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Analysis of G+30 Highrise Buildings by Using Etabs for Various Frame Sections in Zone Iv and Zone V

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Abstract:-From the ancient time, we know that the earthquake is a disaster causing occasion. Construction is now expanding more quickly than ever before, and engineers and researchers must also bear in mind that any possible sway and resulting damage from earthquakes should be kept to a minimum. In the past, scientists and engineers have experimented with ways to make buildings more earthquake resistant. The adoption of lateral load resisting techniques in the construction layout will significantly improve the performance of the structure during earthquakes, according to several functional studies. The current work will be done for the unique situations for High-rise RC building constructions taking shear wall system and bracing system into consideration. To examine the results of special building structures alongside conventional structures on seismic parameters like base shear, lateral displacement, and lateral drifts, as well as the performance of the structure, analysis and modeling for G+ 30 structures will be done using ETABS as a tool. According to IS 1893-2016, the analysis will be carried out for ZONES IV and V in soil type II (Medium Soils).

Keywords: Structural Analysis, Performance Analysis, Shear walls, bracing system, lateral displacement, lateral drift.

I. Introduction

Due to popular awe at the towering tall buildings rising in major cities like Chicago, New York City, Tokyo, Beijing, etc., the word "Tall building" was first used to refer to structures with at least 10 floors and a steel frame in the late 19th century. Architectural historians subsequently improved the technical meaning of the term “Tall building“basing it on technological advancements in the 1880s that made it possible to create towering, multi-story buildings. In contrast to load-bearing masonry buildings, which reached their practical maximum in 1891 with Chicago's Monad Nock Building, this concept was based on the steel skeleton. Tall building design and construction require creating livable, safe places in very tall structures. The structures must be able to hold their own weight, withstand wind and earthquakes, and safeguard inhabitants from fire. However, they must also be easily accessible, even on higher levels, and must provide the residents with amenities and a pleasant environment. Given the delicate balances between engineering, economics, and construction management, Tall building design issues are among the most challenging. Tall buildings are intricate constructions that need ongoing inspection and upkeep to maintain their structural integrity.

When there are structural defects in the horizontal load-bearing frames of a multi-story framework construction, earthquake damage often begins there. The organization of mass, stiffness, and strength in both the vertical and horizontal lines of buildings determine how multi-storey framework constructions behave during strong seismic movements. Recent earthquakes, including the 2015 Nepal earthquake, in which multiple reinforced concrete structures were seriously damaged or toppled, have raised the idea that existing structures should be evaluated for their seismic compatibility. When there are structural defects in the horizontal load-bearing frames of a multi-story framework construction, earthquake damage often begins there. The mass distribution, stiffness, and strength in both the vertical and horizontal lines of buildings are key factors in how multi-story framework structures respond to significant seismic disturbances.

Seismic techniques upgrading refers to alterations made to existing structures to strengthen their resilience to shaking, ground motion, and soil collapse brought on by earthquakes. Now that we have a greater knowledge of the seismic demand on structures and owing to recent experiences with large earthquakes near to urban centres, the requirement for seismic retrofitting is commonly known. Prior to the implementation of present-day seismic codes in the latter part of the 1960s for industrialized countries (US, Japan, etc.) & the late 1970s for many other parts of the world (Turkey, China, etc.), many structures were designed without enough detailing & reinforcement for seismic protection. Numerous research initiatives have been conducted in light of the urgent need.

To analyse the structure three models are developed as follows

Model I: RC Conventional Framed Structure

A typical kind of building construction known as a reinforced concrete (RC) framed structure makes use of reinforced concrete parts, such as columns, beams, and slabs, to give structural support and stability. RC framed constructions are often employed because of their power, toughness, and adaptability. Due to their stability and durability, reinforced concrete and steel reinforcement are appropriate for a number of building types and purposes. The durability and safety of RC framed buildings depend on good design, construction, and maintenance procedures. Since concrete and steel reinforcement together provide strength, flexibility, and longevity, RC framed structures are often employed in residential, commercial, and industrial projects. (See Fig. 1)

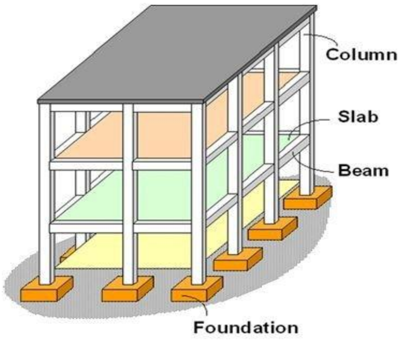


Fig.1RC Conventional structure

Model II: Shear wall Structure

A shear wall needs a solid vertical diaphragm in order to transport lateral loads from exterior walls, floors, & roofs to the ground's foundation along a direction parallel with the planes. The utilization of RC shear walls is designed for use in constructions located in seismic zones because to its excellent strength, stiffness, and ductility. Correctly constructed shear walls not only guarantee safety, but also a sufficient degree of protection against costly structural & non-structural damage amid seismic activity. Because shear walls offer structures a large lot of stiffness & strength, the lateral movement of the structure is significantly reduced, minimizing structural damage. If shear walls are properly designed and constructed, they will be robust & stiff enough to resist horizontal forces. Shear walls play a significant role in high-rise constructions that are susceptible to lateral wind & seismic pressures.(See Fig. 2)

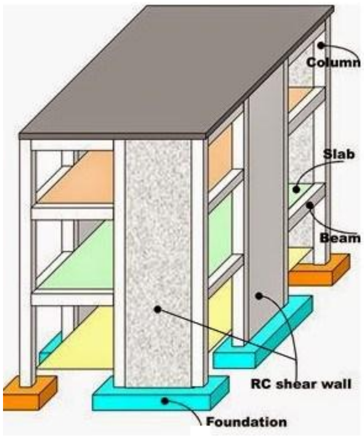


Fig.2Shear wall structure

Model III: Bracing Structure

A very effective and affordable way to withstand horizontal stresses in a frame building is concrete bracing. One of the most important retrofit techniques is bracing, which has been utilized to support the bulk of the world's tallest building structures laterally. Because the diagonals operate under axial stress, bracing is effective because it only requires the smallest possible member sizes to provide stiffness and strength against horizontal shear. In order to increase the strength and/or ductility of existing structures, many academics have looked at a variety of procedures, including infilling walls, adding walls to existing columns, encasing columns, and adding concrete bracing. By improving the lateral stiffness and capacity of the frame, a bracing system enhances the seismic performance of the structure. The bracing system allowed weight to be moved from the frame and into the braces instead of the weak columns, so boosting strength. For structures vulnerable to lateral seismic or wind loads, concrete braced frames are effective structural solutions. Therefore, it makes sense to upgrade reinforced concrete frames with insufficient lateral resistance using concrete bracing systems.(See Fig. 3)

Fig.3 Bracing Structure

II. Literaturereview

Rosinblueth and Holtz et al., In their investigation of entirely uniform structures, gave a solution for a differential equation along with tables that are useful for symmetric structures. If the shear wall is considerably stiffer than the remaining components of the structure, the approach calls for the first approximation, and this gets better in subsequent attempts, to characterize the shear wall in the entire load; otherwise, the first distribution for horizontal stress among walls as well as frames could vary noticeably.[1]

Mo and Jost (1993) et al., among others According to the findings of this research, the maximum deflection for the El Centro record decreased by 30% as a consequence of the influence of concrete strength on the framed shear walls, which were caused by raising concrete strength from 25 MPa to 35.0 MPa. It aids in increasing the maximum shear force by 56% for ten-storey shear walls as well as the maximum deflection by 27% and the maximum shear force by 30% for five-storey shear walls. Steel yielding stress from 413 MPa to 482 MPa has very little impact. Shear reinforcing thus proved inadequate to prevent an early shear failure at the crucial portion. [2]

SatishAnnigiri and Ashok K. Jain (1994) et al., They conducted study on the distribution of storey and floor eccentricity for various lateral load distributions along the building's height. The dynamic techniques defined as per IS: 1893 (1984) and UBC (1991) standards are assessed together with the static methods as per code for torsional analysis. Due to inadvertent eccentricity, the primary thrust. Both a two story, framed-shear wall building and a six story structure with setbacks were taken into consideration. It is appropriate to do a 3-D dynamic study of an asymmetric construction that takes into account the impact of unintentional eccentricity. In order to describe how to calculate design eccentricity and account for accidental torsion, both in static and dynamic analysis, it is necessary to upgrade the torsional provisions in IS: 1893 explanatorily.[3]

Iii. Methodology

Technique for study purpose various soil circumstances whichever is provided in IS456 in use in ETABS program. According to IS456 the Light, Medium, Rigid Strata with Variable base supports Based on movement and weight relation optimum construction were determined. Basic to ETABS planning is the assumption that multi-story structures usually comprise of the same or comparable floor layouts that recur in the vertical position. Planning characteristics that simplify analytical-model creation, and mimic sophisticated earthquake systems.

3.1. Response Spectrum Analysis

Following IS-1893:2002 guidelines, the cumulative sum of modal masses from all considered modes in the analysis should amount to a minimum of 90% of the total seismic mass. For structures lacking horizontal plan irregularities, like those described in ASCE 7-05 and the Diaphragm Planning Guide, concrete slab diaphragms or concrete-filled metal decks with a span-to-depth ratio of 3:1 can be simplistically treated as rigid. Alternatively, if this condition is not met, the structural analysis must explicitly account for the diaphragm's stiffness, albeit without extensive elaboration. Nasser et al. (1993), Mansur et al. (1999), and Abdalla and Kennedy (1988) have presented insights into the effects of openings within rectangular reinforced or prestressed concrete beams, shedding light on stress distribution and the load-bearing capacity of concrete beams containing voids. However, it is important to note that this theory appears to have been primarily validated against available experimental findings, with limited extension to encompass other configurations.

3.2. Pushover Analysis:

During a powerful earthquake, buildings undergo significant inelastic deformation, causing the dynamic characteristics of the structure to evolve over time. As a result, the analysis of structural performance requires the use of inelastic scientific methods that accurately capture these evolving dynamics. Inelastic analytical techniques enhance our understanding of structures by identifying failure modes and assessing the potential for dynamic structural failure. Inelastic analysis methods encompass a combination of inelastic time history analysis and the utilization of inelastic data collected, a process that could otherwise be referred to as pushover analysis. Among these methods, the elastic-plastic time history analysis stands out as the most precise approach for predicting the forces and displacements experienced by various components of a building.

3.3. Objectives Of Study

A thorough literature study is carried outside to describe the goals of present work.

1. To decide the capacity of different structures compared to conventional reinforced concrete structure as a parallel load opposing individuals.
2. Dynamic investigation of the tall framed structures considering response spectrum examination.
3. Utilization of Advanced diagnostic applications of software like Staad.Pro, Etabs for story response plot examination of horizontal load opposing structure and the inter story displacements.
4. To decide the capacity and dynamic investigation in the terms of maximum story displacement and story drift of the tall framed structure subjecting to IS load combinations.

Iv. Buildingmodellingandanalysis

First choose the material property in define, then add the necessary material that is used in the design of the g+30 structure, to do an analysis in ETABS.

By choosing define menu material properties in this case, we had first specified the material property. By providing the necessary information in the defining tab, we introduced new material for our structural components (beams, columns, slab, shear wall, and bracing). Then, by choosing the frame sections as shown below, we defined section size and added the necessary sections for beams, columns, etc. and the details are presented in Table 1, 2 & 3. The plan layout of the structure is depicted in Fig. 4. Dead load distribution on beams and slabs is shown in Fig 5 & 6 respectively. The live load distribution is presented in Fig. 7. Wind pressure coefficients are depicted in Fig. 8 and hinge properties in Fig. 9.

Table 1: Geometrical properties & location factors

Building type	G+30
Plan dimensions	45 x 35 m
No. of bay in X direction	9 Bays
No. of bay in Y direction	7 Bays
Typical storey height	3.3 m
Bottom storey height	3.0 m
Building height	105.3 m
Soil type	Type II (Medium Soils) Combined or Isolated RCC footings with the beams
Design criteria	(As Height of building is greater than 40m up to 90m type) Analysis for all zones. Modal analysis using Response spectrum method and for performance Time history or Push-over analysis is to be performed for the maximum deformed zone.
Zone considering	IV & V
Importance Factor, I	1
Response Reduction Factor, R	5 (SMRF) RC Building with Special Moment Resisting Frame
Performance factor, K	1.0 (Moment resistant frame with appropriate ductility details as given in IS: 437.6-1976* in reinforced concrete or steel)
Support condition of columns	Fixed

Table 2: Section & material properties

Column size	450 x 600 mm
Beam size	300 x 450 mm
Thickness of slab	150 mm
Grade of concrete	M-40
Grade of steel	Fe-550
Shear wall size	230 mm
Bracing System	X Bracings

Table 3: Loading details

Wall load on external beams	13.11 kN/m
Wall load on internal beams	8.55 kN/m
Floor finish load	1.5 kN/m^2
Live load on floor	2 kN/m^2
Terrace finish load	1.5 kN/m^2
Dead load factor	1
Live load factor	0.25 (i.e., 25%)
Load combination considering live load	1.2[DL + IL ± (EL$_X$ ± 0.3 EL$_Y$)] and 1.2[DL + IL ± (EL$_Y$ ± 0.3 EL$_X$)] and
Load combination without considering live load	1.5[DL ± (EL$_X$ ± 0.3 EL$_Y$)] and 1.5[DL ± (EL$_Y$ ± 0.3 EL$_X$)] and

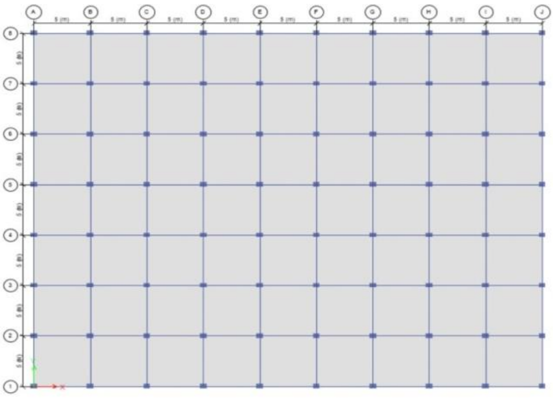


Fig 4. Plan Layout of structure

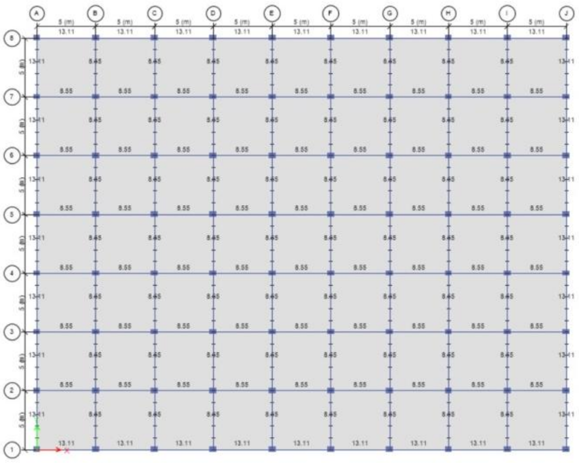


Fig 5. Dead Load on Beams

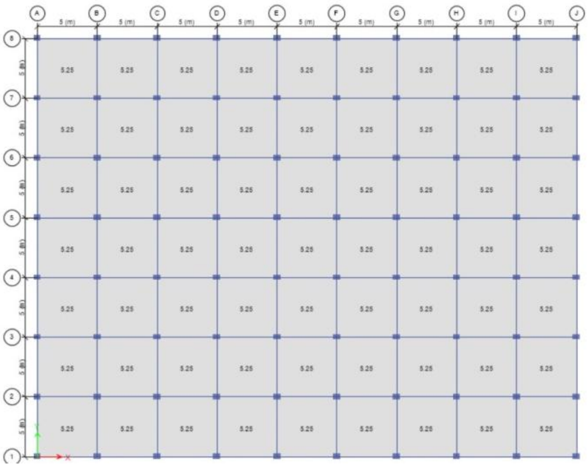


Fig 6. Dead Load on Slab

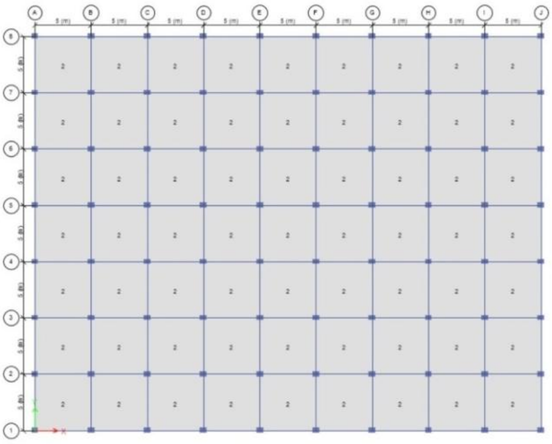


Fig 7. Live load on slab

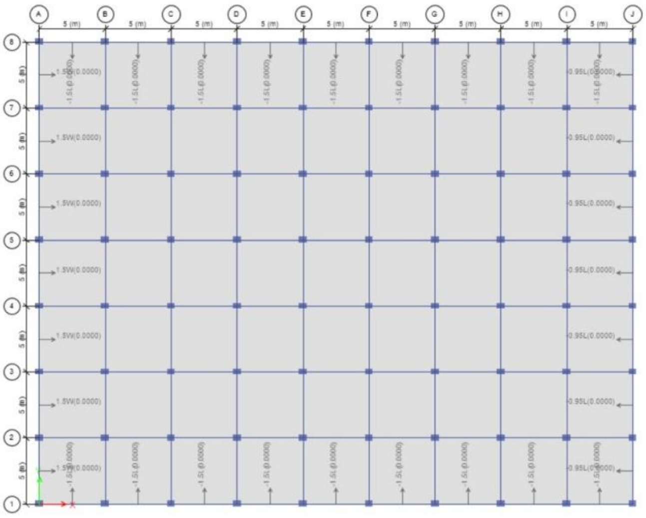


Fig 8. Wind pressure co-efficients of structure

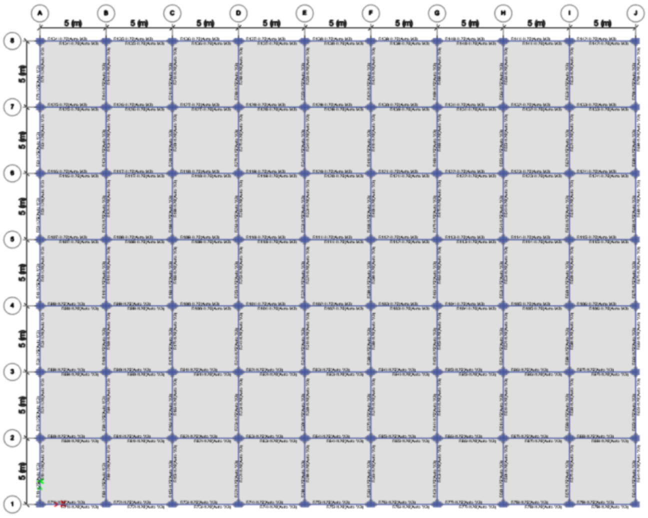


Fig 9. Hinge Properties

The output and display formats for moment, shear and deformed shapes are available after assigning all properties to beams, columns, slab shear walls, bracings, and applying loads. These may be arranged into customizable reports and intricate section cuts illustrating different local response measures. 3D-views of Model I, II & III, and the deformed shapes are shown in Fig 10-15.

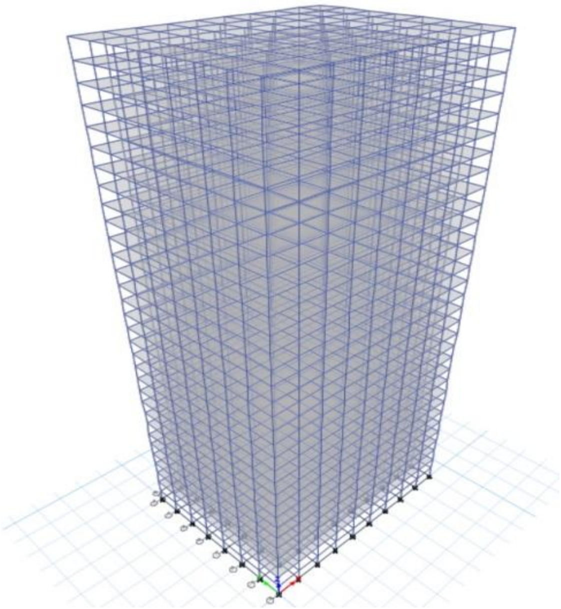


Fig 10. 3D view of Model I

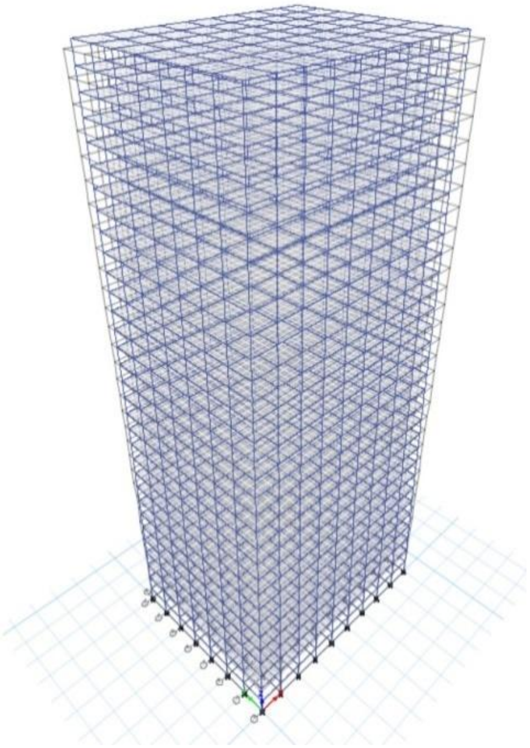


Fig 11. Deformed shape of Model I

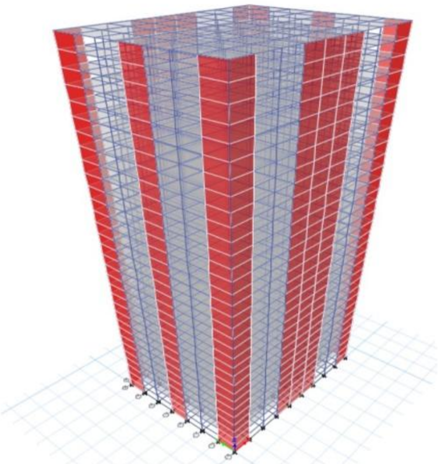


Fig 12.3D View of Model II

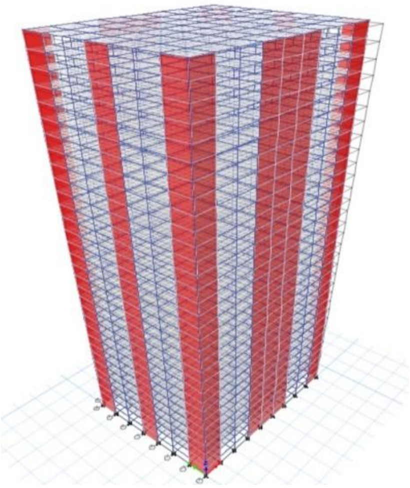


Fig 13.Deformation of Model II

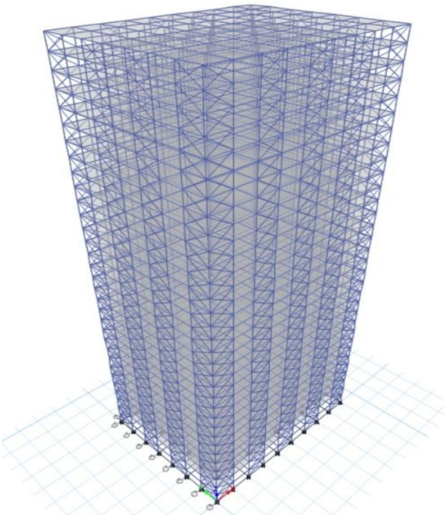


Fig 14.3D View of Model III

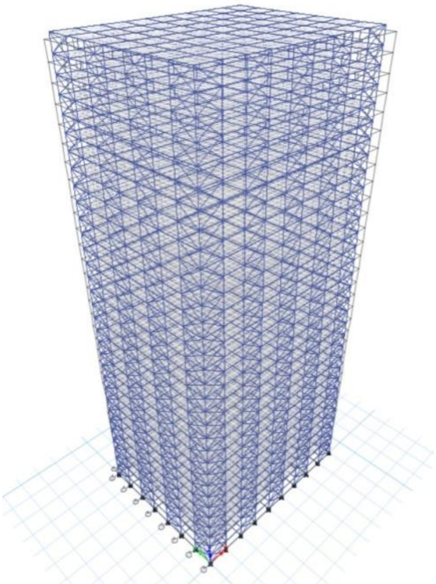


Fig 15.Deformation of Model III

Here the deformation which caused due to performing response spectrum analysis for the RC conventional structure, in the same manner wegot the results of different structure subjected to seismic effects andperforming the response spectrum analysis are presented below in results and discussions.

V. Results And Disscusions

Pushover analysis is used to assess the selected construction model. Pushover analysis was initially carried out using response spectrum analysis to describe gravity and imposed kinds of loads for the earthquake regions IV and V. Then, utilizing displacement control, a lateral non-linear pushover study was completed to analyse the strongest seismographic influence on the RC framed structure. The analysis has been carried out for both ZONE IV and Zone V. The terms in which the findings of the response spectrum are shown as plots for stories are as follows.

5.1.1 Maximum Story Displacement - (Response Spectrum)

The story lateral displacement with respect to the base is referred to as story displacement. The excessive lateral movement of the building may be controlled by the lateral force-resisting system. The acceptable lateral displacement limit in the event of a wind load is H/500.

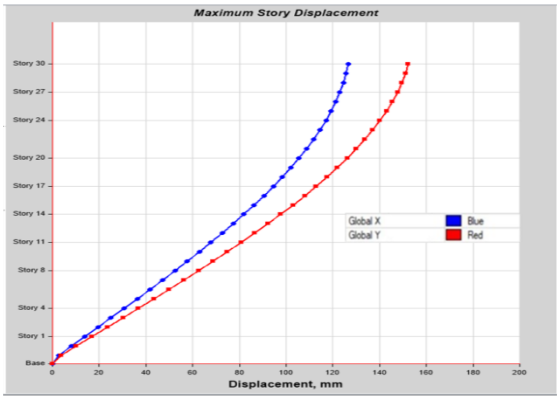


Fig 16. Model I (ZONE IV)

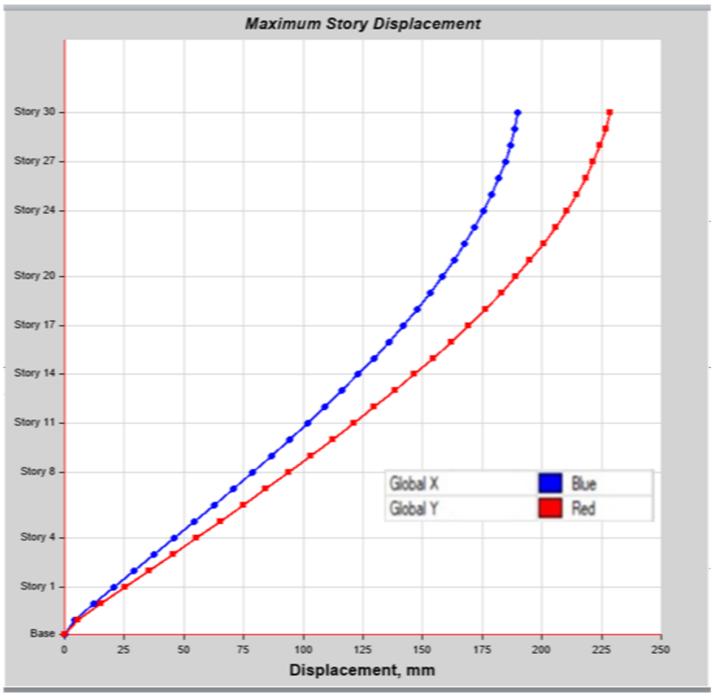


Fig 17. Model I (ZONE V)

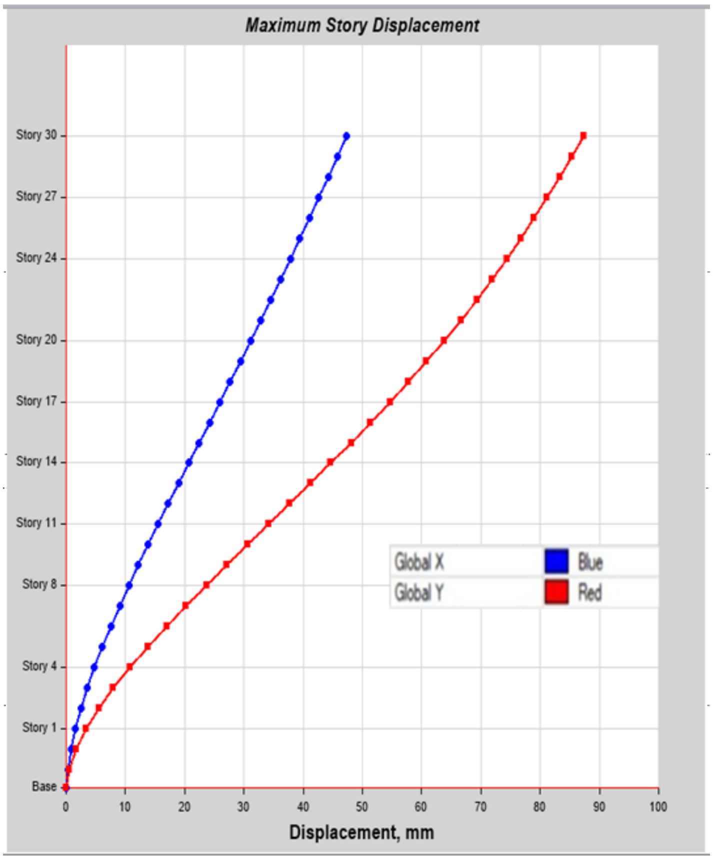


Fig 18. Model II (ZONE IV)

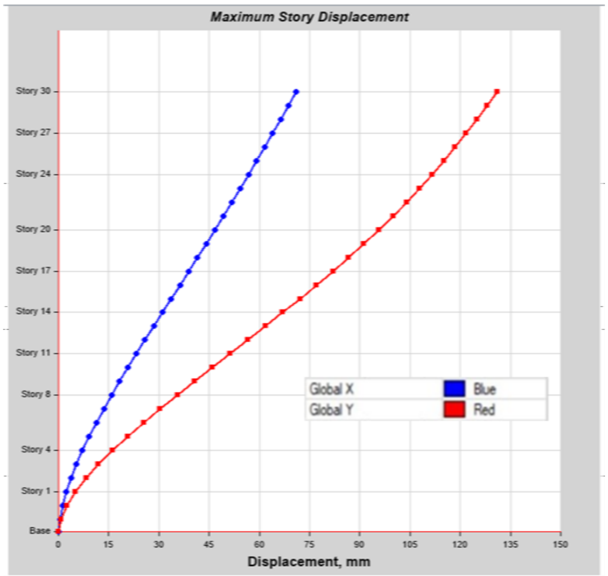


Fig 19. Model II (ZONE V)

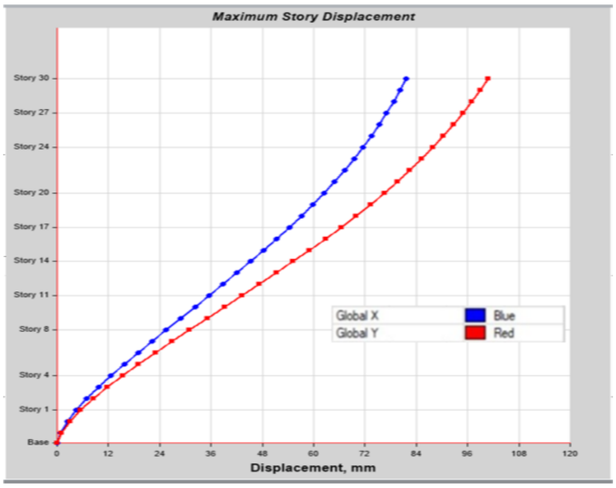


Fig 20. Model III (ZONE IV)

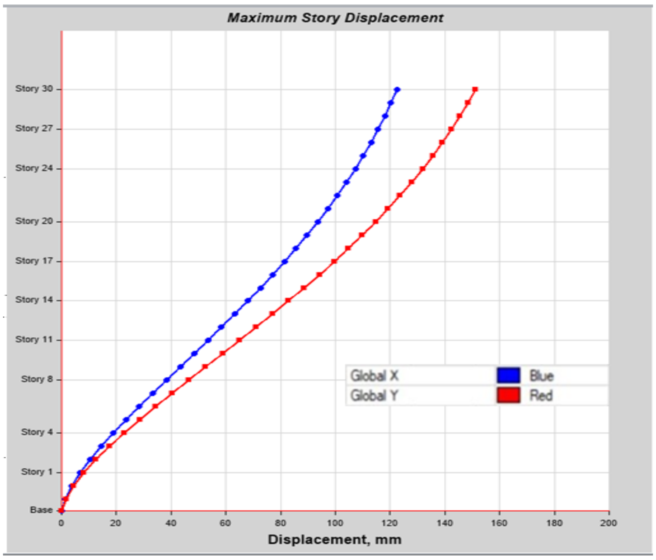


Fig 21. Model III (ZONE V)

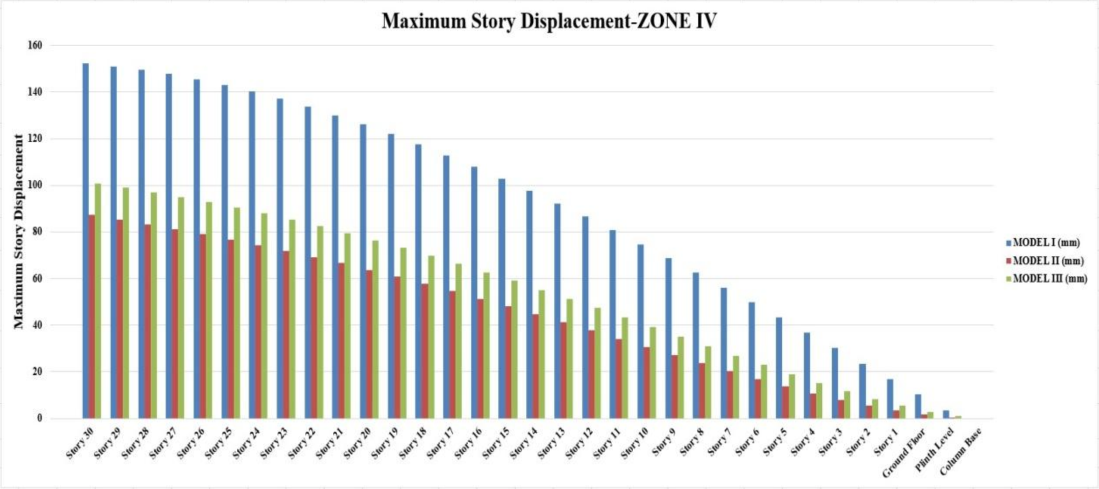


Fig 22. Comparison of Maximum Story Displacement (ZONE IV)

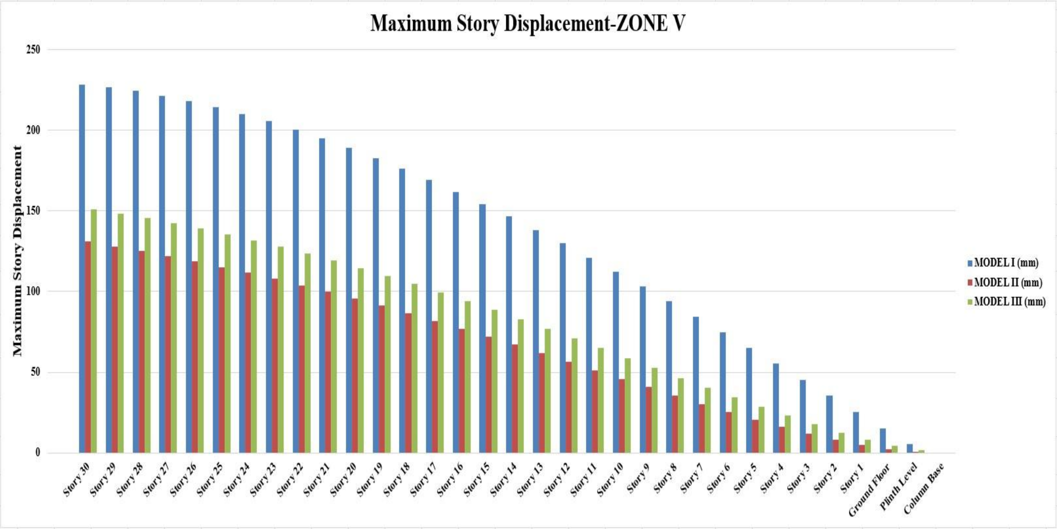


Fig 23. Comparison of Maximum Story Displacement (ZONE V)

The maximum story displacement for Model I, Model II, and Model III in both zones taken into consideration occurred at the top story(story 30) of the structure. According to the results of the response spectrum study, In Zone IV and Zone V respectively the maximum story displacement values for the Model I are 152.351 mm and 228.527 mm. The maximum story displacement values for the Model II are 87.325 mm in Zone IV and130.987 mm in Zone V respectively. In Zone IV and Zone V the maximum story displacement values for the Model III are 100.815 mm and 151.222 mm respectively. When we evaluate the three Models individually, Zone V has substantially higher story displacement values than Zone IV. Model II in Zone V has a maximum story displacement of 130.987 mm which is safer and far lower than Model I in Zone V which has a maximum story displacement of 228.527 mm. Model III is also superior to Model I, because the story displacement in Model III and Model I is 151.222 mm and 228.527 mm, respectively which we can see in fig 16 - 23.

5.1.2 MAXIMUM STORY DRIFT- (Response Spectrum)

Story drift is calculated by dividing the distance between two adjacent stories by the height of each story.

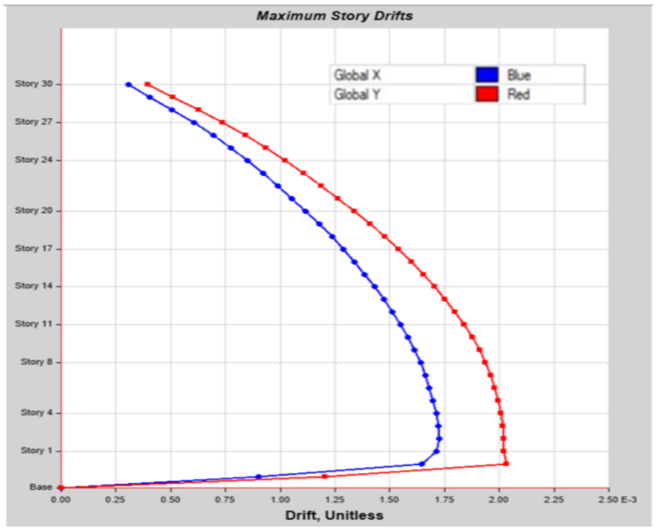


Fig 24. Model I (ZONE IV)

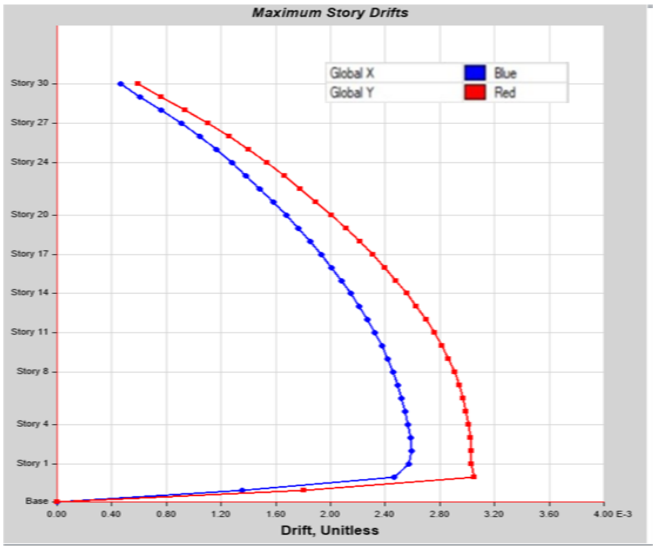


Fig 25. Model I (ZONE V)

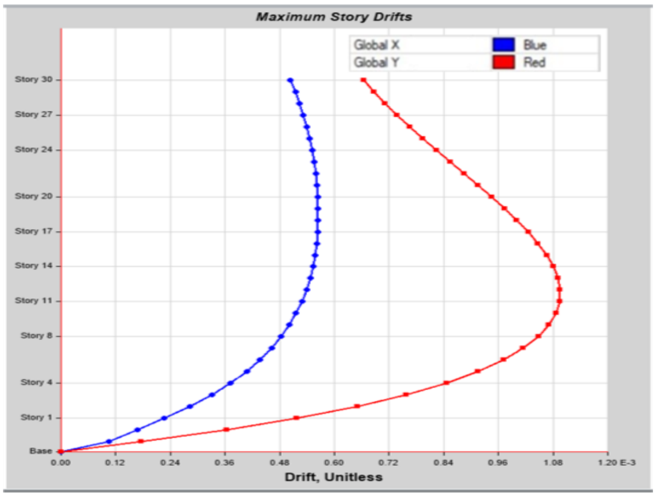


Fig 26. Model II (ZONE IV)

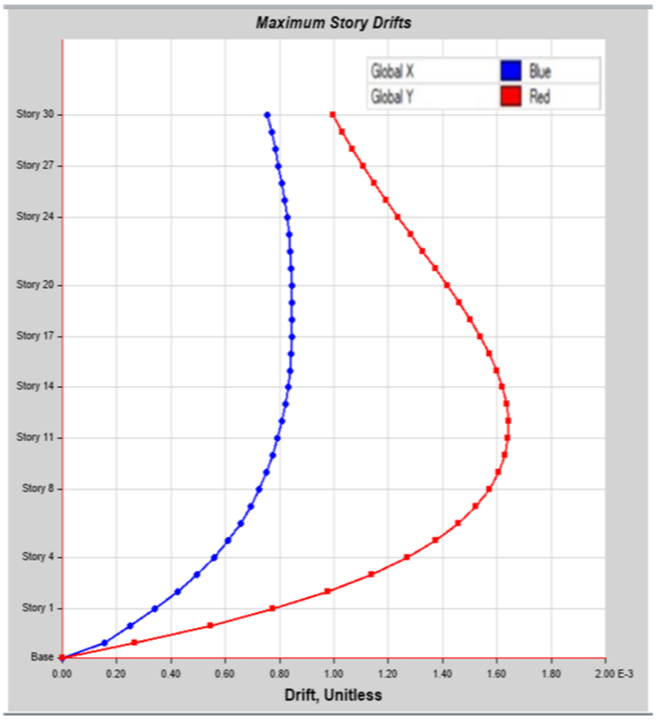


Fig 27. Model II (ZONE V)

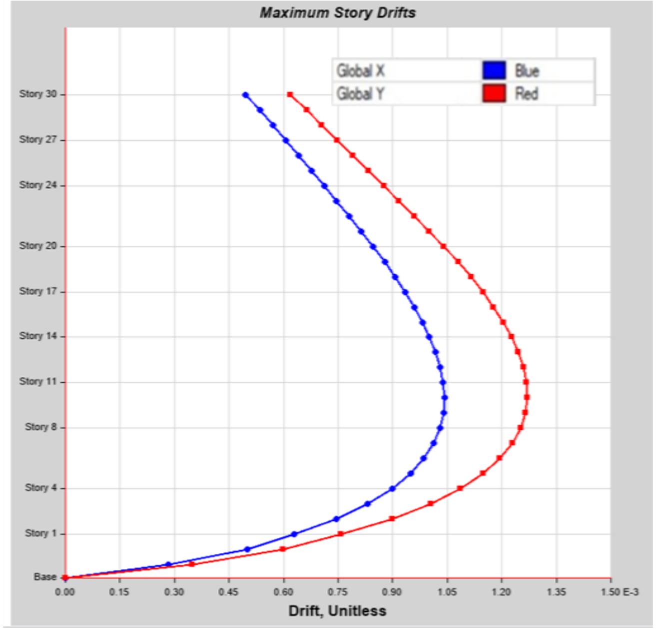


Fig 28. Model III (ZONE IV)

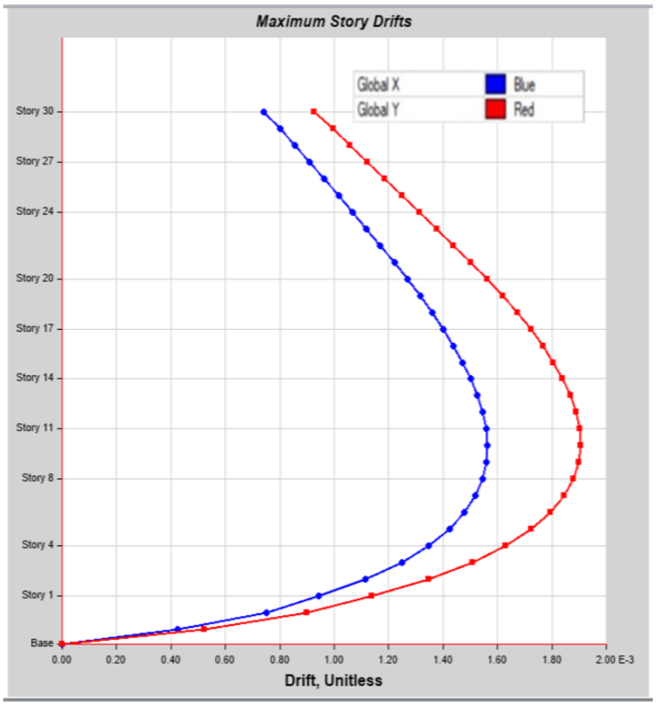
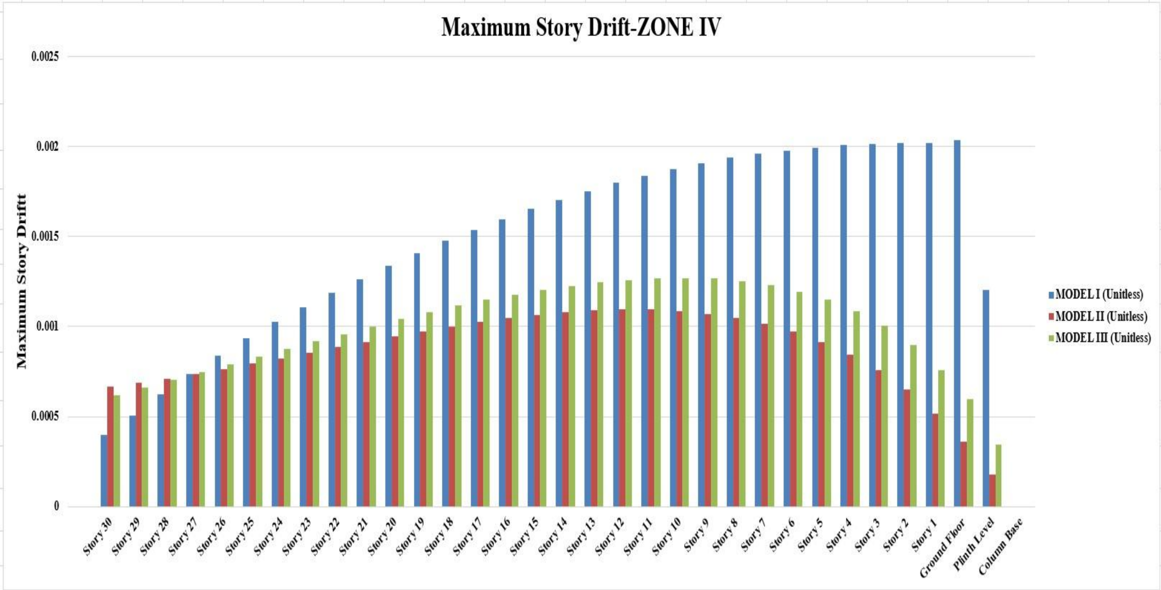


Fig 29. Model III (ZONE V)

Fig 30. Comparison of Maximum Story Drift(ZONE IV)



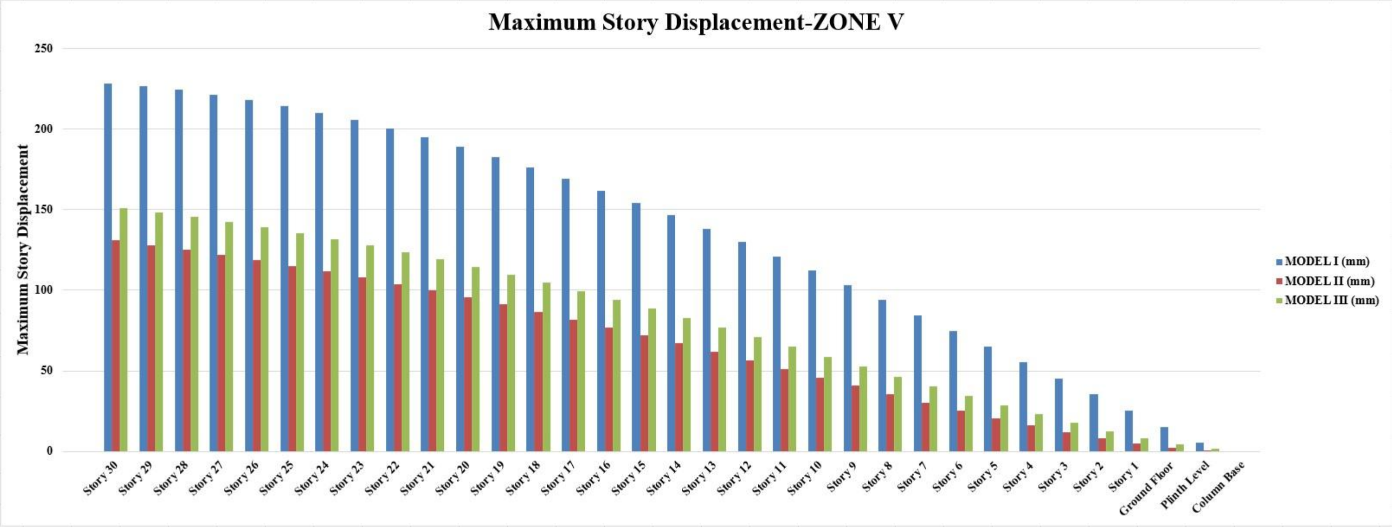


Fig 31. Comparison of Maximum Story Drift (ZONE V)

The maximum story drift for Model I, Model II, and Model III in both zones taken into consideration occurred at the top story (story 30) of the structure. According to the results of the response spectrum study, In Zone IV and Zone V respectively the maximum story drift values for the Model I are 0.002034 and 0.003052. The maximum story drift values for the Model II are 0.001095 in Zone IV and 0.001643 in Zone V respectively. In Zone IV and Zone V the maximum story drift values for the Model III are 0.001271 and 0.001907 respectively. When we evaluate the three Models individually, Zone V has substantially higher story drift values than Zone IV. As the maximum story drift should be less than 0.004 the models are in safer zone. Model II in Zone V has a maximum story drift of 0.001643 which is lower than Model I in Zone V which has a maximum story drift of 0.003052. Model III is also superior to Model I, because the story drift in Model III and Model I considering Zone V is 0.001907 and 0.003052, respectively which we can see in fig 24 - 31.

5.1.3 MAXIMUM STORY SHEAR- (Response Spectrum)

The total of the lateral pressures exerted at each level of the structure is the maximum story shear. As floor forces are added from the top to the bottom of the building to determine cumulative story shears, they should increase as you descend.

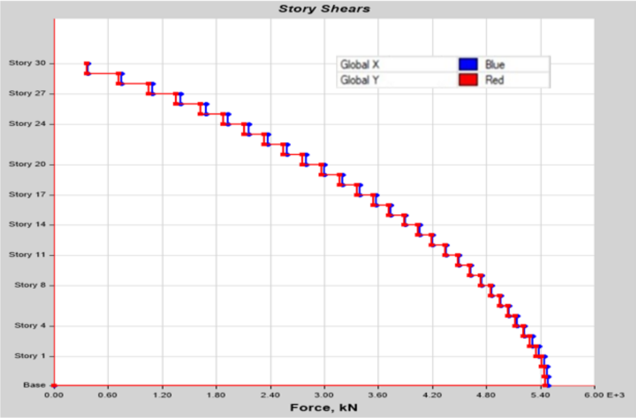


Fig 32. Model I (ZONE IV)

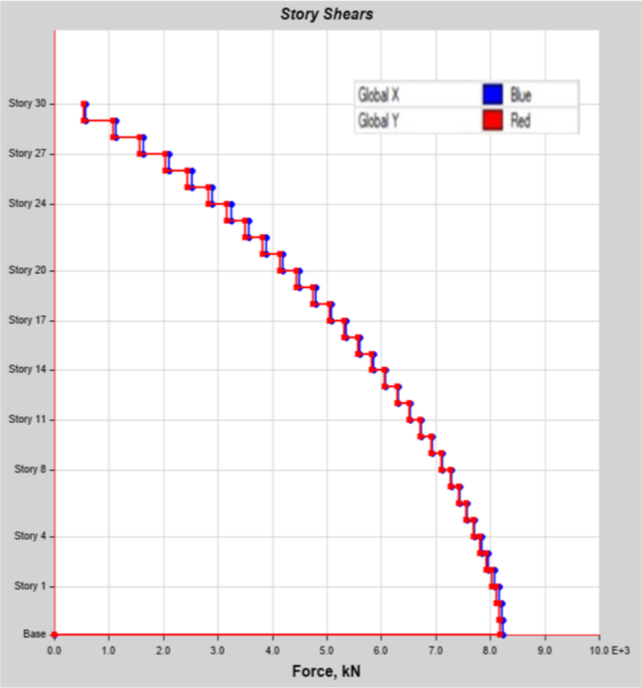


Fig 33. Model I (ZONE V)

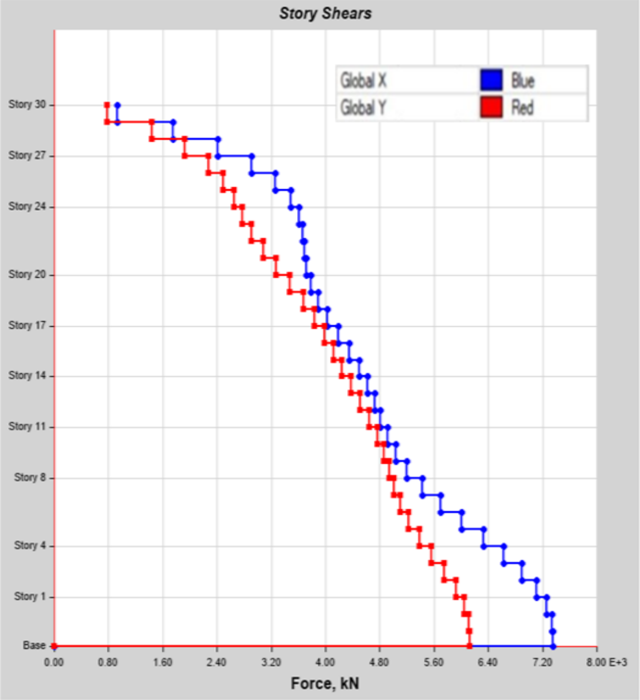


Fig 34. Model II (ZONE IV)

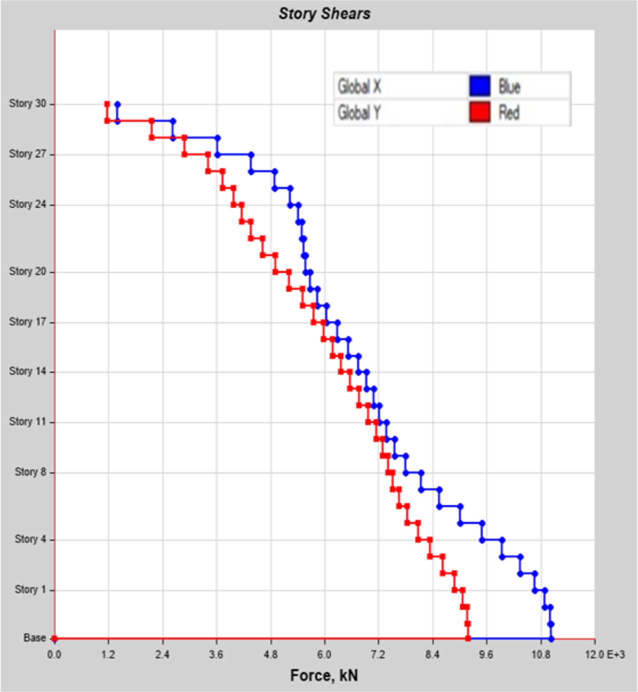


Fig 35. Model II (ZONE V)

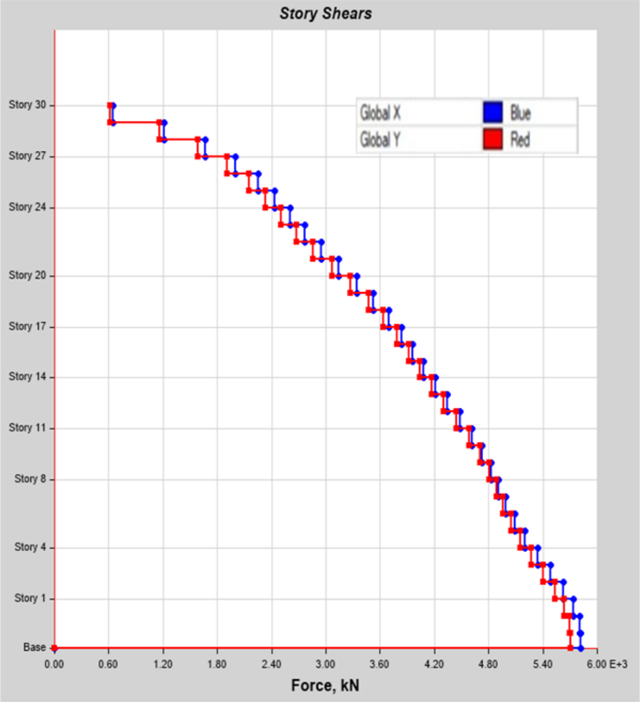


Fig 36. Model III (ZONE IV)

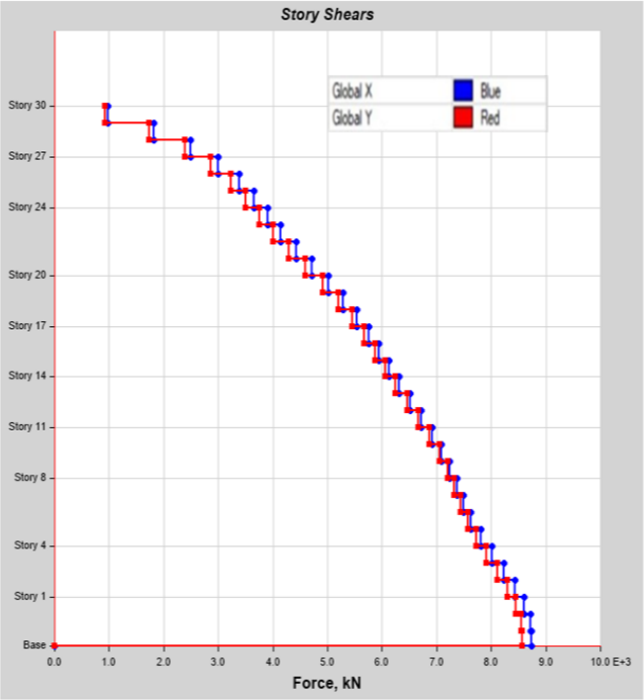


Fig 37. Model III (ZONE V)

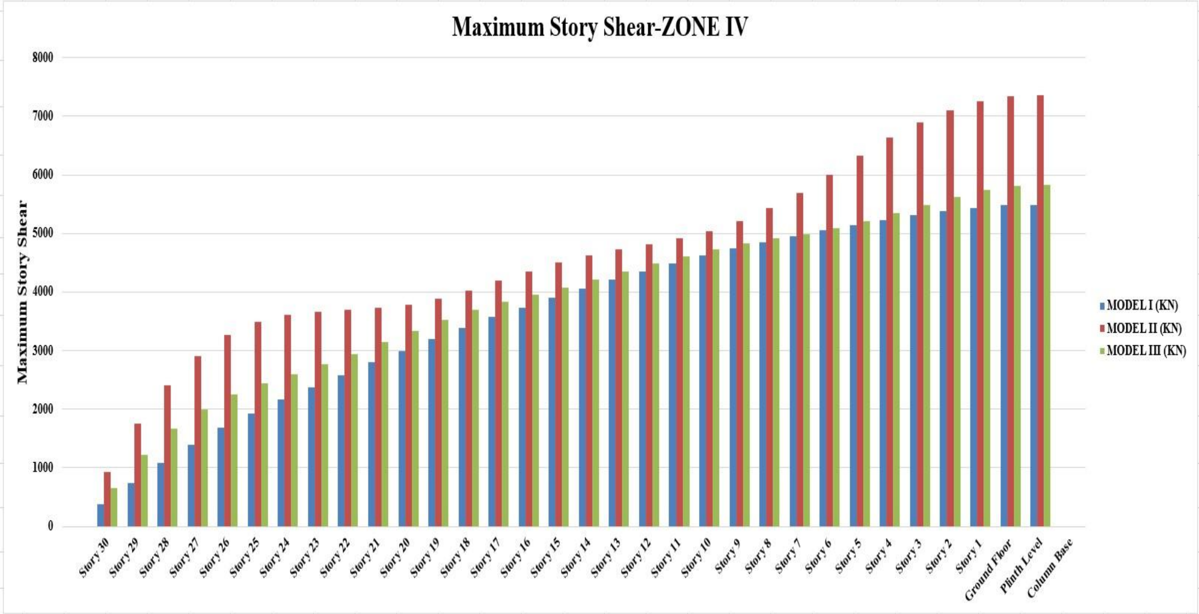


Fig 38.Comparison of Maximum Story Shear (ZONE IV)

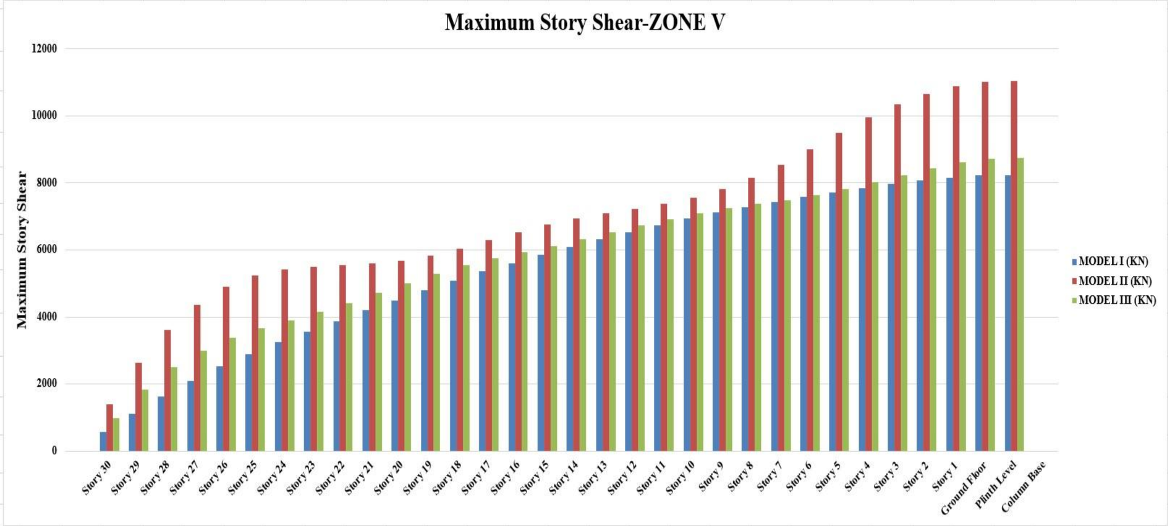


Fig 39.Comparison of Maximum Story Shear (ZONE V)

The maximum story shear for Model I, Model II, and Model III in both zones taken into consideration occurred at the base of the structure. According to the results of the response spectrum study, In Zone IV and Zone V respectively the maximum story shear values for the Model I are 5484.2672 KN and 8226.4009 KN. The maximum story shear values for the Model II are 7354.6113 KN in Zone IV and 11031.9169 KN in Zone V respectively. In Zone IV and Zone V the maximum story shear values for the Model III are 5820.7661 KN and 8731.1491 KN respectively. When we evaluate the three Models individually, Zone V has substantially higher story shear values than Zone IV.

Model II in Zone V has a maximum story shear of 11031.9169 KN which is higher than Model I in Zone V which has a maximum story shear of 7354.6113 KN. Model III is also superior to Model I, because the story shear in Model III and Model I considering Zone V is 8731.1491 KN and 7354.6113 KN, respectively which we can see in fig 32 - 39.

From the aforementioned findings, it can be concluded that shear wall structures, when compared to all other structures, have a bigger influence on retrofitting procedures, and that bracing structures, when compared to typical RC framed buildings, have a greater impact on structures.Now doing the analysis for the nonlinear static pushover analysis using a displacement control method in terms of goal displacement, performance point, and base shear.These are defined as:

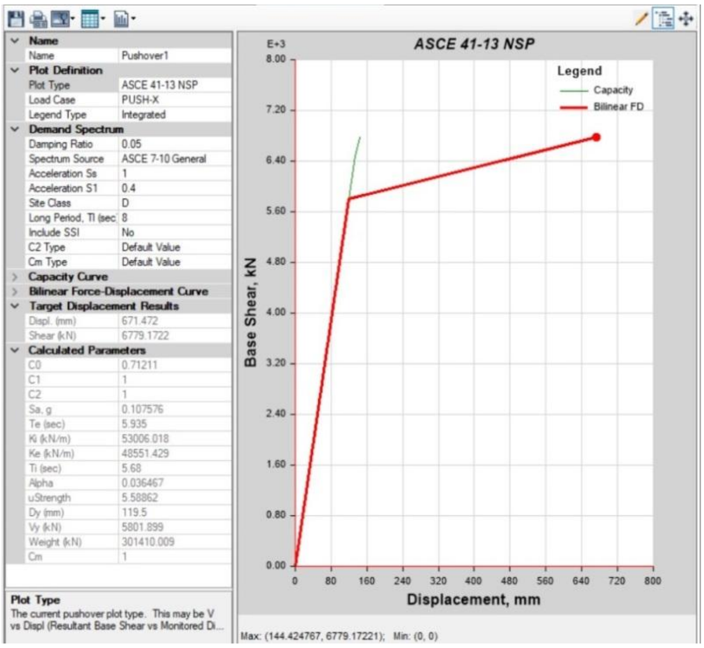
a) **Target Displacement:**Target displacement refers to the greatest drift that a structure may experience under earthquake stresses without completely collapsing.

b) **Performance point:**For a certain damping ratio, the Performance Point—which denotes the condition of the structure's maximum inelastic capacity—can be discovered by finding the intersection of the Capacity Spectrum and Demand Spectrum.

c) **Base shear:**Base shear is an estimate of the greatest anticipated lateral force caused by seismic activity on the base of the building.

5.2 RESULTS FROM PUSHOVER ANALYSIS

5.2.1. MODEL I: (CONVENTIONAL RC STRUCTURE)

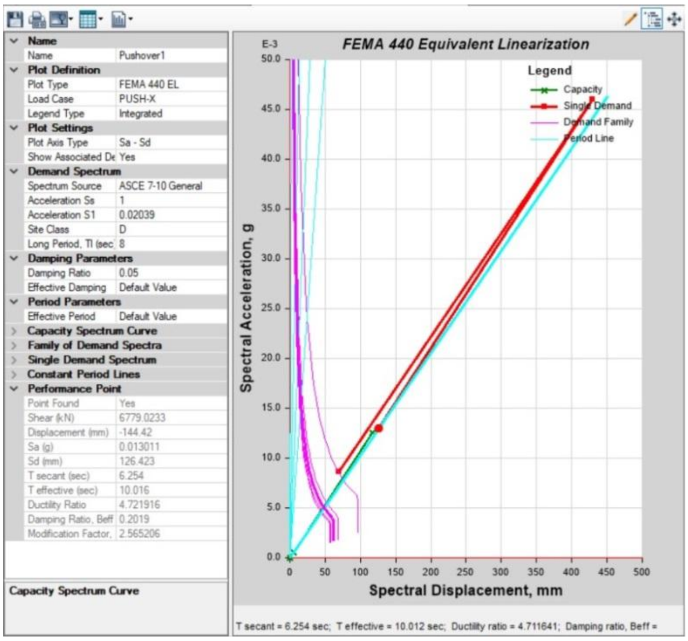


Name	
Name	Pushover1
Plot Definition	
Plot Type	ASCE 41-13 NSP
Load Case	PUSH-X
Legend Type	Integrated
Demand Spectrum	
Damping Ratio	0.05
Spectrum Source	ASCE 7-10 General
Acceleration Ss	1
Acceleration S1	0.4
Site Class	D
Long Period, Tl (sec	8
Include SSI	No
C2 Type	Default Value
Cm Type	Default Value
Capacity Curve	
Bilinear Force-Displacement Curve	
Target Displacement Results	
Displ. (mm)	671.472
Shear (kN)	6779.1722
Calculated Parameters	
C0	0.71211
C1	1
C2	1
Sa, g	0.107576
Te (sec)	5.935
Ki (kN/m)	53006.018
Ke (kN/m)	48551.429
Ti (sec)	5.68
Alpha	0.036467
uStrength	5.58862
Dy (mm)	119.5
Vy (kN)	5801.899
Weight (kN)	301410.009
Cm	1

Plot Type
The current pushover plot type. This may be V vs Displ (Resultant Base Shear vs Monitored Di...

Max: (144.424767, 6779.17221); Min: (0, 0)

Fig 40.Target Displacement Point Results from ASCE 41-13 NSP



Name	
Name	Pushover1
Plot Definition	
Plot Type	FEMA 440 EL
Load Case	PUSH-X
Legend Type	Integrated
Plot Settings	
Plot Axis Type	Sa - Sd
Show Associated De	Yes
Demand Spectrum	
Spectrum Source	ASCE 7-10 General
Acceleration Ss	1
Acceleration S1	0.02039
Site Class	D
Long Period, Tl (sec	8
Damping Parameters	
Damping Ratio	0.05
Effective Damping	Default Value
Period Parameters	
Effective Period	Default Value
Capacity Spectrum Curve	
Family of Demand Spectra	
Single Demand Spectrum	
Constant Period Lines	
Performance Point	
Point Found	Yes
Shear (kN)	6779.0233
Displacement (mm)	-144.42
Sa (g)	0.013011
Sd (mm)	126.423
T secant (sec)	6.254
T effective (sec)	10.016
Ductility Ratio	4.721916
Damping Ratio, Beff	0.2019
Modification Factor,	2.565206

Capacity Spectrum Curve

T secant = 6.254 sec; T effective = 10.012 sec; Ductility ratio = 4.711641; Damping ratio, Beff =

Fig 41.Performance Point Results from FEMA 440 EL

Table 4. Target displacement and performance point for Model I

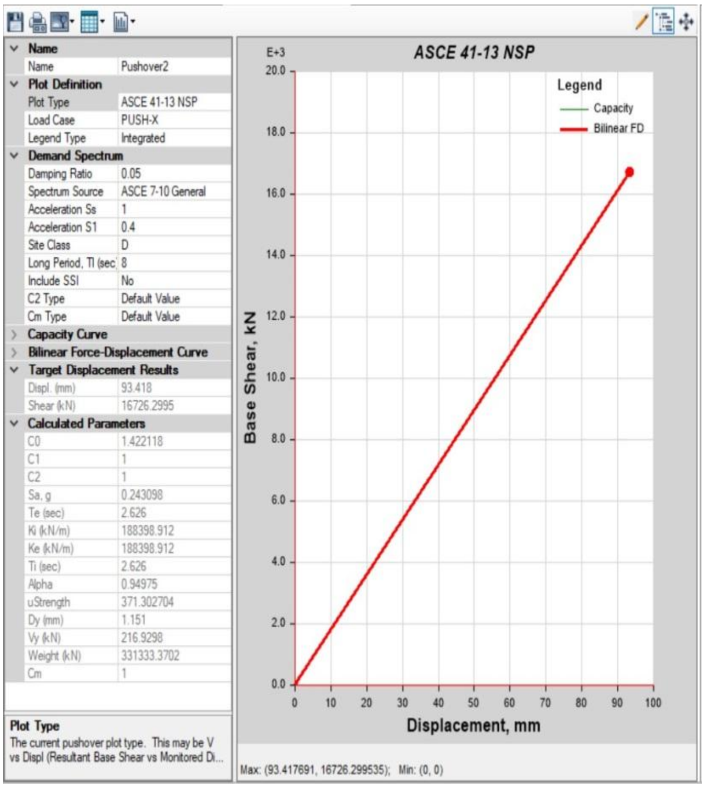
	Displacement (mm)	**Shear (KN)**
Target displacement Point	671.472	6779.1722
Performance Point	144.42	6779.0233

HINGE RESULTS

Table 5.Hinge results for Model I

Step	**Monitored Displacement**	**Base Force**	**A-B**	**B-C**	**C-D**	**D-E**	**>E**	**A-IO**	**IO-LS**	**LS-CP**	**>CP**	**Total Hinges**
	mm	**kN**										
0	0	0	8072	376	0	0	0	8448	0	0	0	8448
1	5.589	296.2622	8048	400	0	0	0	8448	0	0	0	8448
2	134.628	6486.8779	6184	2264	0	0	0	8440	8	0	0	8448
3	144.425	6779.1722	6024	2424	0	0	0	8416	32	0	0	8448

The target displacement point for RC conventional structure is achieved at 671.472 mm, while the performance point is obtained as 144.42 mm which is a nonlinear plastic limit when subjected to the seismic effects. The hinge results show that at the performance point 32 hinges are forming in the immediate occupancy and life safety state.



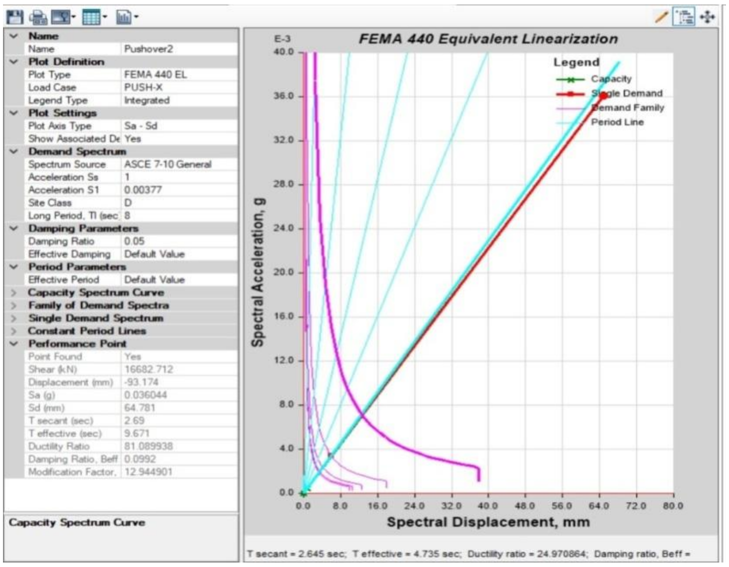
Name	
Name	Pushover2
Plot Definition	
Plot Type	ASCE 41-13 NSP
Load Case	PUSH-X
Legend Type	Integrated
Demand Spectrum	
Damping Ratio	0.05
Spectrum Source	ASCE 7-10 General
Acceleration Ss	1
Acceleration S1	0.4
Site Class	D
Long Period, Tl (sec	8
Include SSI	No
C2 Type	Default Value
Cm Type	Default Value
Capacity Curve	
Bilinear Force-Displacement Curve	
Target Displacement Results	
Displ. (mm)	93.418
Shear (kN)	16726.2995
Calculated Parameters	
C0	1.422118
C1	1
C2	1
Sa, g	0.243098
Te (sec)	2.626
Ki (kN/m)	188398.912
Ke (kN/m)	188398.912
Ti (sec)	2.626
Alpha	0.94975
uStrength	371.302704
Dy (mm)	1.151
Vy (kN)	216.9298
Weight (kN)	331333.3702
Cm	1

Plot Type
The current pushover plot type. This may be V vs Displ (Resultant Base Shear vs Monitored Di...

Max: (93.417691, 16726.299535); Min: (0, 0)

Fig 42. Target Displacement Point Results from ASCE 41-13 NSP

5.2.2. MODEL II: (SHEAR WALL STRUCTURE)



Name	
Name	Pushover2
Plot Definition	
Plot Type	FEMA 440 EL
Load Case	PUSH-X
Legend Type	Integrated
Plot Settings	
Plot Axis Type	Sa - Sd
Show Associated De	Yes
Demand Spectrum	
Spectrum Source	ASCE 7-10 General
Acceleration Ss	1
Acceleration S1	0.00377
Site Class	D
Long Period, Tl (sec	8
Damping Parameters	
Damping Ratio	0.05
Effective Damping	Default Value
Period Parameters	
Effective Period	Default Value
Capacity Spectrum Curve	
Family of Demand Spectra	
Single Demand Spectrum	
Constant Period Lines	
Performance Point	
Point Found	Yes
Shear (kN)	16682.712
Displacement (mm)	-93.174
Sa (g)	0.036044
Sd (mm)	64.781
T secant (sec)	2.69
T effective (sec)	9.671
Ductility Ratio	81.089938
Damping Ratio, Beff	0.0992
Modification Factor,	12.944901

Capacity Spectrum Curve

T secant = 2.645 sec; T effective = 4.735 sec; Ductility ratio = 24.970864; Damping ratio, Beff =

Fig 43.Performance Point Results from FEMA 440 EL

Table 6. Target displacement and performance point for Model II	Displacement (mm)	Shear (KN)
Target displacement Point	93.418	16726.2995
Performance Point	93.174	16682.712

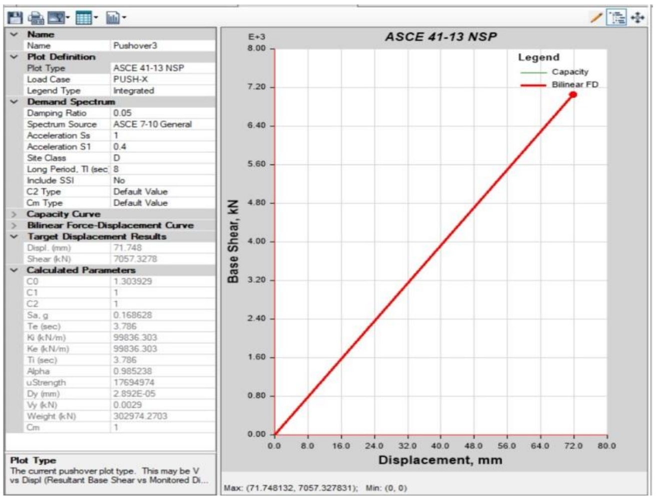
HINGE RESULTS

Table 7.Hinge results for Model II

Step	Monitored Displacement	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
	mm	kN										
0	0	0	7568	880	0	0	0	8448	0	0	0	8448
1	1.151	216.9298	7560	888	0	0	0	8448	0	0	0	8448
2	93.418	16726.2995	6492	1956	0	0	0	8448	0	0	0	8448

The target displacement point for RC conventional structure is achieved at 93.418 mm, while the performance point is obtained as 93.174 mm which is a nonlinear plastic limit when subjected to the seismic effects. The hinge results show that at the performance point all hinges are forming within the immediate occupancy state.

5.2.3. MODEL III: (BRACINGS STRUCTURE)

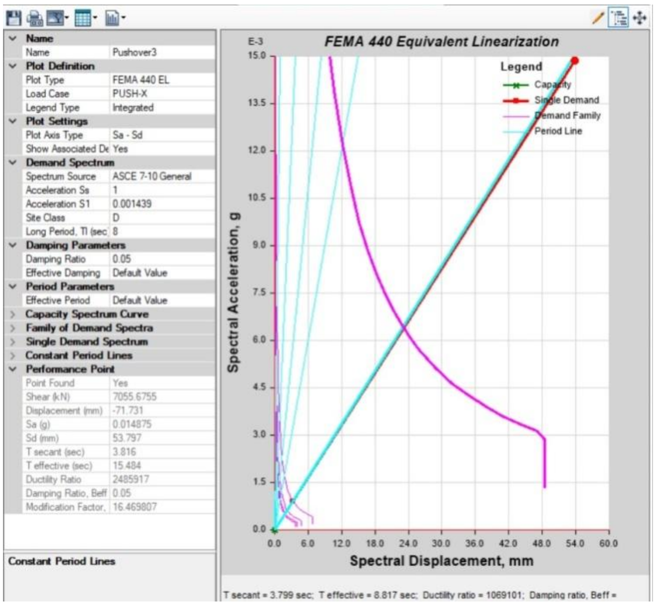


Name	
Name	Pushover3
Plot Definition	
Plot Type	ASCE 41-13 NSP
Load Case	PUSH-X
Legend Type	Integrated
Demand Spectrum	
Damping Ratio	0.05
Spectrum Source	ASCE 7-10 General
Acceleration Ss	1
Acceleration S1	0.4
Site Class	D
Long Period, Tl (sec	8
Include SSI	No
C2 Type	Default Value
Cm Type	Default Value
Capacity Curve	
Bilinear Force-Displacement Curve	
Target Displacement Results	
Displ. (mm)	71.748
Shear (kN)	7057.3278
Calculated Parameters	
C0	1.303929
C1	1
C2	1
Sa, g	0.168628
Te (sec)	3.786
Ki (kN/m)	99836.303
Ke (kN/m)	99836.303
Ti (sec)	3.786
Alpha	0.985238
uStrength	17694974
Dy (mm)	2.892E-05
Vy (kN)	0.0029
Weight (kN)	302974.2703
Cm	1

Plot Type
The current pushover plot type. This may be V vs Displ (Resultant Base Shear vs Monitored Di...

Max: (71.748132, 7057.327831); Min: (0, 0)

Fig 44.Target Displacement Point Results from ASCE 41-13 NSP



Name	
Name	Pushover3
Plot Definition	
Plot Type	FEMA 440 EL
Load Case	PUSH-X
Legend Type	Integrated
Plot Settings	
Plot Axis Type	Sa - Sd
Show Associated De	Yes
Demand Spectrum	
Spectrum Source	ASCE 7-10 General
Acceleration Ss	1
Acceleration S1	0.001439
Site Class	D
Long Period, Tl (sec	8
Damping Parameters	
Damping Ratio	0.05
Effective Damping	Default Value
Period Parameters	
Effective Period	Default Value
Capacity Spectrum Curve	
Family of Demand Spectra	
Single Demand Spectrum	
Constant Period Lines	
Performance Point	
Point Found	Yes
Shear (kN)	7055.6755
Displacement (mm)	-71.731
Sa (g)	0.014875
Sd (mm)	53.797
T secant (sec)	3.816
T effective (sec)	15.484
Ductility Ratio	2485917
Damping Ratio, Beff	0.05
Modification Factor,	16.469807

Constant Period Lines

T secant = 3.799 sec; T effective = 8.817 sec; Ductility ratio = 1069101; Damping ratio, Beff =

Fig 45.Performance Point Results from FEMA 440 EL

Table 8. Target displacement and performance point for Model III

	Displacement (mm)	Shear (KN)
Target displacement Point	71.748	7057.3278
Performance Point	71.731	7055.6755

HINGE RESULTS

Table 9.Hinge results for Model III

Step	Monitored Displacement	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
	mm	kN										
0	0	0	8056	392	0	0	0	8448	0	0	0	8448
1	2.892E-05	0.0029	8056	392	0	0	0	8448	0	0	0	8448

Step	Monitored Displacement	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
	mm	kN										
2	71.748	7057.3278	7465	983	0	0	0	8448	0	0	0	8448

The target displacement point for RC conventional structure is achieved at 71.748 mm, while the performance point is obtained as 71.731 mm which is a nonlinear plastic limit when subjected to the seismic effects. The hinge results show that at the performance point all hinges are forming within the immediate occupancy state.

5.3. Comparision From Pushover Analysis

Table 10.Comparison for Target displacement and performance point

MODEL	Target Displacement (mm)	Performance Point (mm)
MODEL I	671.472	144.42
MODEL II	93.418	93.174
MODEL III	71.748	71.731

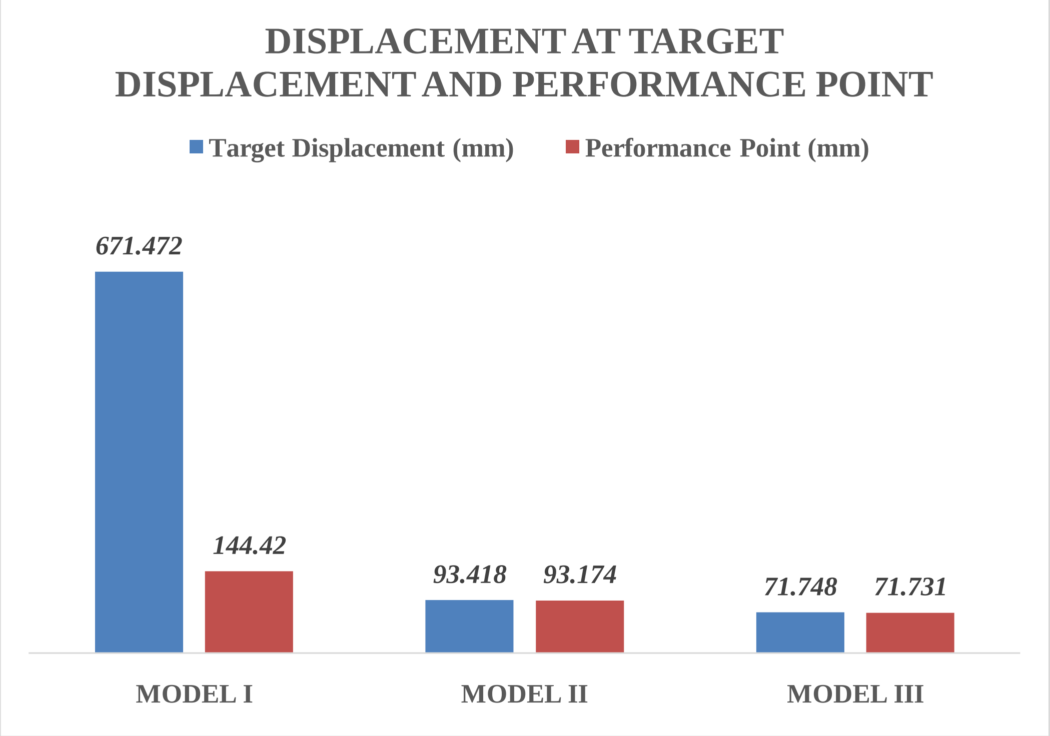


Fig 46.Comparison for Target displacement and performance point

Table 11.Comparison for Base shear at Target displacement and performance points

MODEL	Shear at Target Displacement Point (KN)	Shear at Performance Point (KN)
MODEL I	6779.1722	6779.0233
MODEL II	16726.2995	16682.712
MODEL III	7057.3278	7055.6755

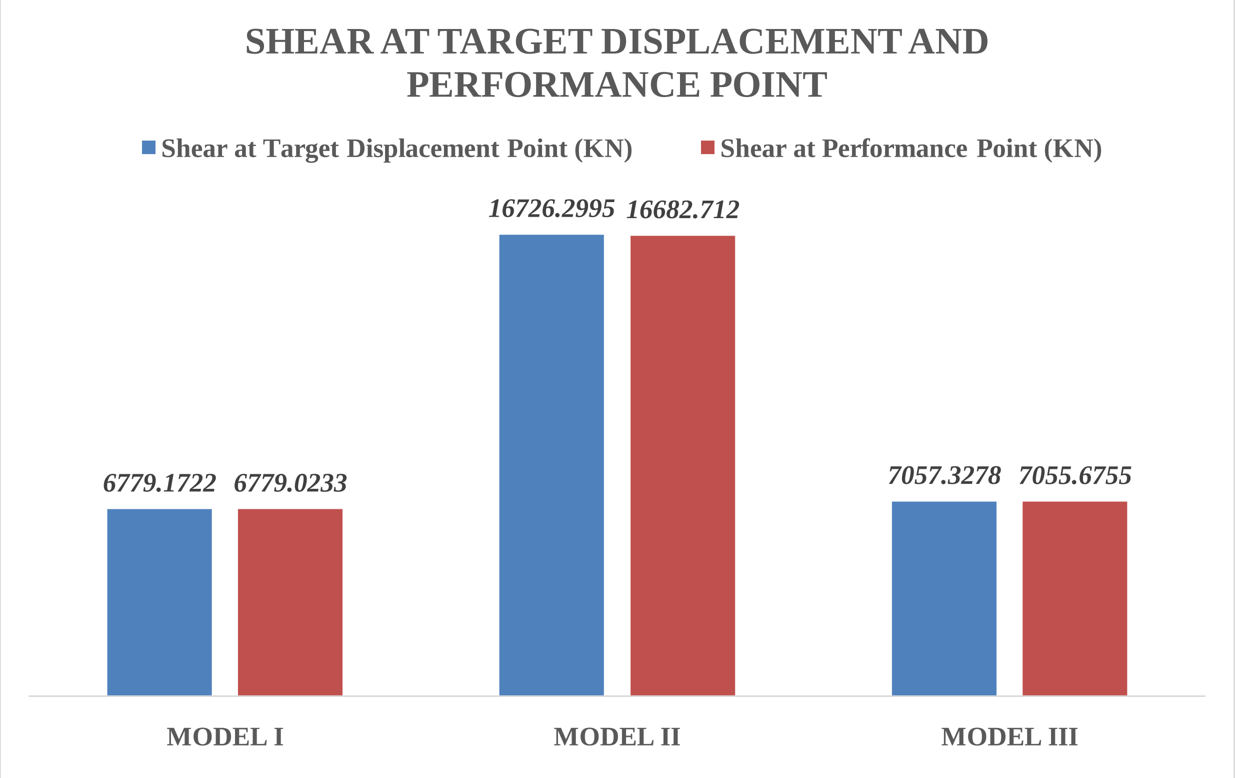


Fig 47.Comparison for Base shear at Target displacement and performance points

Duetotheseismiceffectsinthe Model I, II and III, Model II havethemaximumshearoccursatbaseofthestructure,maximumstorydisplacementoccurredatthetopstorywhichisstory30andthemaximumdisplacementofthestructureisfoundout.

In Y direction also the push over curve of both the models are almost coinciding. Pushover Curvesobtained from this study show that there is no significant difference in the response of thebuilding forthe structure.

The performance point and target displacement results also follow the same phenomenon as the maximum story displacement. Model II has the lower displacement results than all the Models

FromtheabovefiguresModelIIhavethecompatibly lesslateral displacement and performance pointswhenperformingnonlinearstaticpushoveranalysis.

VI. Conclusions

1. The building is more resistant to seismic acceleration due to the shear wall construction. When a structure is modeled, the results of the modal analysis reveal certain peculiar modes. However, it is discovered that such forms get very little mass engagement. As a result, these modes won't materially alter the building's reaction.
2. Shear wall structures, out of the three techniques used, are producing the best findings, leading to the conclusion that shear wall structures are the best way for response spectrum analysis.
3. This research suggests that, compared to shear wall structures, bracing structures may not substantially alter the seismic behavior of framed buildings in analysis.
4. The Pushover Curves obtained from this study show that there is not much significant variation in the response of the friction damper structure, but it is effective to use the bracing structure instead of conventional structure because the performance point is achieved at 71.748 mm and the results from response spectrum analysis are significantly better than those from conventional structure.
5. When compared to the conventional structure in Zones IV and V, the influence of these

structural methods on the seismic response of multi-story buildings significantly reduced the Maximum story displacement, story drift, and base shear, attracting less seismic forces.

6. The installation of bracings also changes how the structures respond to earthquakes. For all the parameters, models with a bracing system showed satisfactory performance, similar to a shear wall construction.
7. For models II and III, base shear has risen while story displacement and tale drifts have decreased.
8. The construction is thus only secure when utilized with a shear wall, and further research has to be done on it using various difficulties.

Vii.References

[1]. Rosinblueth and Holtz “Analysis of shear walls in tall buildings” (1960).
[2]. Mo and Jost A report of seismic response of multistory reinforced concrete framed shear walls using a nonlinear model.(1993)
[3]. SatishAnnigiri research scholar and Ashok K. Jain. "Torsional provisions for asymmetrical multistory buildings in IS: 1893" (1994).
[4]. Clough.R, King I.P and Wilson E.I- “Structural analysis of multi storied buildings” (1964).
[5]. Khan, F.R. and S. Brounis, J.A,„Introduction of shear wall with frames in concrete Sabrcounis structure under lateral loads (1964).
[6]. Girijavallabhan, C. V. Analysis of Shear Walls with Openings. Journal of the Structural Division. 1969. 95(10): 2093- 2104.
[7]. Paulay. T, and Priestley, "Seismic design of reinforced concrete and masonry buildings" (1992).Y.L.Mo and C. J. Kuo. 1998.
[8]. Structural behavior of reinforced concrete frame-wall components, department of civil engineering, national Cheng kung University, Tainan, 701, Taiwan.
[9]. JJ-Humar and S. Yavari "design of concrete shear wall buildings for earthquake induced torsion" (2002).
[10]. Tolga Aki. S “LATERAL LOAD ANALYSIS OF SHEAR WALL-FRAME STRUCTURES” (2004).
[11]. Gary R. Searer and Sigmund A. Freeman ,"design drift requirements for long-period structures”(2004).13th World Conference on Earthquake Engineering Vancouver, B.C., Canada August 1-6, 2004 Paper No. 3292
[12]. DhimanBasu and Sudhir K. Jai “Alternative method to locate Centre of rigidity in asymmetric buildings” (2006).
[13]. Thomas, N. and Salonikios., “Analytical Prediction of the Inelastic Response of RC Walls with Low Aspect Ratio” (2007).
[14]. Characteristics & Applications of Different Types of Dampers as Seismic Energy Dissipater, Dharmesh Chandnani[1], Riddhi Joshi[2], Kumarpal Trivedi[3], International Journal of Computer Science and Network, Volume 5, Issue 2, April 2016, ISSN (Online) : 2277-5420.
[15]. ATC4 (AppliedTechnologyCouncil)“SeismicEvaluationandRetrofitofConcreteBuildings”Volume 1,Nov-1996.
[16]. FEMA273 (FederalEmergencyManagementAgency)“GuidelinesfortheSeismicRehabilitationofBuildings”Oct-1997.
[17]. FEMA356“Pre-standardandCommentaryfortheSeismicRehabilitationofBuildings”Nov-2000.
[18]. FEMA440“ImprovementofNonlinearStaticSeismicAnalysisProcedures”June-2005.
[19]. IS456:2000“PlainandReinforcedConcreteCodeofPractice(FourthRevision)”
[20]. IS1893:2002“CriteriaforEarthquakeResistantDesignofStructures(PartI)”.