COMPARISON OF CODAL RECOMMENDATIONS FOR DESIGN OF UNSYMMETRICAL BUILDINGS

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Abstract: The research on asymmetric buildings has been extensive primarily focusing on the sta-bility of a structure when subjected to earthquake. Based on them numerous guidelines have been laid out to ensure safety. I have in this paper tried to evaluate the ef-fectiveness of the guidelines provided in the IS: 1893 (2000). Asymmetric buildings are more common now than they have ever been and their popularity has been growing primarily due to the functionality they provide. Due to the frequent earthquakes that India suffers being at the junction of two tectonic plates it has become increasingly important to study Indian buildings for seismic safety. The buildings are analyzed based on the effect of torsion which is the main cause of damage for Asymmetric Buildings.

I. INTRODUCTION

Structures have been prone to earthquakes since the first structure was built. Earlier accredited to the wrath of gods there have been many elaborate rituals in civilizations around the globe to keep the Gods appeased and cities safe which then evolved into festivals but we now know otherwise. Earthquakes which are some of the most severe natural catastrophes known to man are still a modern menace and though we don't pray our way for safety anymore Earthquake resistance of buildings has taken a more scientific turn and still is a major area of research. Though one of the most catastrophic events in nature earthquakes themselves do not kill people although they may result in some of the highest death toll known. The primary damage caused by an earth quake is to a building or a natural structure and not people. The collapses of such man-made structures like buildings lead to people using them getting crushed or trapped by the debris. The higher the rise the greater is the fall, due to its unique nature earthquakes are more menacing to the more developed urban areas than rural areas as these tend to be more dense populated with more high-rise buildings in a concentrated space for utilizing the expensive commodity effectively. Rapid urbanization has propelled the pri-ority of Earthquake resistance.

The limitation of space in urban cities has caused many new changes in the struc-ture of buildings. The apartment complexes used to be a collection of apartments from the ground up while the limitation of parking spaces in the current decade has led to the transformation of the lower floors into parking spaces for the residents. The design though provides utility but also makes the building asymmetric. Seismic damage sur-veys and analyses conducted after the earthquakes have shown that the modes of failure of the structures . It is apparent that the most vulnerable structures are those, which are asymmetric in nature. Hence the seismic behavior of an asymmetric structure has become important.

II. AREA OF STUDY

This study proposes to analyze the relative effectiveness of the critical torsional provi-sions as supplied by the IS 1893:2002 (Part 1). The study tries to analyze the use of the provision and their effectiveness by designing a structure without considering the torsional provisions and then comparing its ability to resist the effect of earthquake forces in comparison to a structure designed in accordance to the necessary torsional provisions.

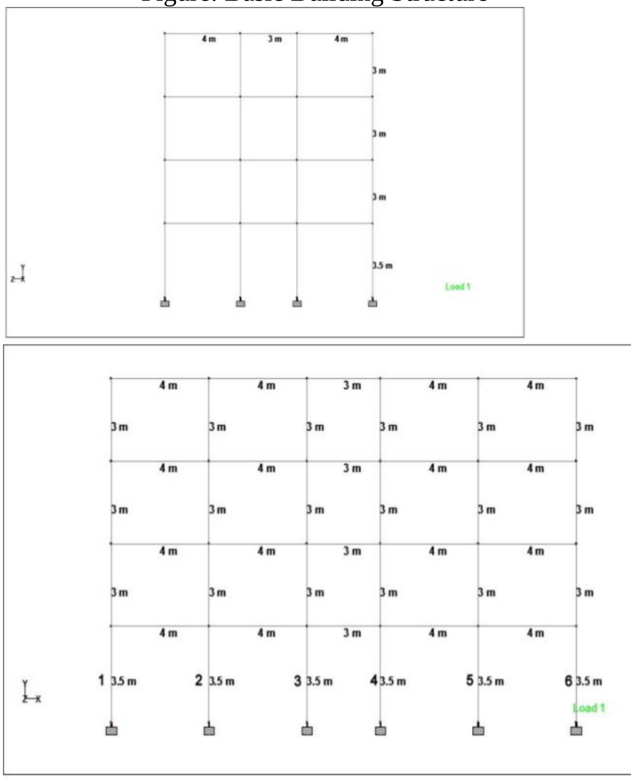
Scope of Study

The effects of the torsional provisions are studied on 4 and 10 story RC frame residential building model 11m in width and 19m in length. The bottom story is about 3.5m in height and the rest are all 3m in height. The effect of the stiffness of the slabs are molded using diaphragm constraints. The building is considered to be symmetric with respect to the stiffness distribution. Only Mass Eccentricity was considered for this problem. The Mass Eccentricity was also assumed to be unidirectional along the length of the structure. The Loading is also taken to be unidirectional. The supports of the structure are assumed to be fixed. The P-4 effect on the structure is not considered in the scope of the study.

III. METHODOLOGY

The structure was modeled in SAP 2000 for the purpose of analysis the building design and other analysis were also conducted with Etabs. The structures are two models on of 4 stories of 12.5m in height and other is of 10 stories with 30.5m in height structure with 4 bays in the X direction of spans lengths of 4m at the 2 spans at the periphery and the central span is about 3m in length. The structure has 3 spans in the Y direction with the 2 spans at the periphery being 4m each and the central span is about 3m in length. The material assumed is Concrete of grade M20 and the Steel used is Fe 415. The Beams are considered to have a cross-section size of about 300x600m and the columns are made of the same cross section sizes with the longer side along the longer span. The Structure is loaded with a live load of about 3KN/m2 as per the live load requirements from IS 845 Part II assuming the structure to be a residential building. The load was applied to the center of mass at the first try for symmetric building. The center of mass (CM) was then applied at a point 1.9m away from the Centroid of the structure. The design of the structure was designed in Etabs as per IS:456. The designed reinforcements were then taken imported into the SAP 2000 software and Pushover analysis was conducted on the structure. The Hinge used in the model was based on FEMA 356 for the respective columns and beams. The Degrees of Freedom for the Beams was M3 and for the Columns was P-M2-M3. The Pushover analysis is then conducted and the occurrence of hinges is observed. Two Load Cases were constructed to conduct the analysis in both directions the force is applied as an acceleration.

Figure: Basic Building Structure



IV. CODAL PROVISIONS

The basic approach of design codes is application of linear static or dynamic load meth-ods for design based on Earthquake Loading. Some of the codal provisions are studied in the following.

As per [IS 1893 (Part 1), 2002] the Static Eccentricity (e) is defined in the design codes as the distance between the Center of Mass (CM) and Center of Rigidity (CR) of the structure. The Center of Rigidity is defined as “the point through which the resultant of the restoring forces of a system acts.". The Center of Mass is defined as “the point through which the resultant of the masses of a system acts. This point corresponds to the center of gravity of masses of system."

The Design Eccentricities (e_{di},e_{si}) are obtained based on the values of the static ec-centricity after accounting for the dynamic amplification of torsion and allowance for accidental torsion induced by rotational component of ground motion. Most design eccentricities are based on the formula

$$e_{di} = e + b$$

$$e_{si} = e - b$$

Table : Values in different codes

IS 456	IBC 2003	NZ 4203:1992	NBCC 1995
1.5	1	1	1.5
0.05	$0.05A_x$	0.1	$0.1A_x$
1	1	1	0.05

INDIAN STANDARD 1893: 2002

The IS 1893: 2002 assumes the inertial force caused by the Earthquake to act at the Center of Mass (CM) of the structure. The Static Eccentricity (e) is the distance between the Center of Mass (CM) and Center of Rigidity (CR) of the structure. The Design Eccentricity is obtained by using the formula for $e_d = 1.5e_s + 0.05b$. The code has been modified to correctly include the stiffness of the in ll walls in calculation of the Time Period (T) of the structure. Neglecting the stiffness of the in ll wall causesthe calculated period to be higher leading a reduced calculated Earthquake Load. Thecode has been revised to calculate the Time Period (T) of the building as $T = 0.09h/(d)^{0.5}$ instead of the old code.

CANADIAN CODE NBCC 1995

The Canadian code also follows a similar eccentricity pattern with the values of 1.5, $0.1A_x$ and 0.5. The NBCC [1995] recommends using a 3-D dynamic analysis to evaluate the effect of torsion, the accidental torsion is accounted for by applying a torque equal to floor force times 0.1 b at each floor which are then subtracted from the results obtained from the 3-D analysis to calculate the maximum design force.

Summary

All codes examined use the concept of minimum eccentricity to be assumed during design calculation for safety. The value of the dynamic eccentricity is also generally calculated based on the same formula involving the static eccentricity the width of the structure based on the direction of the eccentricity in question. The basis of difference among the codes is primarily on the values of the coefficients used in the formula while some codes prescribe a direct formula for calculation others codes prescribe a particular constant value.

Modeling and Analysis

Building Geometry

The plan of the building is taken from The building plan is symmetric. The columns are along the aligned to the face of the building as shown in Figure 5.1. The model is based on the plan geometry in [Kilar, 2001]

Material Properties

The material to be modeled is assumed to be M20 concrete with the reinforcements to be of Fe415 Steel. The Material properties were modeled after the provision in IS 456: 2000.

Modeling

The model was first analyzed through Etabs to check the design calculations. The mode was then again redesigned in SAP 2000 for analysis. The model was divided into sections. The model consists of frame sections B1 for Beams and C1,C2 for Columns respectively. The beam section has a dimension of 300 mm in width and 600 mm in depth of M20 concrete. The re-bars were modeled in HYSD415 bars for the longitudinal reinforcement and Mild steel Fe 250 bars for the confinement reinforcements. The beams were not divided as the changes in the beam reinforcement was not the priority. The Model was then analyzed in a pushover analysis and Time History Analysis.

Reinforcement provided in models

The reinforcements are compared between the 3 models which are based on the same building model and are identical in all respects except the application and position of the Lateral Earthquake force applied. The reinforcements in the columns are of special interest.

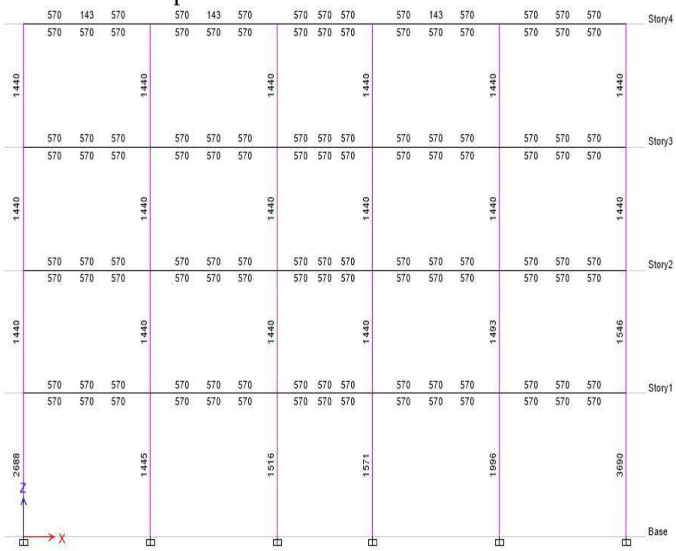


Figure : Reinforcements required for 12.5 m Model, considering Mass Eccentricity (Outer Face)

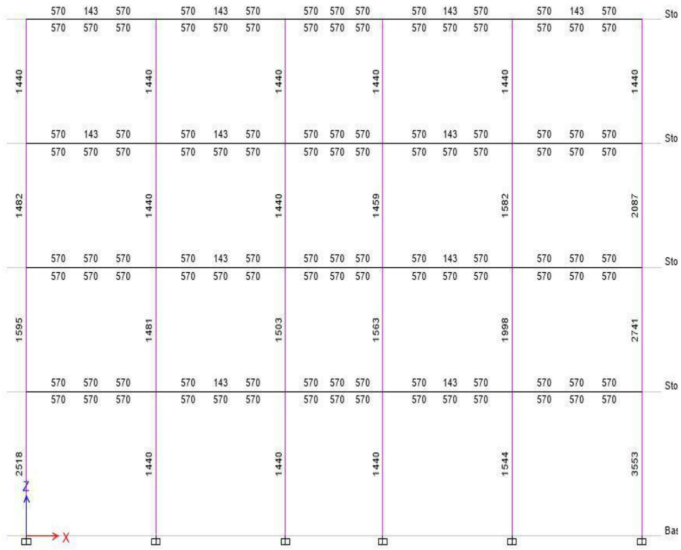


Figure : Reinforcements required for 12.5 m Model, considering Mass Eccentricity (Inner Face)

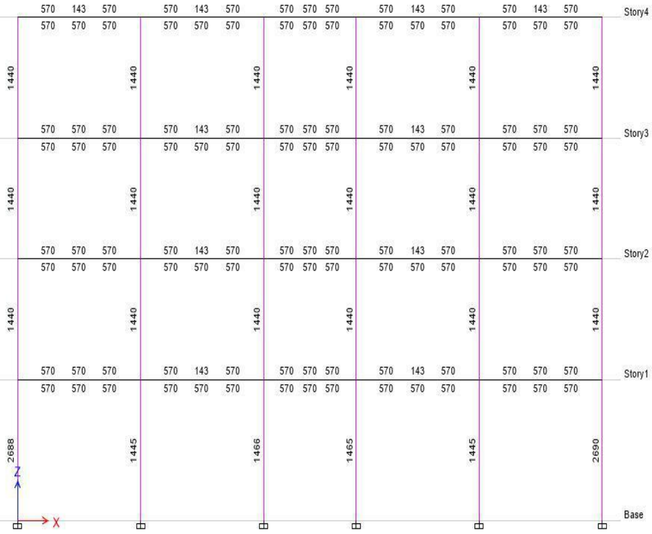


Figure: Reinforcement required for 12.5m model without considering Mass Eccentricity (Outer Face)

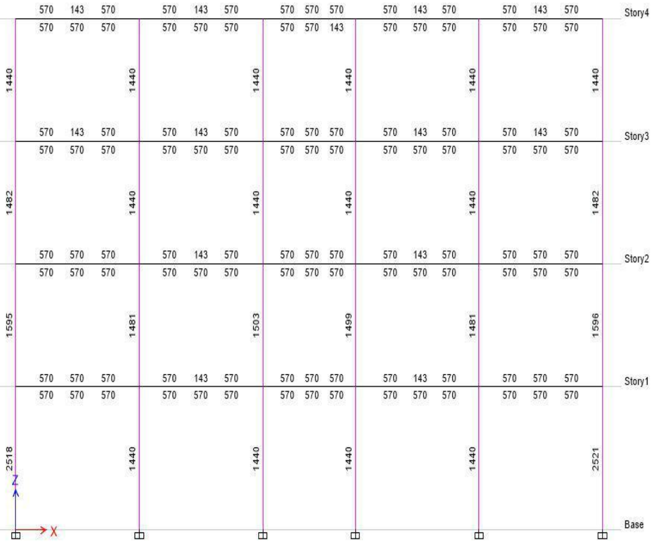


Figure: Reinforcement required for 12.5m model without considering Mass Eccentricity(Inner Face)

The minimum value of the reinforcement for a column section is 0.8% of the gross area for a compression member which in this case amounts to 1400 mm^2. Most of the section in involving the control section still retains the minimum reinforcement as in Table 7.2. After applying the Earthquake Load the reinforcements at the base change to a higher value although they remain symmetric as shown in Table 7.2. The application of an mass eccentricity causes the columns to have an eccentric reinforcement with the columns at the far end having lower reinforcements than the near end of the structure in the direction of the eccentricity . The beam reinforcements remain almost constant irrespective of the application of the lateral forces.

Now considering the change in reinforcements in all the respective models. For the purpose of reference lets us number the columns from right to left as 1 to 6 as in the Figure 5.1 . The Figures 7.1,7.2,7.3,7.4 7.5,7.6,7.7 and 7.8, show the reinforcement area.

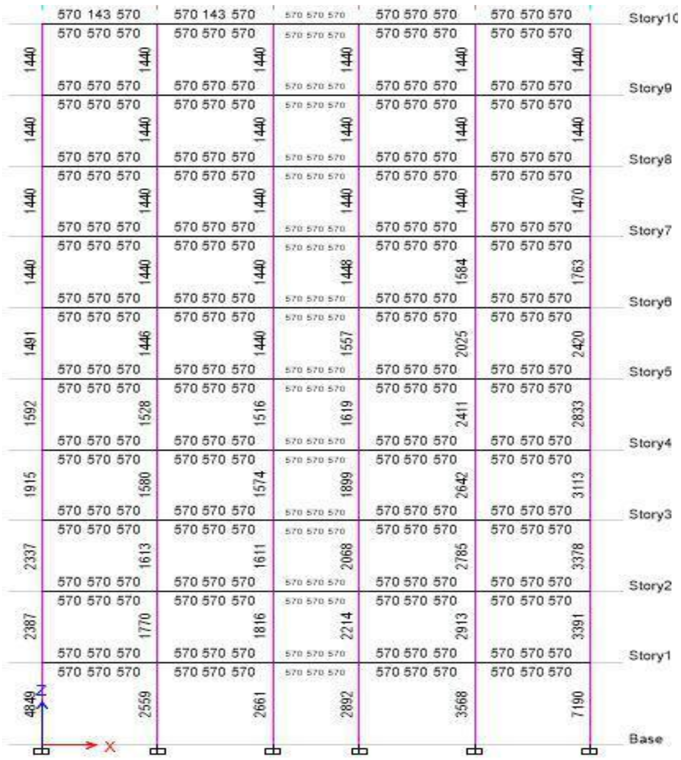


Figure: Reinforcement required for 30.5m model after considering mass eccentricity (Outer face)

Table 7.1: Reinforcement Comparison Table for 12.5m model

Columns		Reinforcements		
	Y	Control	As1	As2
1	-6	1440	2930	2930
	-2	1440	2808	2808
	1	1440	2808	2808
	5	1440	2930	2930
2	-6	1440	1520	1520
	-2	1440	1440	1440
	1	1440	1440	1440
	5	1440	1620	1520
3	-6	1440	1494	1507
	-2	1440	1440	1440
	1	1440	1440	1440
	5	1440	1494	1507
4	-6	1440	1494	1549
	-2	1440	1440	1440
	1	1440	1440	1440
	5	1440	1494	1549
5	-6	1440	1521	1688
	-2	1440	1440	1488
	1	1440	1440	1488
	5	1440	1521	1688
6	-6	1440	2931	3280
	-2	1440	2801	3224
	1	1440	2801	3224
	5	1440	2931	3280

required for the particular section based on the design loads. The section shown is the base of the buildings so as to show the maximum change in reinforcement for the models based on the loads. The section is observed in the XZ plane as this is the plane with the maximum of columns visible at any point and the Y coordinate is varied, the origin is considered near the middle of the building span.

When only the dead and live loads are applied the models tend to have the same re-inforcements at the columns which is the minimum reinforcement which is 0.8% of the Gross area of the column. In this case it amounts to $1440mm^2$.

In case of the 12.5m model the basic reinforcement requirement is the same for the control structure of the minimum reinforcement $1440mm^2$. But when the earthquake force is induced the reinforcements on the $2690mm^2$ in the outer base columns. The inner base column have a slightly smaller reinforcement of $1445mm^2$ to $1466mm^2$ as in Table 7.1, The change in reinforcement after using the code is from $1440mm^2$ to $2690mm^2$ which is about $1250mm^2$ which is an increment of about 46.46%. While the inner column the reinforcement remains close the minimum reinforcement value even after applying the Earthquake force. The change is reinforcement of the column is $1468mm^2$ from $1440mm^2$ with a difference of about $28mm^2$ and increment of 1.9%. The innermost columns have a reinforcement at the outer face of $2518mm^2$ with a increase of $1078mm^2$ increment in 42.81%. At the same time the inner most columns dont show any change from the minimum value. In case of the asymmetric structure. The eccentricity was induced in the Y direction such that the point of application of the load is closer to the 4th 5th and 6th column and far from the 1st 2nd and 3rd column. The outer columns on the side away from the point of application did not show any major change the value of reinforcement is almost the same. On the 6th which is situated near the point of application of the force the change in column reinforcement was from $2690mm^2$ to $3690mm^2$ with a change of $1000mm^2$ and an increment in

reinforcement of 27.1%. The outermost column in the inner face also show a change from 2581mm^2 to 3553mm^2 which is an increase of 972mm^2 or an increment of 27.35%.
The 30.5m model shows more change in reinforcements than in 12.5m model. The reinforcements at the base are compared in Table 7.2. From Figure 7.5 the change in reinforcement for the 1st column is from 4807mm^2 to 7391mm^2 which is a change of 2584mm^2 and an increment of 34.96%. The inner face columns at the periphery also experience a change in reinforcement from 4177mm^2 to 7204mm^2 with an increase of 3027mm^2 and increment of 42% as shown in Figure 7.6 . The change in reinforcement of the inner columns is small from Figure 7.5 the change in inner columns ranges from 2721mm^2 to 2952mm^2 with an increase of 230mm^2 which is an increment of 7.8%. The innermost core columns almost remain the same even after considering mass eccentricity from 1500mm^2 to 1612mm^2 with and increase of 122mm^2 which is an increment of 6.9%.

Table 7.3: Reinforcement Required compared to reinforcement provided on 12.5m model

Columns	Control		As1		As2	
	Reinf Req^0	Reinf Provided	Reinf Req^0	Reinf Provided	Reinf Req^0	Reinf Provide
C1	1440	8@16	2688	8@22	2688	8@25
C2	1440	8@16	1445	8@16	1445	8@18
C3	1440	8@16	1466	8@16	1516	8@16
C4	1440	8@16	1465	8@16	1571	8@16
C5	1440	8@16	1445	8@16	1996	8@18
C6	1440	8@16	2690	8@22	3690	8@25

Table 7.4: Comparison of Reinforcement require to reinforcement provided in 30.5m model

Column	Control		As1		As2	
	Reinf Req^0	Reinf Provided	Reinf Req^0	Reinf Provided	Reinf Req^0	Reinf Provide
C1	1440	8@16	4849	12@25	4849	12@28
C2	1440	8@16	2559	12@18	2559	12@20
C3	1440	8@16	2545	12@18	2661	12@18
C4	1440	8@16	2545	12@18	2892	12@18
C5	1440	8@16	2559	12@18	3568	12@20
C6	1440	8@16	4849	12@25	7190	12@28

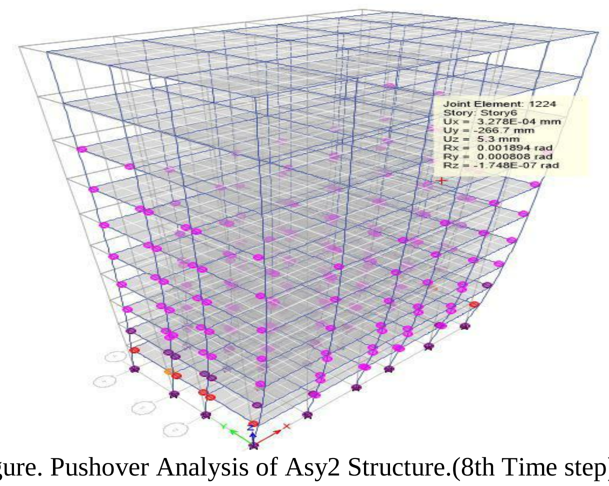


Figure. Pushover Analysis of Asy2 Structure.(8th Time step)

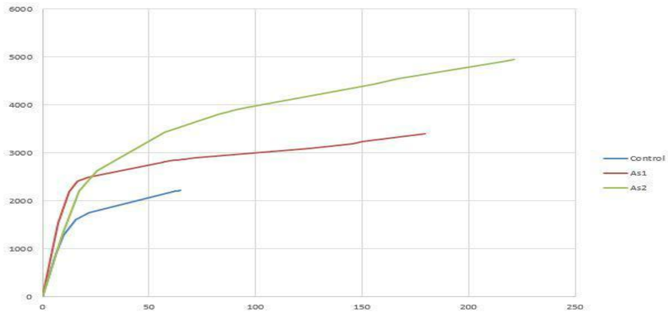


Figure Pushover Analysis for 12.5m Model

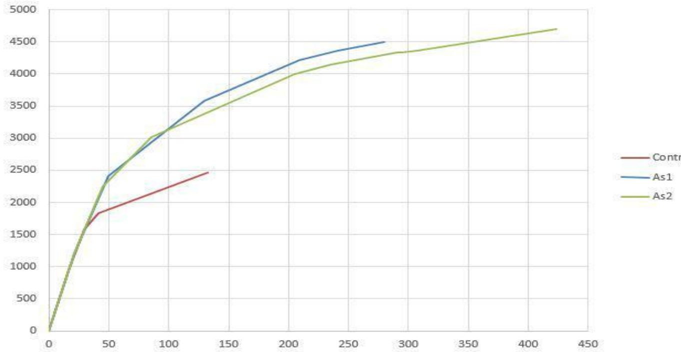


Figure: Pushover Analysis 30.5m Model

V. CONCLUSION

As per the data presented in the previous Section 7.4 it can be concluded that though the impact of the earthquake force is great on the 12.5 m model the resultant effect of the eccentricity is small for the 12.5 story model while the the 30.5m model experiences a more significant change when the mass eccentricity is applied . Hence the useful for tall structures like the 30.5m model but not so effective for the smaller 12.5m model. The change in the inner section of the building is small for the 12.5 and the 30.5 model, while the difference increases as we approach the periphery hence it is proposed that to save time the inner most columns can be designed for the column to the periphery and the design can be applied to all the innermost columns as the variation is very small while the outer columns at the buildings periphery need to be designed separately. The rise in the reinforcement required with the height of the building makes it possible for a simpler formula for calculation of the reinforcements of the structure thought the exact formulation of the formula will require study of more models and further study.

Plastic Hinges

The Plastic Hinges are used for performing the pushover analysis. The plastic hinges are induced at the edges of each structural member such that they divide the frame into the individual members. The beams have an M3 type hinge at the end which take only the moment into account while the Columns have the P2-M2-M3 hinge type assigned to them which include the effect of axial force and the effects of bi-axial bending. Their primary purpose is to serve as an energy damping device for allowing deformations of seemingly rigid sections in earthquake engineering.

Static Pushover Analysis

The Pushover analysis is a Nonlinear Static analysis in which the structure is subjected to a displacement controlled lateral load pattern which continuously increases till the structure is forced from its elastic behavior to nonelastic behavior till the collapse condition is reached. There is also another variant of the static pushover analysis in which the structure is first subjected to the lateral load in one direction and then the same stressed structure is subjected to similar loading in the opposite direction. This approach is known as a Cyclic Pushover Analysis its been replaced by the use of Time History Analysis using periodic functions.

Time History Analysis

The Time History Analysis is a form of non-linear dynamic analysis. The similarity

between the dynamic and static analysis was maintained by keeping the standard hinges used for the static analysis. The analysis was done by neglecting the geometric irreg-ularities like the P-4 effect. The modal analysis is done with Ritz vectors which give a more accurate model than just Eigen Vectors. The analysis was intended to be done on all 3 models but there were numerous cases where the ground motion analysis wasn't completing hence as the data is incomplete the data was not used in the main text for drawing conclusion. The most complete set of data was that of in the 30.5m Asy2 model and here the output is shown for all the ground motions. The ground motions for the other models are mostly not complete hence not shown here. The Earthquake ground motions considered are as follows showing the input followed by the output as a plot between joint rotation and time.

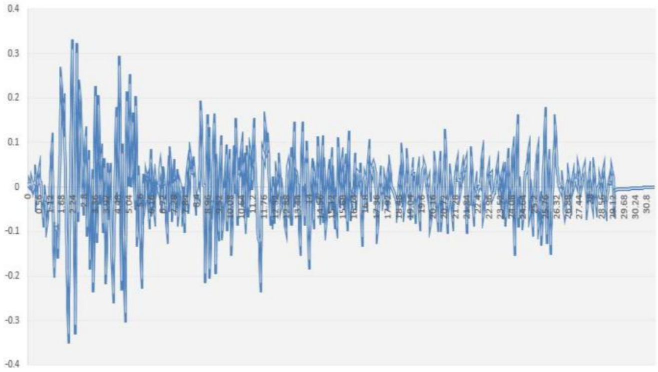
Table C.1: Earthquake Magnitudes

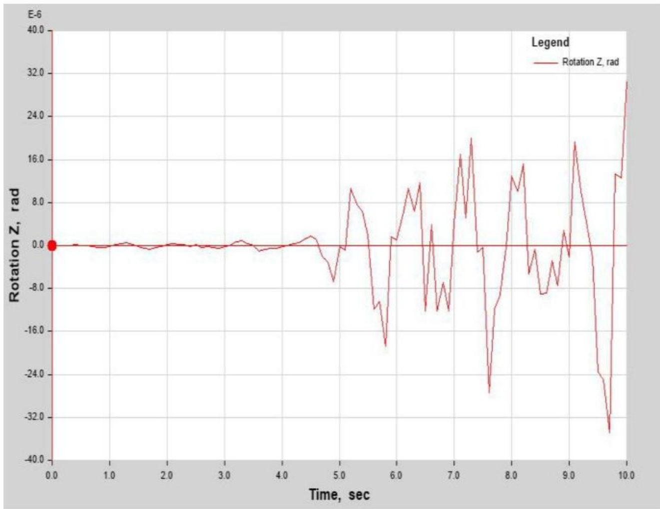
No	Earthquake	Magnitude	Epicenter (Km)
1	Loma Prieta-Oakland, October 17, 1989	7.1	3.5
2	Loma Prieta-Corralitos, October 17, 1989	7.1	7
3	Northridge-Santa Monica, January 17, 1994	6.7	23
4	Northridge-Sylmar, January 17, 1994	6.7	16
5	Northridge-Century City, January 17, 1994	6.7	20
6	Landers- Lucerne Valley, June 28, 1992	7.3	42
7	Sierra Madre-Altadena, June 28, 1991	5.6	12.6
8	Imperial Valley Earthquake-El Centro, October 15,1979	6.6	13.2
9	1992 Cape Mendocino Petrolia earthquakes	7.2	
10	Sierra Madre-Yermo, June 28, 1991	5.6	35.5

Table C.2: Earthquake Durations and PGA

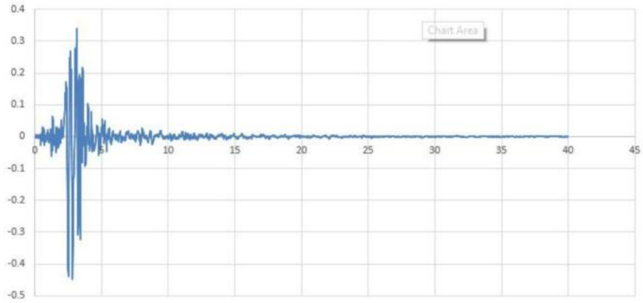
No	Earthquake	Duration (sec)	PGA (g)
1	Loma Prieta-Oakland, October 17, 1989	40	0.28
2	Loma Prieta-Corralitos, October 17, 1989	40	0.63
3	Northridge-Santa Monica, January 17, 1994	60	0.37
4	Northridge-Sylmar, January 17, 1994	60	0.84
5	Northridge-Century City, January 17, 1994	60	0.22
6	Landers- Lucerne Valley, June 28, 1992	48	0.68
7	Sierra Madre-Altadena, June 28, 1991	40	0.44
8	Imperial Valley Earthquake-El Centro, October 15,1979	40	0.37
9	1992 Cape Mendocino Petrolia earthquakes	40	.186
10	Sierra Madre-Yermo, June 28, 1991	80	.4

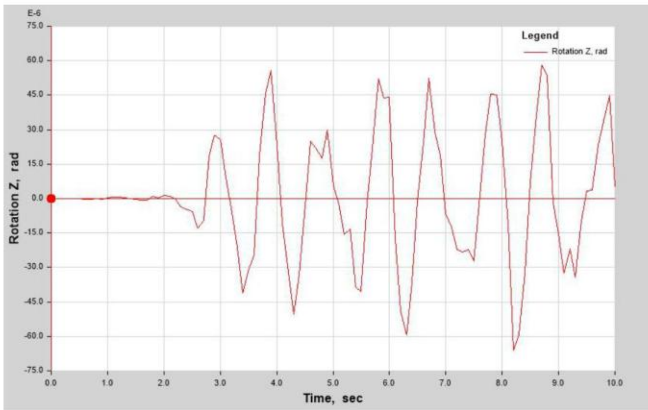
El Centro Earthquake



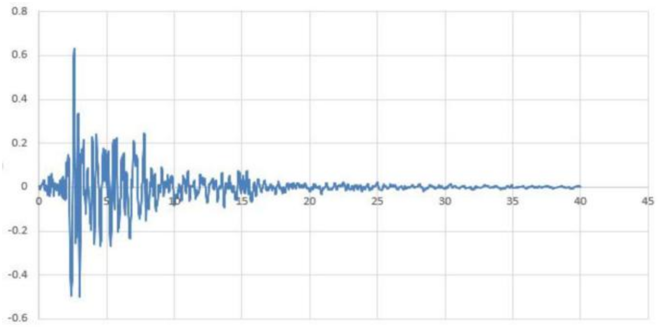


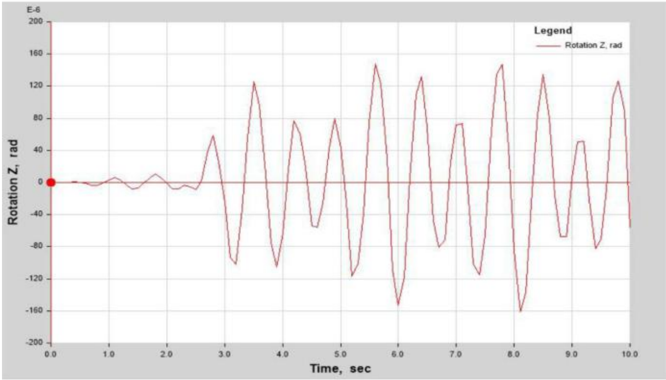
Sierra Madre-Altadena





Loma Prieta-Corralitos





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