EQX and EQY are same as seismic force does not depend on direction in case of symmetrical structure. Although our structure is not identical in plan but this is the case where we considered without infill. Time period calculation for without infill case depends upon the height of the building.

As per IS 1893:2002 the expression for time period in case of without infill is given as

$$T = 0.075\ H^{0.75}$$

Here height of the structure is 64.8, so after putting this value in above expression we get T= 1.71 seconds

This time period is used to calculate the $\frac{Sa}{g}$

Since time period is same for both directions, so $\frac{Sa}{g}$ value will be same which will result in similar horizontal acceleration coefficient. Hence the value of EQX and EQY will be same.

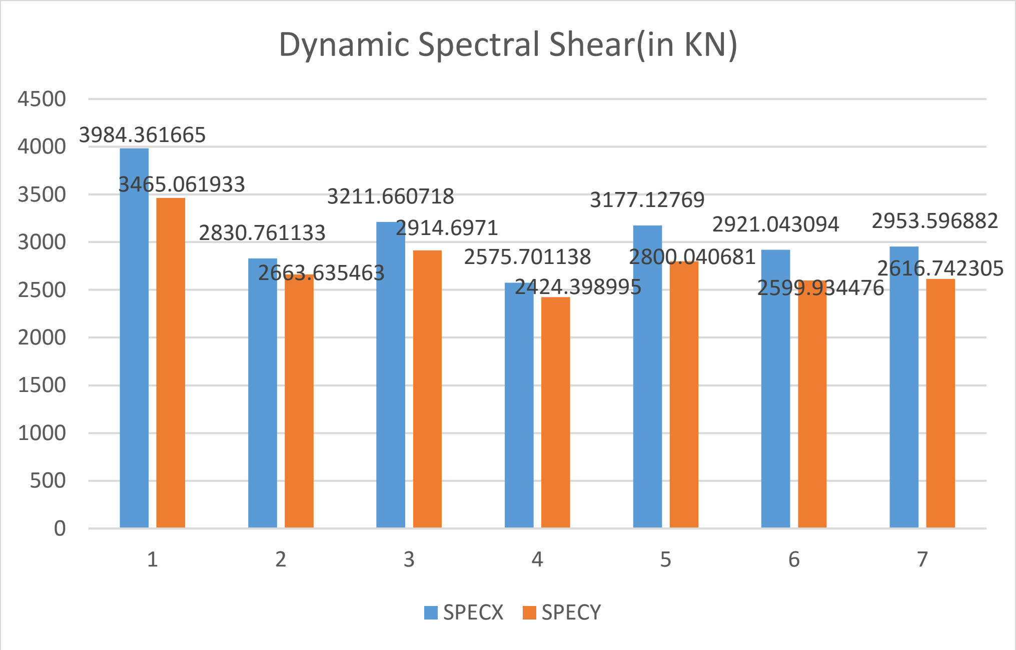


Fig28: spectral shear variation in x and y direction

Spectral shear is highest in case where diaphragm is continuous. Since spectral shear is dynamic shear it will always be less than static shear.

From above graph it is clear that spectral shear is minimum in case of model 4. As model 4 is less flexible so it cannot resist more dynamic loading than other models.

In case when diaphragm is continuous the capacity of resisting load is more and since there is no discontinuity structural stiffness is high.

6.5 MAXIMUM STOREY DISPLACEMENT OF TOP STOREY

TABLE 14: Maximum storey displacement of top storey of models

MODEL	MAXIMUM STOREY DISPLACEMENT	
	EQX(meters)	EQY(meters)
1	0.07	0.09
2	0.0852	0.081
3	0.0735	0.0834
4	0.0739	0.0763
5	0.068	0.083
6	0.07	0.085
7	0.0723	0.0842

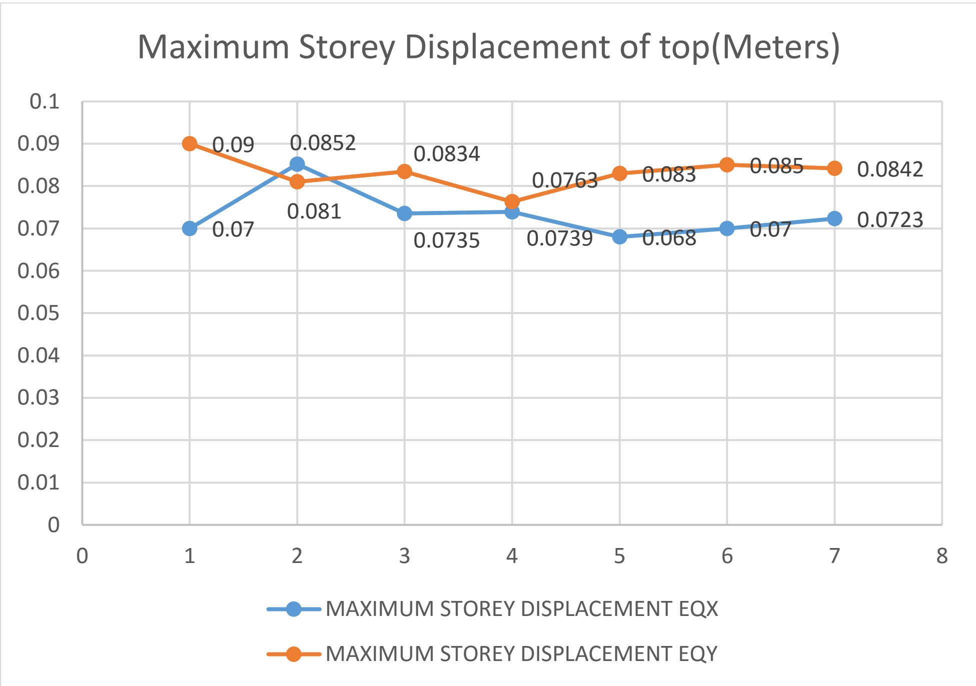


Fig29: Maximum storey displacement of top floor in meters

Maximum displacement of top storey is in model 2 and minimum in model 5 in case of EQX. Maximum top storey displacement in EQY direction is in model 1 and minimum in model 4.

6.6 OVERTURNING MOMENT AT BASE

TABLE 15: Overturning moment at the base of models

MODEL	OVERTURNING MOMENT AT BASE	
	EQX(KN-m)	EQY(KN-m)
1	341977.9	-341978
2	266487.3	-266487
3	286254.3	-285479
4	234265.7	-234266
5	272890.3	-272890
6	255669.3	-255669
7	262128.1	-255669

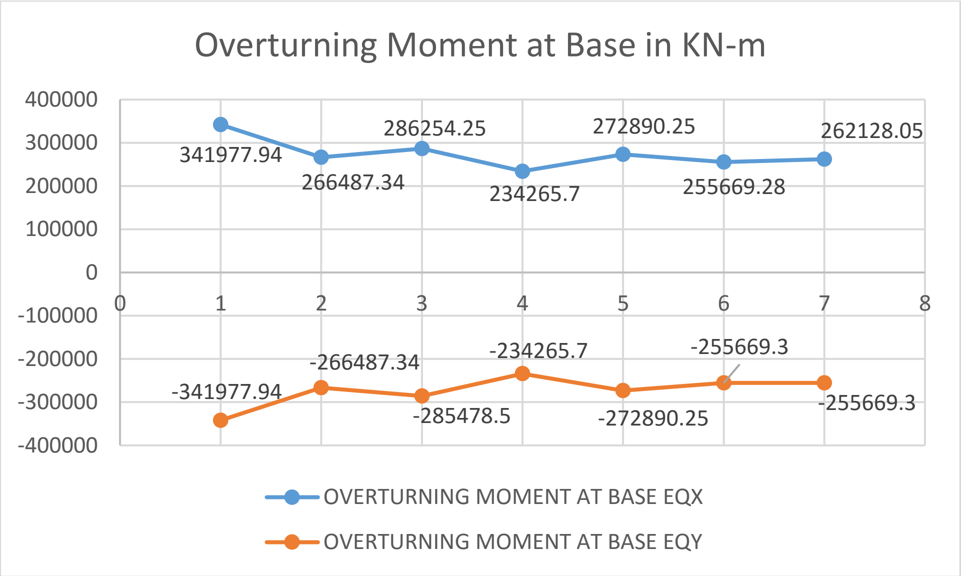


Fig30: Comparison of Overturning moments at base in KN-m

Structure without diaphragm discontinuity will experience maximum overturning moment at its base as the mass of structure is maximum in model1.
Also model 4 experience minimum overturning moment at its base as it has minimum weight.
Rest models overturning moment is shown in above graph.

6.7 MAXIMUM STOREY DRIFT

TABLE 16: Maximum storey drift of all models

MODEL	MAXIMUM STOREY DRIFT	
	EQX(Meters)	EQY(Meters)
1	0.0013	0.0017
2	0.00164	0.00164
3	0.0014	0.0017
4	0.00144	0.0016
5	0.00132	0.0017
6	0.00136	0.00171
7	0.00141	0.00175

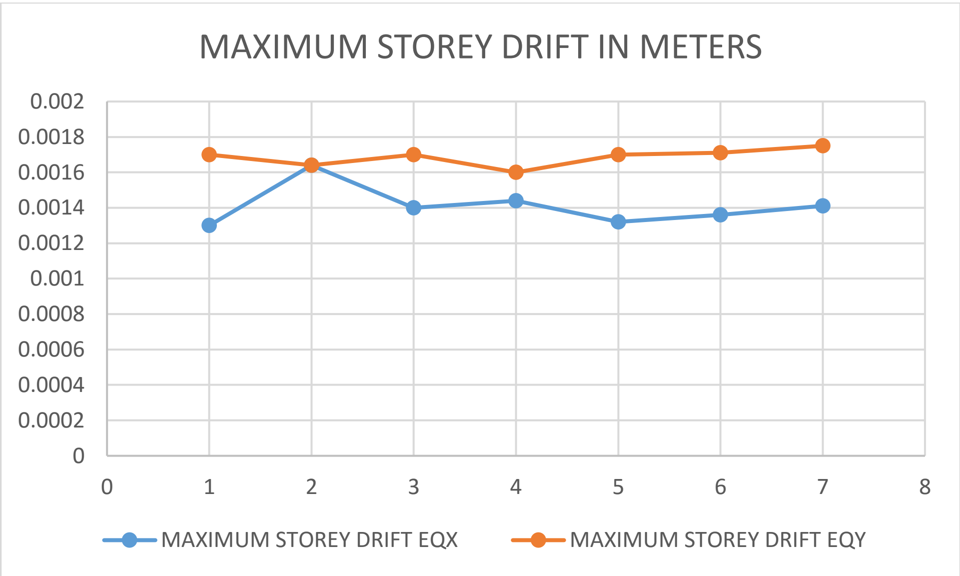


Fig31: Comparison of maximum storey drift in meters

Maximum storey drift is in model 2 and minimum in model 5 in case of EQX. Whereas in case of EQY storey drift is maximum in model 7 and minimum in model 4.

6.8 MODAL MASS PARTICIPATION OF MODELS

TABLE 17: Model mass participation ratio in percentage

MODEL	MODAL MASS PARTICIPATION RATIO	
	UX	UY
1	94.68	94.76
2	90.39	90.36
3	94.73	94.91
4	89.34	90.29
5	94.62	94.89
6	94.75	95.12
7	90.49	91.45

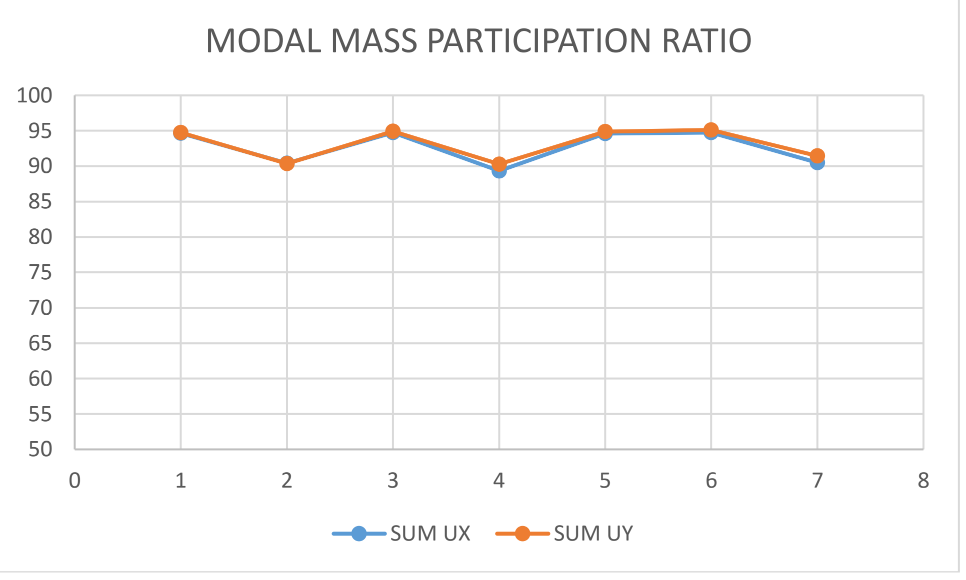


Fig32: Comparison of modal mass participation ratio

As per IS-1893:2002 number of modes used in the analysis should be such that the entire sum of modal masses of all modes considered is at least 90 percent of the total seismic mass. Here the minimum modal mass is 90 percent.

Modal Analysis results illustrates that there are some unusual modes when diaphragm discontinuity is modelled. However, the mass participation for those modes is found to be negligible. Therefore, these modes will not change the response of the structure significantly.

6.9 PUSHOVER ANALYSIS RESULTS

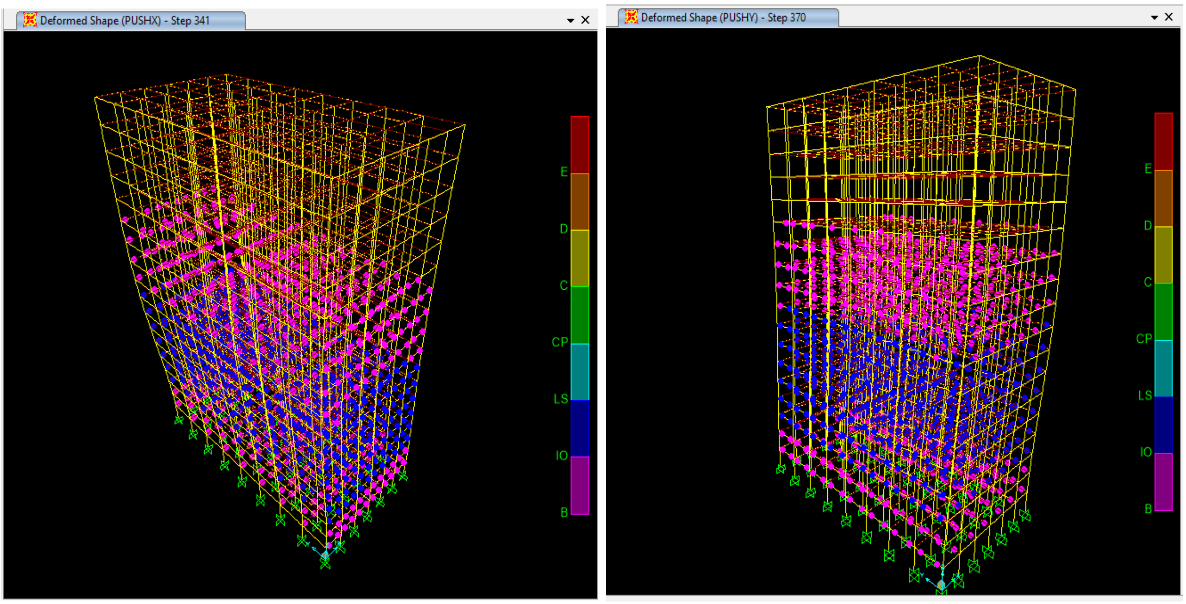


Fig33: Hinges formed at performance point for PUSHX and PUSHY in MODEL 1

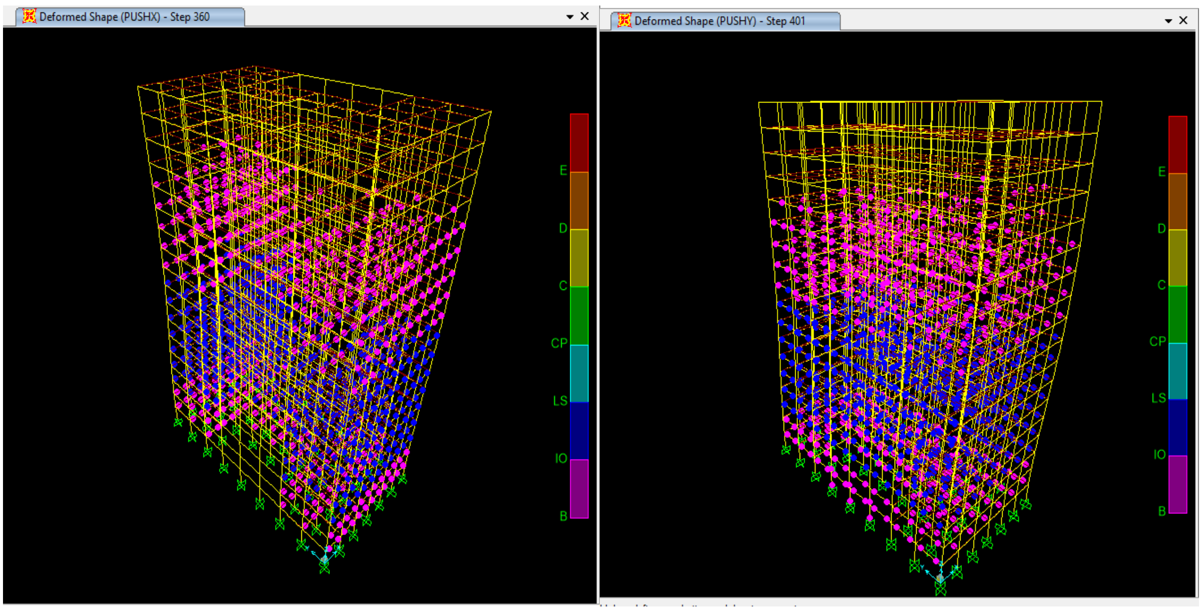


Fig34: Hinges formed at performance point for PUSHX and PUSHY in MODEL 2

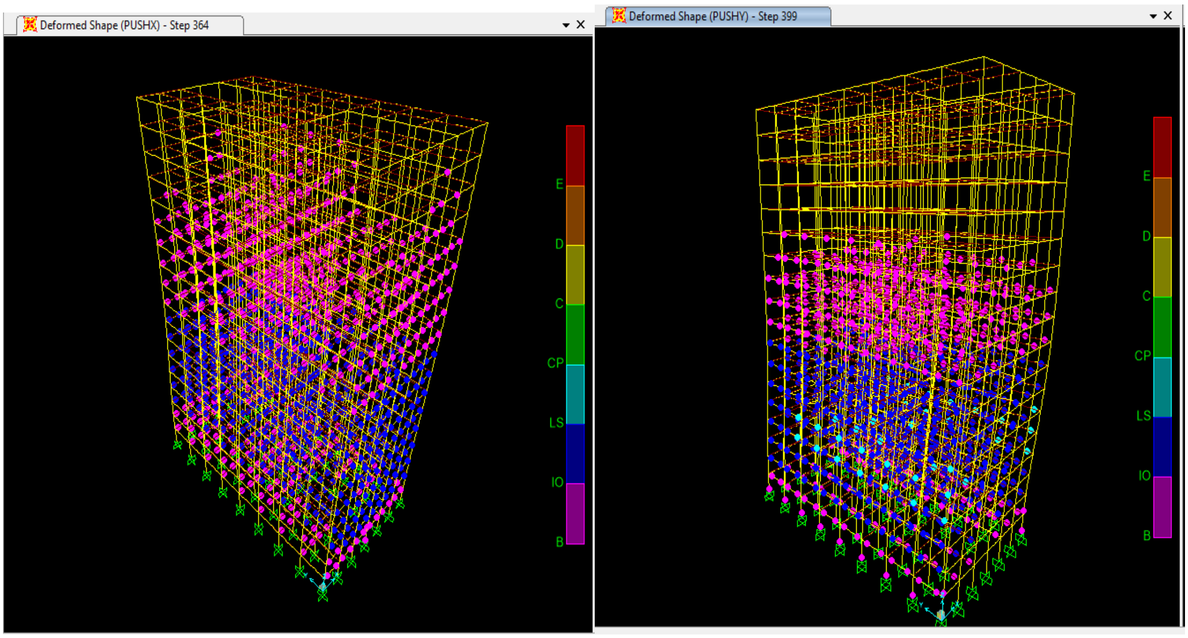


Fig35: Hinges formed at performance point for PUSHX and PUSHY in MODEL 3

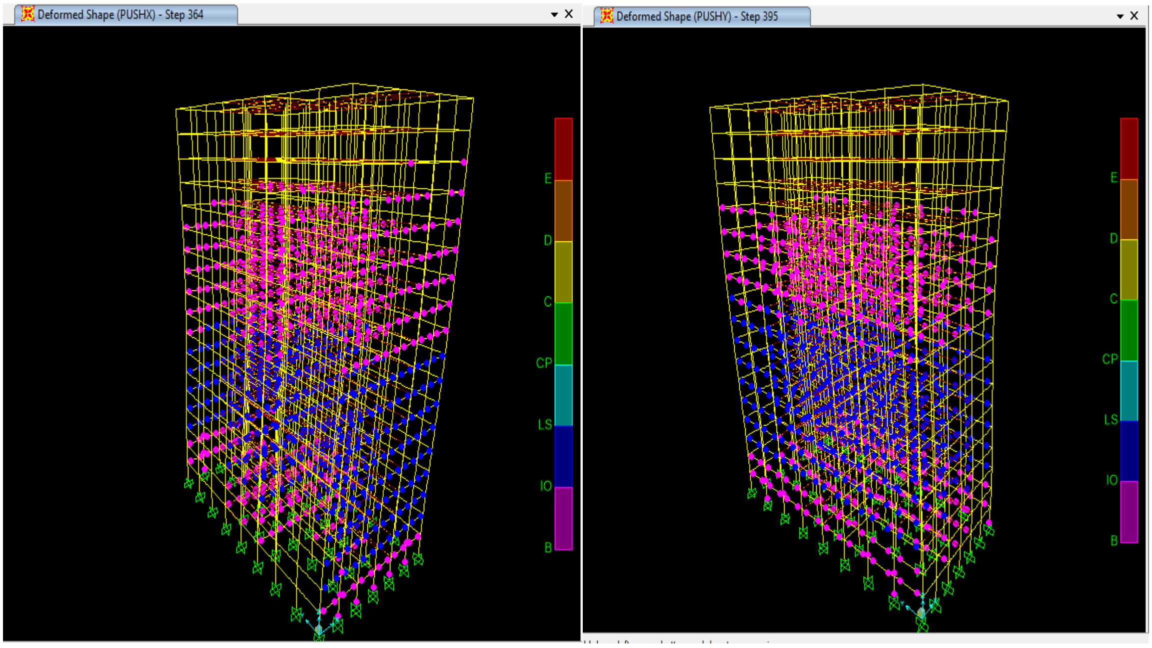


Fig36: Hinges formed at performance point for PUSHX and PUSHY in MODEL 4

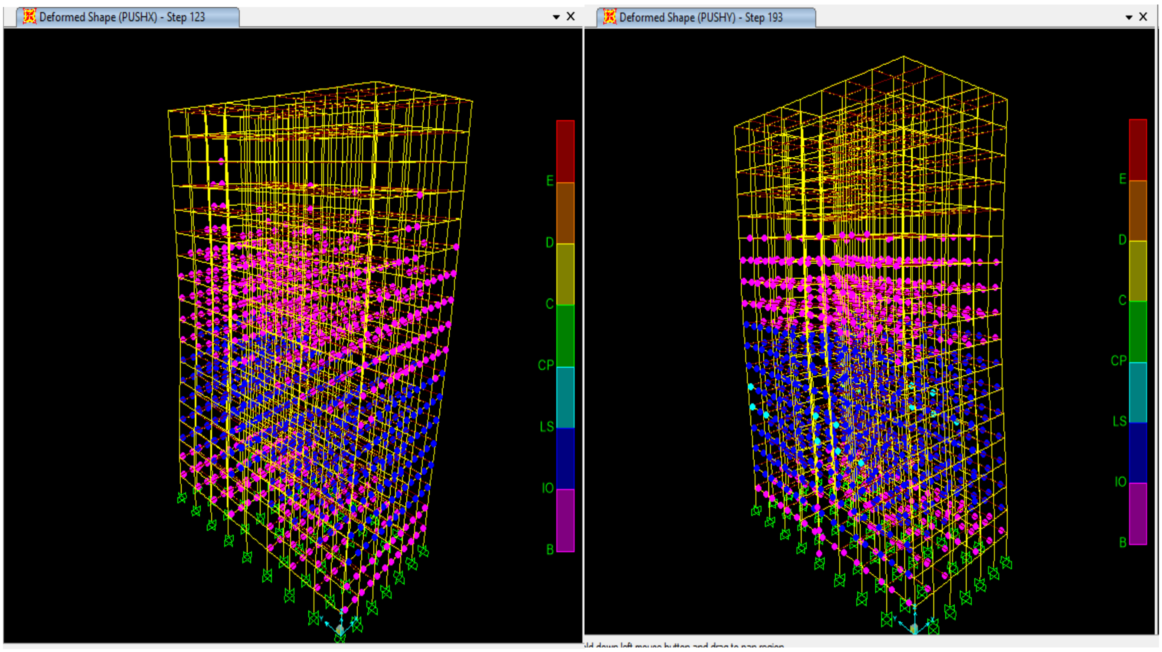


Fig37: Hinges formed at performance point for PUSHX and PUSHY in MODEL 5

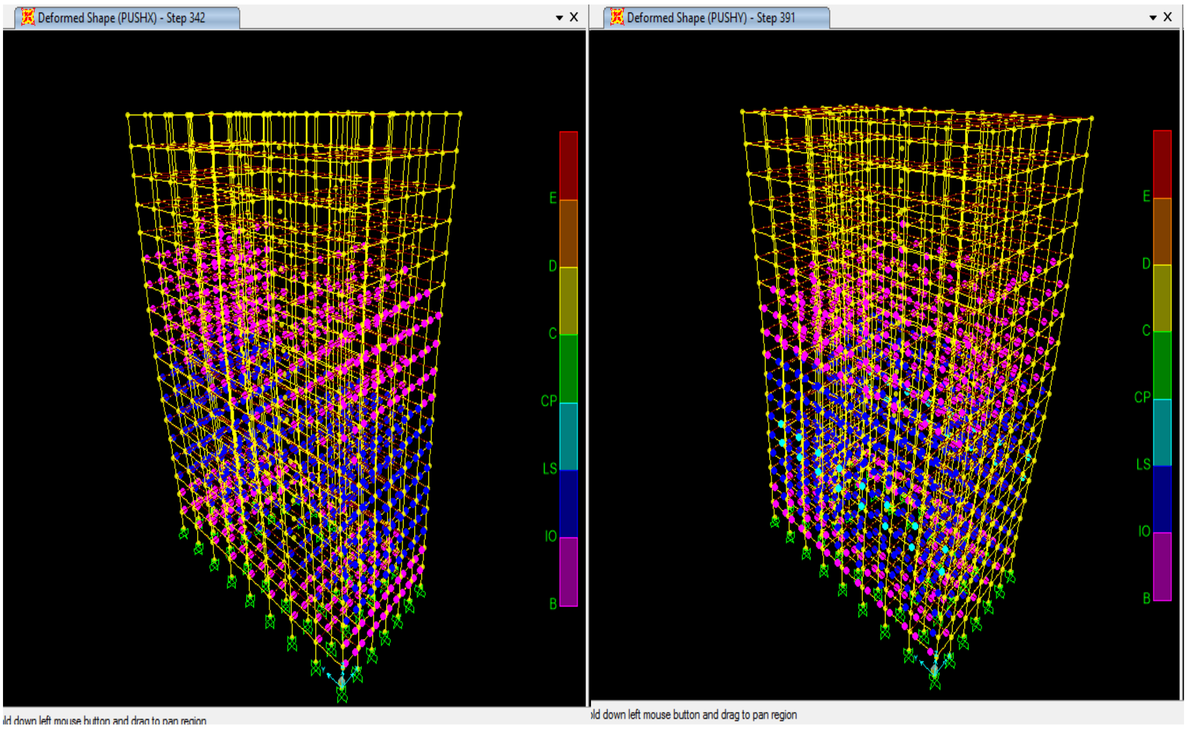


Fig38: Hinges formed at performance point for PUSHX and PUSHY in MODEL 6

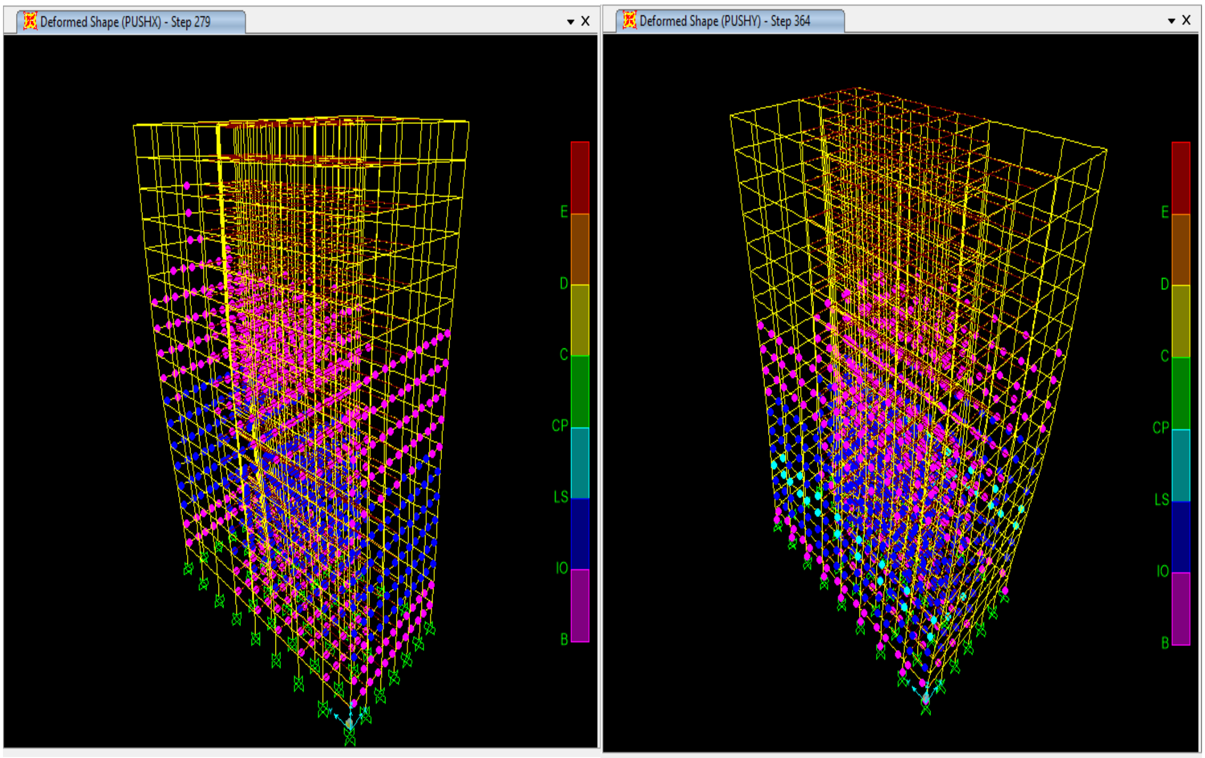


Fig39: Hinges formed at performance point for PUSHX and PUSHY in MODEL 7

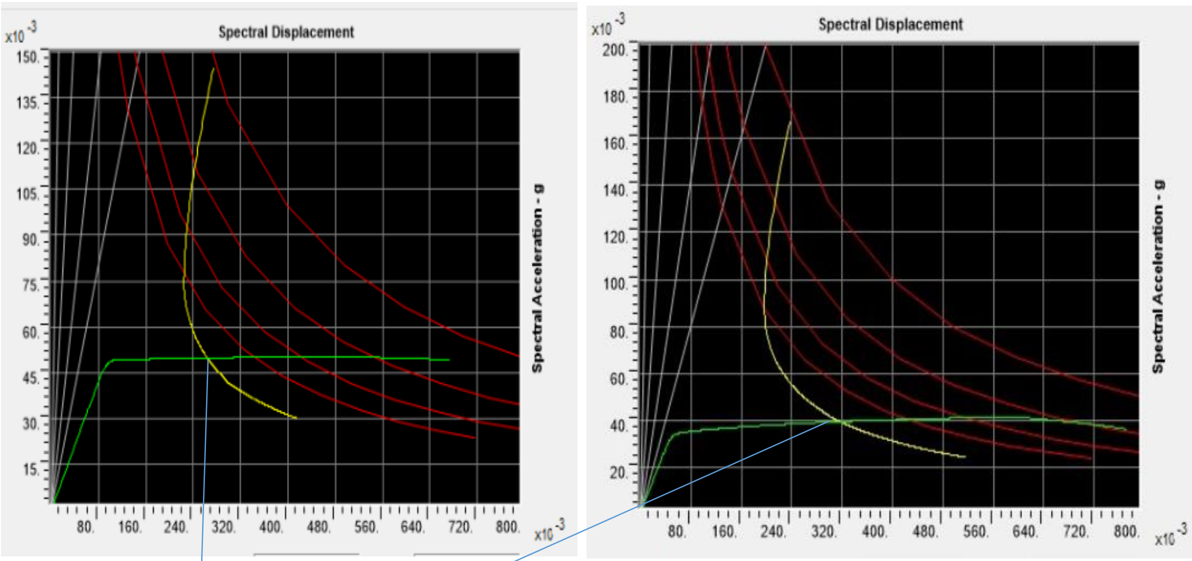
TABLE 18: Performance level at performance point

Model	Performance level at performance point	
	PUSHX	PUSHY
1	IO-LS	IO-LS
2	IO-LS	IO-LS
3	IO-LS	LS-CP
4	IO-LS	IO-LS
5	IO-LS	LS-CP
6	IO-LS	LS-CP
7	IO-LS	LS-CP

PUSHOVER CURVES IN BOTH DIRECTIONS

MODEL 1

ATC 40 curves obtained after complete pushover analysis is attached and the pattern shows Seismic demand curve meets the capacity curve, the point where these two curve meets is called performance point.



PUSHX PUSHY

Fig40: ATC 40 pushover curve for model 1

Performance point

Target displacement can be shown in FEMA 356 curve obtained after analysis in SAP2000.

MODEL 2

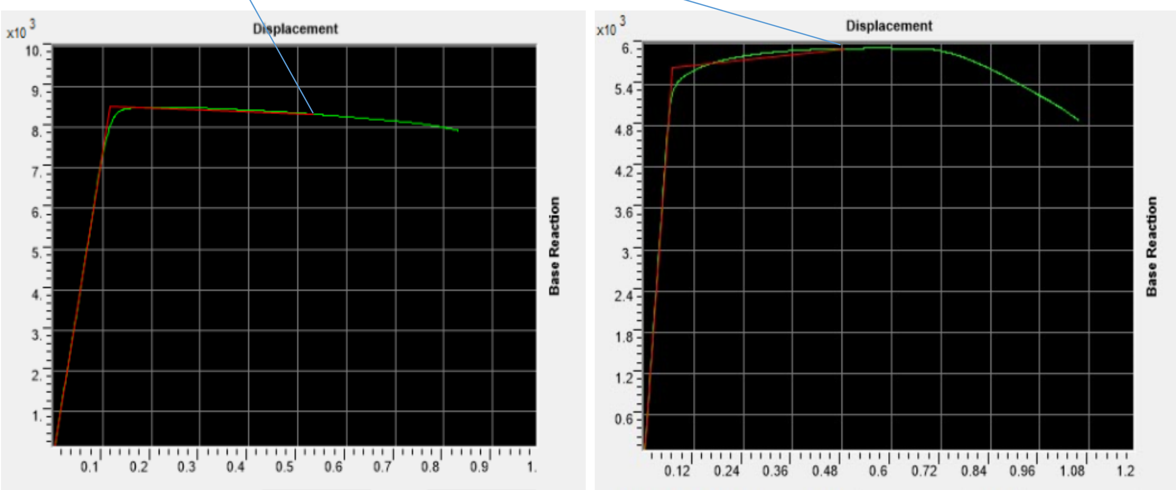
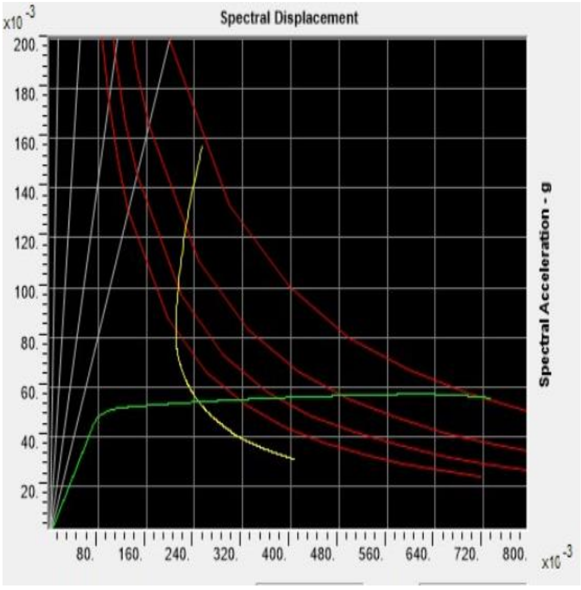
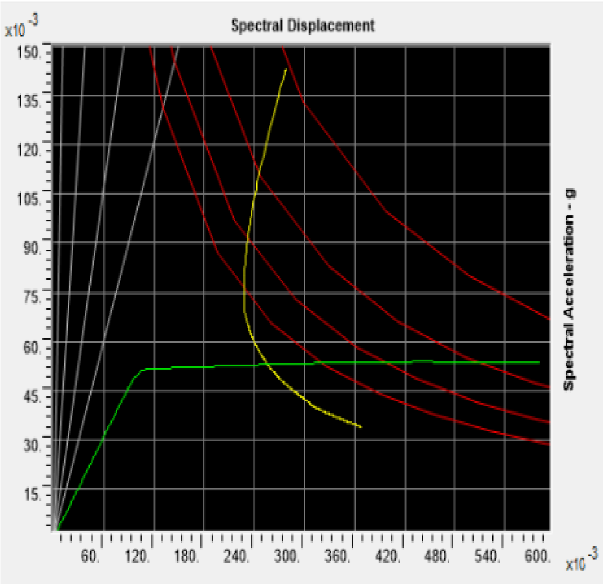


Fig41: FEMA 356 pushover curve for model 2

MODEL 3



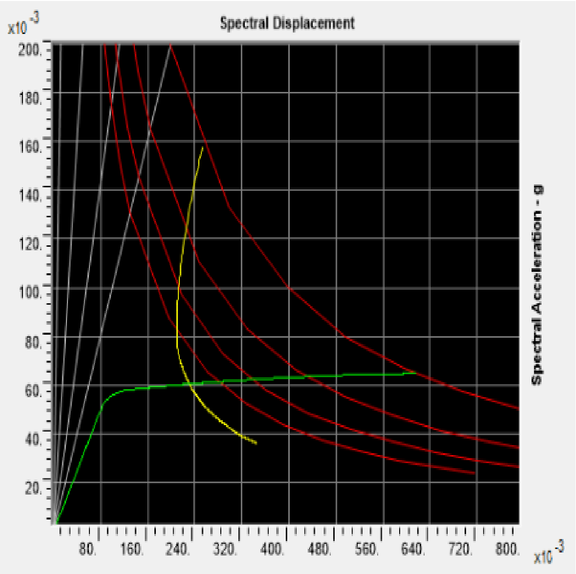
PUSHX CURVE



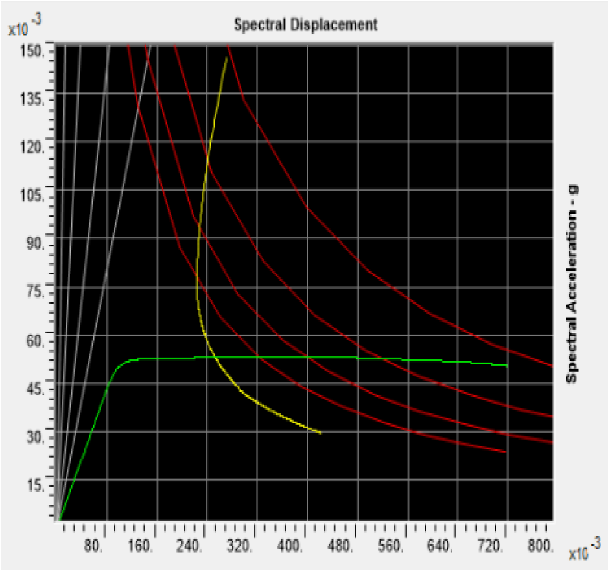
PUSHY CURVE

Fig42: ATC 40 pushover curve for model 3

MODEL 4



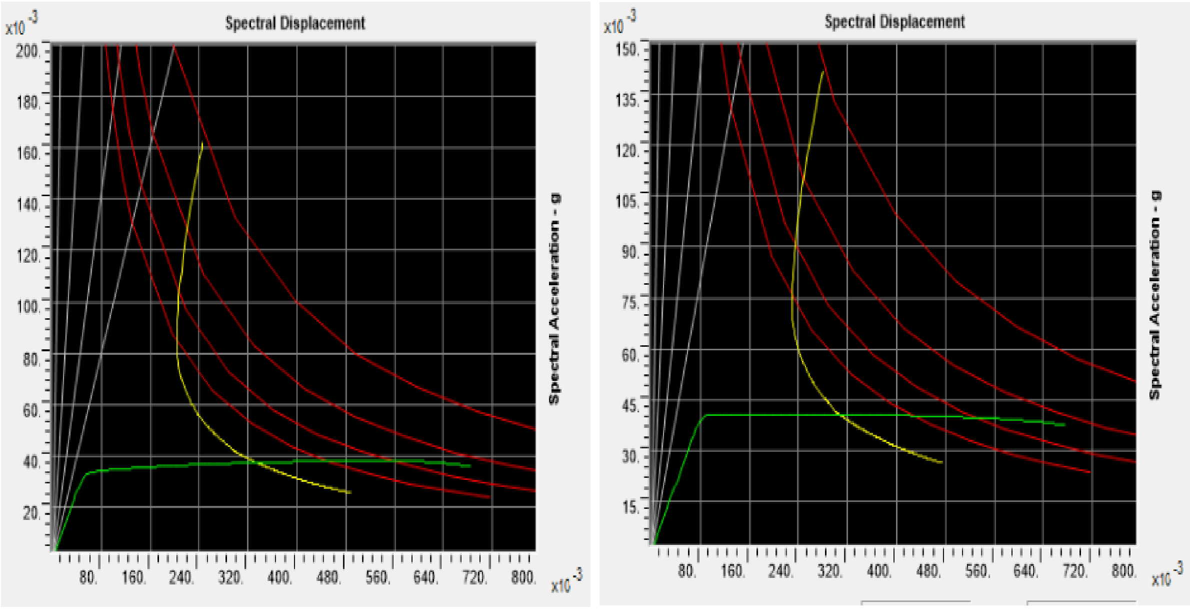
PUSHX CURVE



PUSHY CURVE

Fig43: ATC 40 pushover curve for model 4

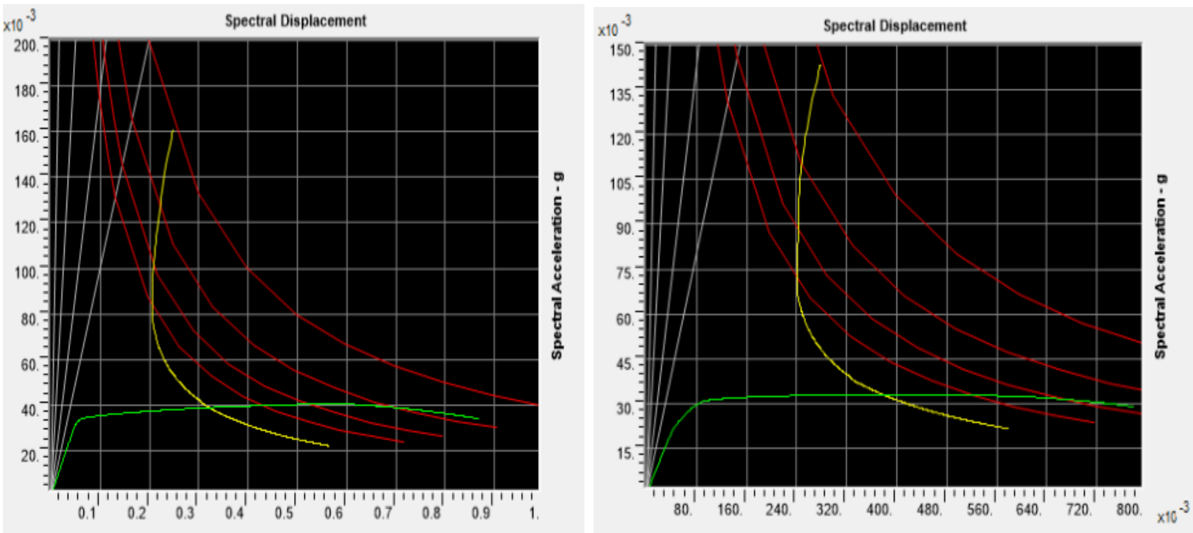
MODEL5



PUSHX CURVE PUSHY CURVE

Fig44: ATC 40 pushover curve for model 5

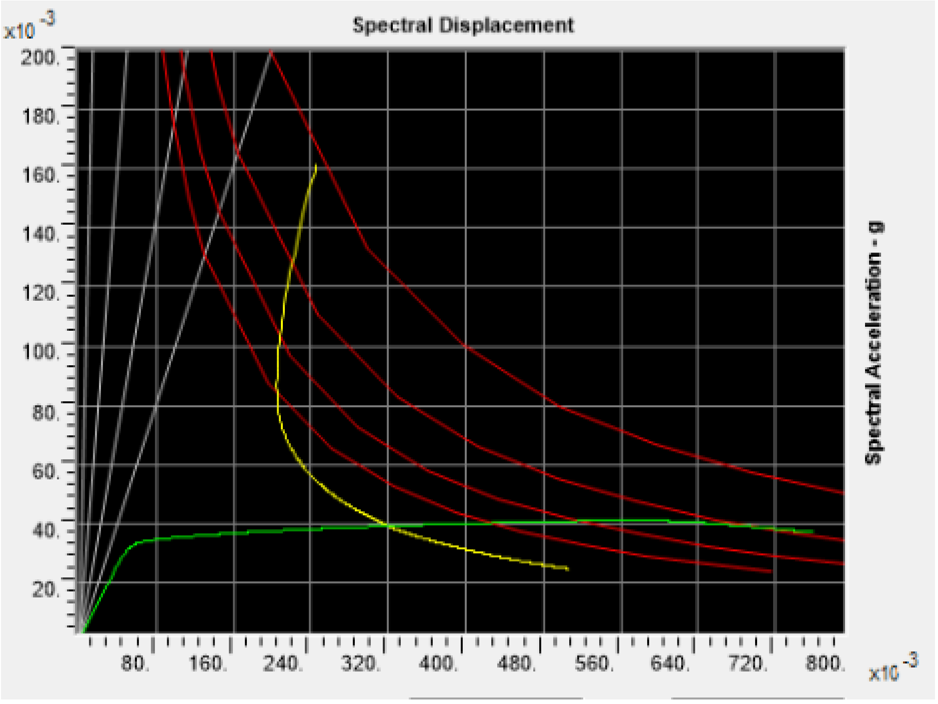
MODEL 6



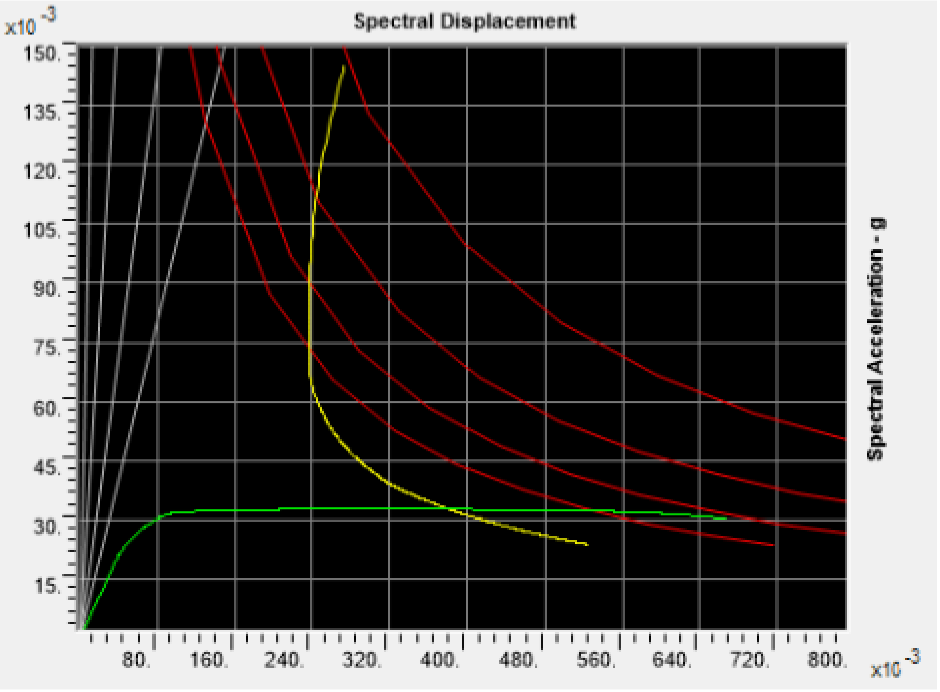
PUSHX CURVE PUSHY CURVE

Fig45: ATC 40 pushover curve for model 6

MODEL 7



PUSHX CURVE



PUSHY CURVE

Fig46: ATC 40 pushover curve for model 7

TABLE 19: Performance level of building at collapse condition

MODEL	PERFORMANCE LEVEL AT COLLAPSE	
	PUSHX	PUSHY
1	C-D	C-D
2	C-D	C-D
3	C-D	C-D
4	C-D	D-E
5	C-D	C-D
6	C-D	C-D
7	C-D	C-D

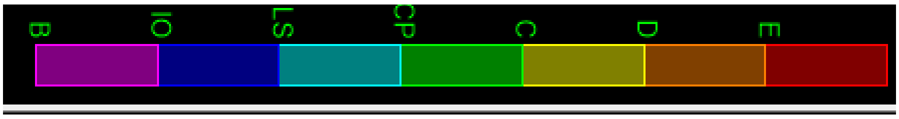


Fig47: Acceptance criteria for performance level

6.10 CONCLUSION

1. The influence of diaphragm discontinuity on the seismic response of multi-storeyed buildings played a key role in reducing the static and dynamic base shear, hence drawing lesser seismic forces.
2. The behaviour of building is improved when diaphragm discontinuity is closer to the centre of mass of the building.
3. From the results obtained we can conclude that the structures with diaphragm discontinuity are more flexible or in other words regular structures without diaphragm discontinuity are much stiffer than those containing diaphragm discontinuity.
4. Natural time period of regular model is higher than the others containing diaphragm discontinuity
5. Since natural frequency is inversely proportional to natural time period, the frequency of regular frames are lesser than others containing diaphragm discontinuity
6. Model 4 is most flexible than others and shows
7. For G+17 building with diaphragm discontinuity for all models modal mass participation is almost same. Therefore diaphragm discontinuity does not have much effect on modal mass participation.
8. In case of nonlinear pushover analysis base shear increases as compared to static analysis. In our analysis it increases by a factor of 1.3 to 1.7
9. Wind is not governing in our analysis, so all the designs were done considering earthquake forces.
10. Model 7 is more vulnerable to earthquake forces as hinges are in LS-CP performance level and no of hinges are much more than others in PUSHY.

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