Study Of Plan Irregularity Of High Rise Building Special Moment Steel Frames

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Abstract- *IS 1893 (part1):2016 describes various types of irregularities in building as per clause 7.1 and clause 7.7 suggests Dynamic analysis by Time History Method (THA) or Response Spectrum Method (RSA) for irregular buildings. For regular building Equivalent Static Analysis (ESA) which is based on empirical time period is suggested. From previous research it is observed that behaviour of irregular building during earthquake is more disasters. In irregular building excessive stresses or forces may develop in particular portion of the structure which may cause severe damage during earthquake. It is necessary to identify the performance of such building during earthquake and design it for better performance. The attempt is made to study the seismic forces effect when moment resisting steel frame is provided to the same structure. This paper is focused on irregularity in plan due to Re-entrant corner. buildings with large projections of Re-entrant corners results in torsion. The whole analysis work is carried out by using SAP 2000.*

Keywords- Plan Irregularity, IS 1893 (Part1) 2016, re-entrant corners, building, Moment resisting steel frame, SAP 2000.

I. INTRODUCTION

Among categorizations of seismic behaviour that have been adopted in modern codes is extreme torsional irregularity. Torsional irregularity is not an unfamiliar concept, having been expressed in codes in various forms for decades. It is an issue that engineers have learned to deal with, particularly in seismically active areas. Extreme torsional irregularity, however, is a somewhat newer concept and subset within the larger issue of torsional behaviour[1]. It is something that can greatly limit and restrict flexibility in choosing seismic force-resisting systems and configurations.

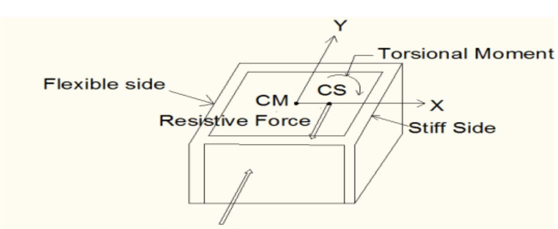


Fig 1.1: Generation of torsional moment in asymmetric structures

Recent codes have defined torsional irregularity as the condition where the maximum story drift, including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts at the two ends of the structure. A little pencil work will show this means that if one end of a rectangular structure drifts more than 1.5 times the other end, torsional irregularity is said to exist. For the newer category of extreme torsional irregularity, the calculation steps are fundamentally the same, but this designation is assigned to structures where the maximum story drift, including accidental torsion, at one end of the structure transverse to an axis is more than 1.4 times the average of the story drifts at the two ends of the structure[1]. Again, in simple terms, this means that if one end of a rectangular structure drifts in excess of 2.33 times the other end, extreme torsional irregularity is said to exist.

II. METHODOLOGY

The base of this analysis is finite element analysis. The finite element analysis is a numerical technique. In this method all the complexities of the problems, like varying shape, boundary conditions and loads are maintained as they are but the solutions obtained are approximate. Because of its diversity and flexibility as an analysis tool, it is receiving much attention in engineering. The fast improvements in computer hardware technology and slashing of cost of computers have boosted this method, since the computer is the basic need for the application of this method. A number of popular brand of finite element analysis packages are now available commercially. Some of the popular packages are STAAD-PRO, GT-STRUDEL, NASTRAN, NISA and ANSYS. Using these packages one can analyze several complex structures. The finite element analysis originated as a method of stress analysis in the design of aircrafts. It started as an extension of matrix method of structural analysis.

Civil engineers use this method extensively for the analysis of beams, space frames, plates, shells, folded plates, foundations, rock mechanics problems and seepage analysis of fluid through porous media. Both static and dynamic problems can be handled by finite element analysis.

III. DATA PREPARATION FOR DESIGN OF STRUCTURE

Structural framing system

The proposed structure is a 10 storey building. Eccentricity provided for bracing is 0.3mm Details of super structure are described below,

Location	:	Seismic zone IV
Zone factor	:	0.24
Importance factor	:	1.2
Type of structure	:	10 storeyed steel frame building
Type of occupancy	:	Hotel
Height of structure	:	33 m
Typical storey height	:	3 m
Depth of foundation	:	1.5 m

A. Material Properties

The strength of structure depends upon strength of material from which it is made.

Unit weight of masonry:	:	20\KN/m^3
Unit weight of R.C.C	:	25k KN/m^3
Unit weight of steel	:	79 KN/m^3
Grade of concrete for R.C.C and Steel	:	M20
Grade of steel	:	HYSD bars for reinforcement Fe 415
Modulus of Elasticity for R.C.C.	:	5000 X √fck N/mm2
Modulus of Elasticity for Steel	:	2.1 x 105 N/mm2

B. Load Consideration

In this study two types of loads are considered, which is gravity load that includes dead and live load and another is lateral load that includes seismic and wind load.

1. Dead load

Dead load includes self-weight of structure.

External wall 150x240x650 mm	=	Around $3KN/m^2$
Internal Wall 125x240x650 mm	=	Around $3KN/m^2$
Slab Load 100 mm thick	=	$2.5KN/m^2$
Unit weight of concrete	=	25 KN/m^3

2. Live load

Live load s as per IS 875 Part 2 1987

Storage	= $3KN/m^2$
Passage	= 4 KN/m^2
Staircase	= $5KN/m^2$
Rooms	= $2KN/m^2$
Roof	= $1.5KN/m^2$

3. Wind Load

The wind pressure on a structure depends on the wind response of the structure. The Wind Load is assign as per IS 875 Part 3 1987.

Wind Intensity	= 47 m/s
K1	= 1
K2	= 0.93
K3	= 1
P=0.6*k1*k2*k3*Vb*Vb	= 1232.622 N/m^2

4. Earthquake Load

Required data for earthquake load are as per IS 1893:2016.

Seismic Zone	: 0.24
Response Reduction Factor	: 5
Important Factor	: 1.2
Rock and Soil Site Factor	: 2
Type of Structure	: 2
Damping Ratio	: 0.05

5. Load combination

Load combination are as per IS 1893:2016 and 875:1987

a. 1.7DL + LL
b. 1.7DL +/- EQ
c. 1.7DL +/- WL
d. 1.3DL + LL +/- EQ
e. 1.3DL + LL +/- WL
f. 0.9DL +/- 1.7EQ
g. 0.9DL +/- 1.7WL
h. 1.7*DL+/- Time History

i. 1.3*DL + LL +/- Time History

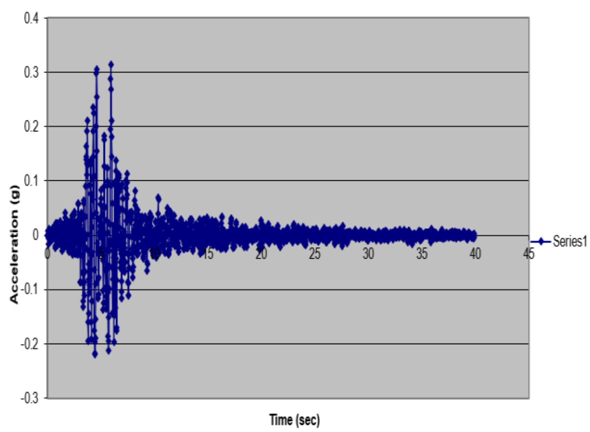
C. Dimensions consideration for design:

For steel frame

Beams	Main Beam
	UB 356X171X45
	UB 356X171X51
	Secondary Beam
	ISMC 225
	ISMC 175
Column	UC 305X305X158
Staircase	ISMC 175
Bracing	ISA 130X130X16

D. Time History

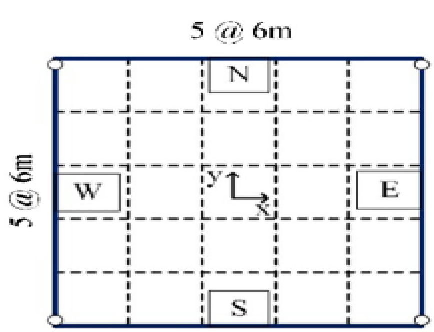
Time history includes live load, super impose load, for lateral load ground motion UTTARKASHI 1991 ground motion is used.



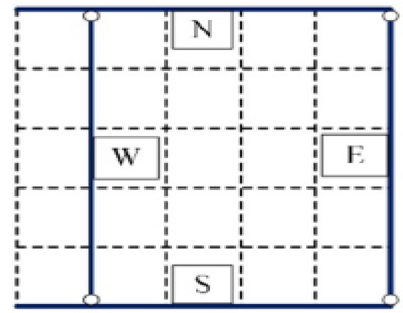
Graph 5.1: Acceleration vs Time

For irregular building changes are

Slab thickness is 150mm including floor finish. Size of beam, for main beam UB 406x178x74. For secondary beam ISMC 250. Wall load is consideres as 4.2KN/m^2 the thickness of brick is 200x240x650mm.



MODEL-II



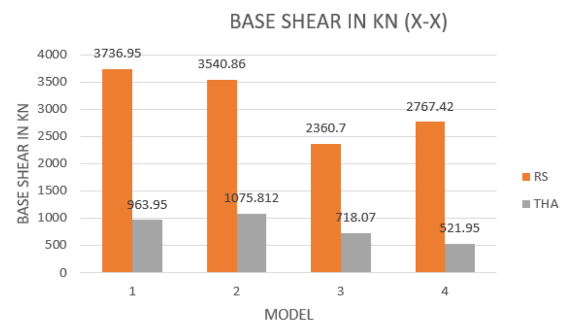
IV. COMPARISON OF RESULTS

Base shear has been tabulated and also represented graphically for models with respect to X and Z direction.

Base Shear +X:

BASE SHEAR IN KN (X-X)			
	Model	**RS**	**THA**
Without Bracing	1	3736.95	963.95
	2	3540.86	1075.812
With Bracing	3	2360.70	718.07
	4	2767.42	521.737

Table 6.1: Base Shear in X Direction.



Graph 6.1: Base Shear in X Direction.

From above graph, it is observed that, the base shear value of model 1 and 2, is decreased by 36.82% and 21.84% resp. in modal 3 and 4. Modal 3 and 4 are the same model 1 and 2 but they are provided with moment resisting steel frame. The results are obtained from response spectrum method.

The base shear value of model 1 and 2, is decreased by 25.5% and 51.48% resp. in modal 3 and 4. Modal 3 and 4 are the same model 1 and 2 but they are provided with moment resisting steel frame. The results are obtained from time history analysis method.

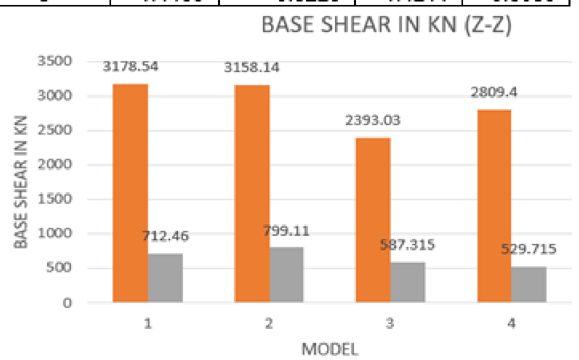
Base Shear +Z:

BASE SHEAR IN KN (Z-Z)			
	Model	**RS**	**THA**
Without Bracing	1	3178.54	712.46
	2	3158.14	799.11

With Bracing	3	2393.03	687.315
	4	2809.40	529.715

Table 6.2: Base Shear in Z Direction

STOREY NO.	STOREY DRIFT IN CM +X		STOREY DRIFT IN CM +Z	
	RS	TH	RS	TH
10	0.2508	0.115	0.2639	0.1129
9	0.5053	0.1293	0.1254	0.1539
8	0.5301	0.1806	0.6766	0.2103
7	0.6566	0.2372	0.8494	0.2874
6	0.7411	0.2935	0.9534	0.3602
5	0.8104	0.3047	1.0532	0.389
4	0.8528	0.3739	1.1017	0.4701
3	0.8792	0.8099	1.1372	0.5105
2	0.8899	0.485	1.1309	0.6065
1	1.4468	0.8225	1.4211	0.8039



Graph 6.2: Base Shear in Z Direction.

From above graph, it is observed that, the base shear value of model 1 and 2, is decreased by and 11.04% resp. in modal 3 and 4. Modal 3 and 4 are the same model 1 and 2 but they are provided with moment resisting steel frame. The results are obtained from response spectrum method.

The base shear value of model 1 and 2, is decreased by 17.56% and 33.71% resp. in modal 3 and 4. Modal 3 and 4 are the same model 1 and 2 but they are provided with moment resisting steel frame. The results are obtained from time history analysis method

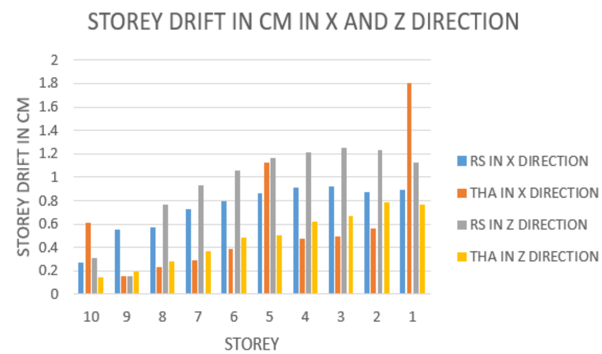
STOREY DRIFT

The story drift for all modal is been tabulated and also represented graphically.

Modal 1:- Normal building with TYPE-I arrangement

STOREY NO.	STOREY DRIFT IN CM +X		STOREY DRIFT IN CM +Z	
	RS	TH	RS	THS
10	0.2689	0.6091	0.3071	0.1473
9	0.5511	0.1554	0.1567	0.1894
8	0.5768	0.2323	0.7619	0.2847
7	0.723	0.2884	0.9312	0.3666
6	0.7984	0.384	1.0571	0.4818
5	0.8658	1.1215	1.1599	0.5027
4	0.9095	0.4797	1.2164	0.6241
3	0.9193	0.4905	1.2489	0.6706
2	0.8737	0.566	1.2271	0.7833
1	0.8966	1.8077	1.1294	0.7624

Table 6.3: Storey drift in X & Z direction

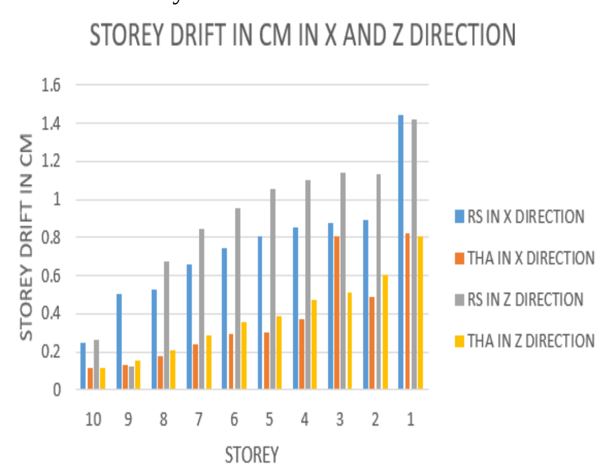


Graph 6.3: story drift in X & Z direction.

From the above graph in response spectrum analysis the story drift at storey 1 is increased by 70% and 72.80% than storey 10 in X and Z direction. Where as in case of time history analysis the story drift at storey 1 is increased by 66.30% and 80.67% than storey 10 in X and Z.

Modal 2:- Irregularity at alternate floor building TYPE-I arrangement.

Table 6.4: Storey drift in X & Z direction



Graph 6.4: story drift in X & Z direction.

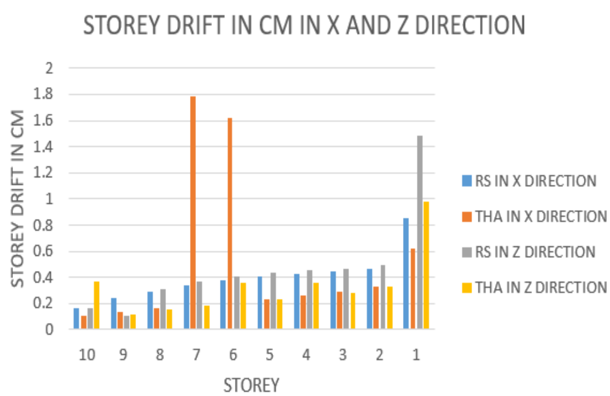
From the above graph in response spectrum analysis the story drift at storey 1 is increased by 82.66% and 81.42% than storey 10 in X and Z direction. Where as in case of time history analysis the story drift at storey 1 is increased by 86.01% and 85.95% than storey 10 in X and Z.

Modal 3:- Normal building with moment resisting steel frame.

resisting steel frame.

STOREY NO.	STOREY DRIFT IN CM +X		STOREY DRIFT IN CM +Z	
	RS	**TH**	**RS**	**TH**
10	0.1637	0.1023	0.1674	0.3683
9	0.2399	0.1324	0.107	0.118
8	0.2949	0.1614	0.3145	0.1551
7	0.3433	1.7836	0.3668	0.1798
6	0.3827	1.6233	0.4086	0.355
5	0.4114	0.2372	0.439	0.2352
4	0.431	0.2643	0.4594	0.3577
3	0.4431	0.2896	0.4698	0.2861
2	0.4617	0.3263	0.4971	0.3285
1	0.8513	0.6194	1.4806	0.979

Table 6.5: Storey drift in X & Z direction



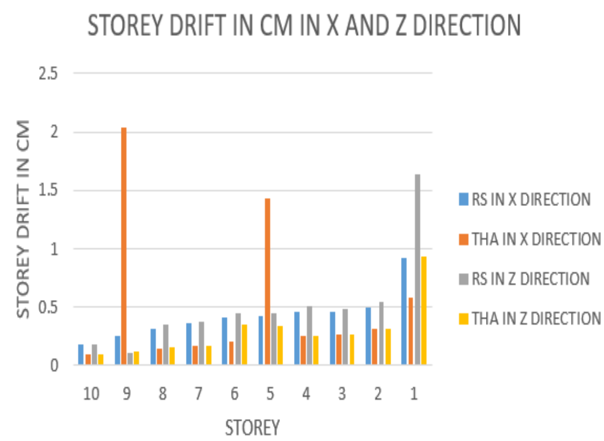
Graph 6.5: story drift in X & Z direction.

From the above graph in response spectrum analysis the story drift at storey 1 is increased by 80.77% and 88.69% than storey 10 in X and Z direction. Where as in case of time history analysis the story drift at storey 1 is increased by 83.48% and 62.37% than storey 10 in X and Z.

Modal 4:- Irregularity at alternate floor building with moment resisting steel frame.

STOREY NO.	STOREY DRIFT IN CM +X		STOREY DRIFT IN CM +Z	
	RS	**TH**	**RS**	**TH**
10	0.1806	0.099	0.1877	0.0993
9	0.2545	2.0414	0.1038	0.1173
8	0.3173	0.1507	0.347	0.152
7	0.3579	0.1738	0.3792	0.1716
6	0.4094	0.2005	0.4486	0.3475
5	0.425	1.4358	0.4501	0.3442
4	0.46	0.2517	0.5034	0.2556
3	0.4558	0.2692	0.4798	0.2652
2	0.4945	0.3102	0.544	0.3184
1	0.9176	0.585	1.6398	0.9386

Table 6.6: Storey drift in X & Z direction



Graph 6.6: story drift in X & Z direction.

From the above graph in response spectrum analysis the story drift at storey 1 is increased by 80.31% and 88.55% than storey 10 in X and Z direction. Where as in case of time history analysis the story drift at storey 1 is increased by 83.07% and 89.42% than storey 10 in X and Z.

V. CONCLUSION

In this study modelling of multi-storeyed building with plan irregularity is done. In accordance with IS1893-2016 for simulation purpose finite element analysis Staad-Pro is used following conclusions are formed after studying L-shape Building with variation of height.

1) Base shear gets decreased by 36.82% and 24.71% when moment resisting steel frame is provided to the normal building without mass irregularity in X and Z direction for RS analysis.
2) Base shear gets decreased by 25.5% and 17.56% when moment resisting steel frame is provided to the normal building without mass irregularity in X and Z direction for THA analysis.
3) The calculated value for eccentricity in moment resisting steel frame is 0.3mm which gives the relevant values of base shear, displacement, story drift.
4) From story drift results it is observed that by providing bracing the drift at top and bottom storey is reduced by 164.26% and 105.32% by RS analysis. Whereas by THA the values are 595.40% and 131.09%, in X and Z direction for normal building. While for irregularity at alternate storey, the storey drift results it is observed that by providing bracing the drift at top and bottom storey is reduced by 72% and 63.422% by RS analysis. Whereas by THA the values are 71.125% and 115.389%, in X and Z direction for normal building.

REFERENCES

[1] H. Gokdemir ⇑ , H. Ozbasaran, M. Dogan, E. Unluoglu, U. Albayrak Eskisehir Osmangazi University, Department of Civil Engineering, 26480, Eskisehir, Turkey), Effects of torsional irregularity to structures during earthquakes
[2] Alcocer, S., and Klingner, R. (2006). Tecomán earthquake, México January 21, 2003. SMIS y EERI, México, D.F. (In Spanish and English)
[3] Arnold C. y Reitherman R. (1982). Building configuration and seismic design. John and Willey sons, New York, United States.
[4] CAEE (2005). Reconnaissance report on the December 26, 2004 Sumatra Earthquake and tsunami. The Canadian Association for Earthquake Engineering, Canada.
[5] Dogangün, A. (2004). Performance of reinforced concrete buildings during the May 1, 2003 Bingöl earthquake in Turkey, Eng. Struct. 26, 841-856.
[6] Earthquake Engineering Research Institute (1983). El-Asnam, Algeria Earthquake, 10 October, 1980; A Reconnaissance and Engineering Report. EERI, United States of America. 7. Earthquake Engineering Research Institute (1989). Loma Prieta Earthquake, October 17, 1989; Preliminary Reconnaissance Report Earthquake Engineering Research Institute, Oakland, California.
[7] Earthquake Engineering Research Institute (2006). The Mw 6.3 Java, Indonesia Earthquake of May 27, 2006. Learning from earthquakes. EERI special report, August 2006, United States of America.
[8] Hopkins, O. (1993). The Philippines earthquake of July 1990-Lessons for us all from the destruction and reconstruction. Proceedings of the Tom Paulay Symposium, SP 157-21, September 20-22, La Jolla, California: 465-486.
[9] Humar, J., Lau, D., and Pierre, J. (2001). Performance of buildings during the 2001 Bhuj earthquake, Can. J. Civ. Eng. 28, 979-991.