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Dynamic Response of High Rise Structures Under The Influence of Shear Walls

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ABSTRACT
This study presents the procedure for seismic performance estimation of high-rise buildings based on a concept of the capacity spectrum method. In 3D analytical model of thirty storied buildings have been generated for symmetric buildings Models and analyzed using structural analysis tool ETABS. The analytical model of the building includes all important components that influence the mass, strength, stiffness and deformability of the structure. To study the effect of concrete core wall & shear wall at different positions during earthquake, seismic analysis using both linear static, linear dynamic and non-linear static procedure has been performed. The deflections at each storey level has been compared by performing Equivalent static, response spectrum method as well as pushover method has also been performed to determine capacity, demand and performance level of the considered building models. From the below studies it has been observed that non-linear pushover analysis provide good estimate of global as well as local inelastic deformation demands and also reveals design weakness that may remain hidden in an elastic analysis and also the performance level of the structure. Storey drifts are found within the limit as specified by code (IS: 1893-2002) in Equivalent static, linear dynamic & non-linear static analysis.

Keywords: Shear Wall, Story Drift, Displacement, ETABS, High Rise Buildings.

I. INTRODUCTION

Earthquake disaster had always been one of the great natural calamities trust upon the mankind since time immemorial and bringing in its wake untold miseries and hardship to the people affected. Indian subcontinent has been experienced with some of the most severe earthquake in the world. Simplified approaches for the seismic evaluation of structures, which account for the inelastic behavior, generally use the results of static collapse analysis to define the global inelastic performance of the structure. Currently, for this purpose, the nonlinear static procedure (NSP) which is described in FEMA-273/356 and ATC-40 (Applied Technology Council, 1996) documents are used. Seismic demands are computed by nonlinear static analysis of the structure subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a predetermined target displacement is reached. Pushover methods are becoming practical tools of analysis and evaluation of buildings considering the performance-based seismic philosophy. This is evident by the recent implementation of pushover methods in several international seismic guidelines and codes, such as the Federal Emergency Management Agency standard 273 (FEMA- 273), Euro-Code 8 (EC-8) and International Building Code (IBC-2003). In these seismic regulations, pushover methods of analysis such as the N2-method and the capacity spectrum method are recommended for determining the inelastic responses of the building due to earthquake ground motions. One main step in these pushover methods of analysis for determining the seismic demands is the construction of the pushover curve of the building by using an adequate lateral load pattern simulating the distribution of inertia forces developed through the building when subjected to an earthquake. This pushover curve represents the lateral capacity of the building by plotting the nonlinear relation between the base shear and roof displacement of the building. The intersection of this pushover curve with the seismic demand curve determined by the design response spectrum represents the deformation state at which the performance of the building is evaluated. Structures designed according to the existing seismic codes provide minimum safety to preserve life and in a major earthquake, they assure at least gravity load bearing elements of non-essential facilities will still function and provide some margin of safety. However, compliance with the standard does not guarantee such performance. They typically do not address performance of non-structural components neither provide differences in performance between different structural systems. This is because it cannot accurately estimate the inelastic strength and deformation of each member due to linear elastic analysis. Although an elastic analysis gives a good indication of the elastic capacity of structures and indicates where first yielding will occur, it cannot predict failure mechanisms and account for redistribution of forces during progressive yielding. Inelastic analyses procedures help demonstrate how buildings really work by identifying modes of failure and the potential for progressive collapse. The use of inelastic procedures for design and evaluation is an attempt to help engineers better understands how structures will behave when subjected to major earthquakes, where it is assumed that the elastic capacity of the structure will be exceeded. This resolves some of the uncertainties associated with code and elastic procedures. The capacity spectrum method, a nonlinear static procedure that provides a graphical representation of the global force-displacement capacity curve of the structure (i.e., pushover) and compares it to the response spectra representations of the earthquake demands, is a very useful tool in the evaluation and retrofit design of existing concrete buildings. The graphical representation provides a clear picture of how a building responds to earthquake ground motion.

II. ANALYTICAL MODELLING

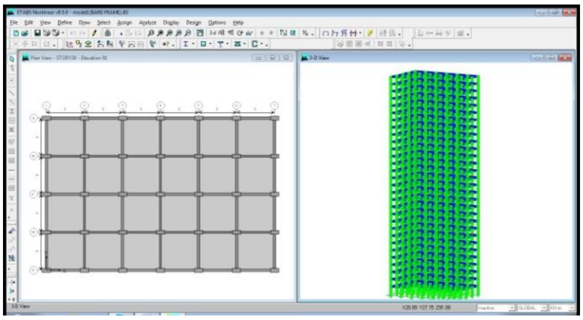
Model 1: Bare frame:- Building is modeled as bare frame. For the Analysis, a typical frame plan dimensions 30mx20m and height 91m is considered. The longer plan dimension is taken on X-direction, the shorter one as Y-direction and Z-direction is taken in the vertical direction. The aspect ratio is taken as 1.5 so as to study the effect due to the orientation of shear walls along longer plan dimension. Along the longer dimension in the plan, six frames are considered. Along the shorter direction, four bays are considered. the ground storey height is taken as 4m and the rest of the storeys are taken to be 3m high. Upto to the 20th storey,the column cross section is taken as 1.20mx0.50m. for the rest 10 storeys, the column cross section is taken as 1.10mx0.50m.Up to 3rd storey,the beam cross section is taken as 0.3mx0.6m.From 3rd storey to the 20th storey,the beam cross section is taken as 0.30mx0.525m.For the remaining top ten storeys the cross section of beams are taken as 0.30mx0.45m.The floor slabs are modeled as membrane element of 0.15m thickness. All the supports are modeled as fixed supports. Linear and Non-Linear analysis is conducted on each these models.

The loads are considered for the analysis are given below.

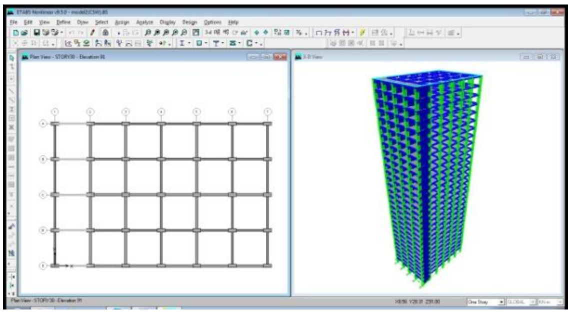
Dead Load:-The dead load of structure is obtained from Table 1,Page 8,of IS 875-Part 1-1987.The permissible value for unit weight of reinforced concrete varies from 24.80kN/m^3 to 26.50kN/m^3.From the table, the unit weight of concrete is taken as 25kN/m^3, assuming 5% steel in the reinforced concrete.

Imposed Load:- The imposed load on the floor is obtained from Table 1 of IS 875 (Part 2) – 1987. The uniformly distributed load on the floor of the building is assumed to be 4kN/m^2 (for assembly areas, corridors, passages, restaurants, business and office buildings, retail shops etc).

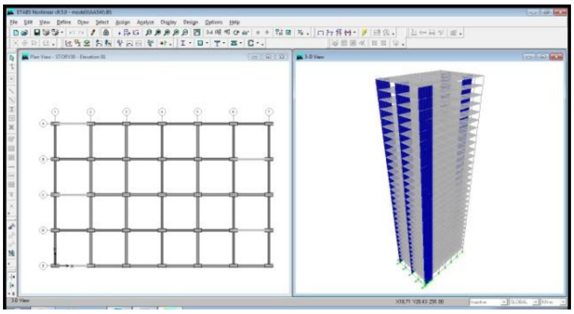
Earth Quake Load:- The structure is assumed to be in Hyderabad (Zone 2 as per IS 1893 – 2002). So the zone factor is taken as 0.10 as per Table 2 of IS 1893 – 2002. The damping is assumed to be 5%, for concrete as per Table 3 of IS 1893 –2002. Importance factor is taken as 1.5 as per Table 6 of IS 1893 – 2002.



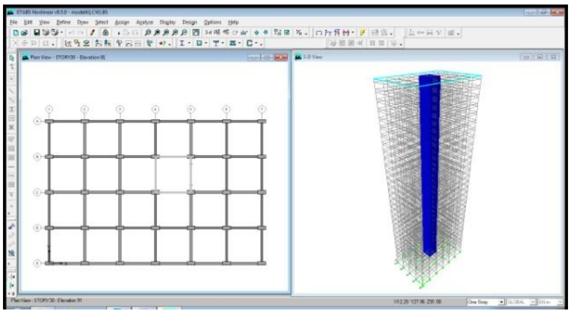
Model 2: Bare frame with conventional shear walls (CSW):-The second model is obtained by added conventional shear walls to the bare frame the arrangement of conventional shear walls along x-direction. The shear walls adopted area bay wide and a storey height and without opening. This improves the stiffness of the shear walls the shear wall system can be provided conveniently by adjusting the utility of the area adjacent to the shear wall panels. However masses of floor finish and imposed live load is added at each storey.



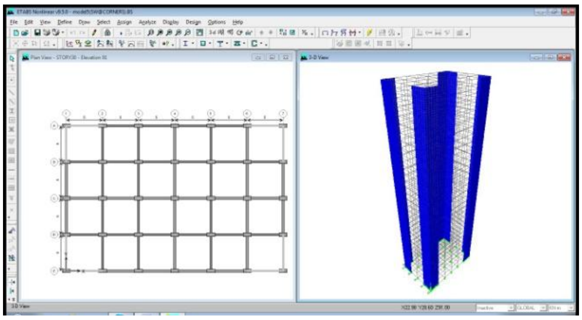
Model 3: Bear frame with alternate arrangement of conventional shear wall system (AASW):-In the conventional shear wall system all the shear wall in a frame are provide one above the other. The shear walls in the alternate storeys are placed at two extreme ends of frame. In this case models with shear wall provided along x-direction. However masses of floor finish and imposed live load is added at each storey.



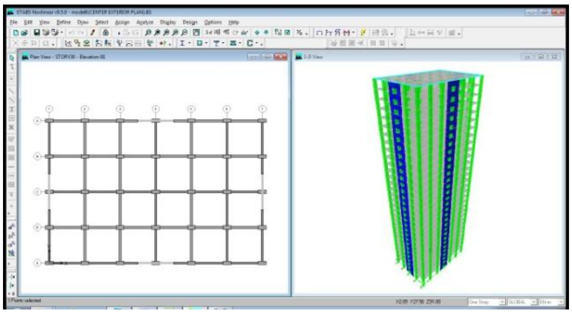
Model 4: Bare frame with lift-core walls (LCW):- The high rise structure will be having lifts. The core-walls (shear walls) around the lift core will add up to the stiffness of the structure, there by reducing the deflection. This set of model is tended to study the effect of lift core walls on the response of the structure. Three sides of the lift chamber are having shear wall panels and fourth side is left open to provide access to the lift chamber. However masses of floor finish and imposed live load is added at each storey.



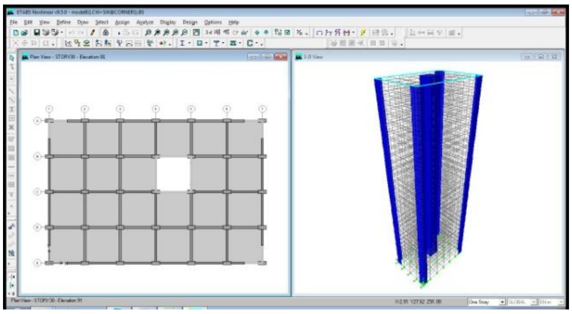
Model 5: Bare frame with conventional shear wall (L-section) at exterior corners:- In this case the conventional shear wall are placed at exterior corners (L-section) in the structure. However masses of floor finish and imposed live load is added at each storey.



Model 6: Bare frame with conventional shear wall at centre of exterior panel:-In this case conventional shear wall is provided at center in both x and y-direction. However masses of floor finish and imposed live load is added at each storey.



Model 7: Bare frame with LCW and SW at Corners:-In this case model is prepared by adding shear wall at corner and lift core wall to the bare frame. However masses of floor finish and imposed live load is added at each storey.



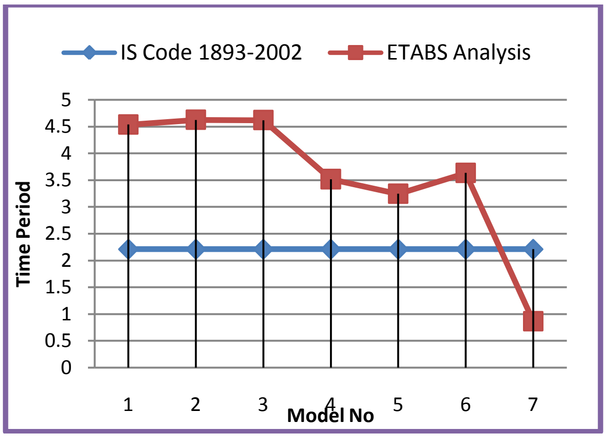
III. RESULTS AND DISCUSSION

The results of the selected building studied are presented and discussed in detail. The results are included for building models and the response results are computed using the response spectrum and pushover analysis. The analysis and design of the different building models is performed by using ETABS analysis package.

NATURAL PERIODS:-From the Table 1 and Graph 1, it is observed that the time period obtained by the IS code and by the ETABS analysis possess a huge difference. The table shows that the natural time period of bare frame model from ETABS is almost twice more than that of the value obtained from code. For models-2,3,4,5 & 6 the time period obtained from ETABS is higher as compared to the corresponding values from the IS code. Out of all the models the time period is maximum for model-2 and minimum for model-7. From ETABS analysis it can be observed that from the below table 1 vertical period of bare frame (model 1) is greater than four (model-4,5,6,7) cases of building models and while comparing model to each other, the model 4,5,6 and 7, time periods are 22.25%, 28.42%, 28.41%, 20.07%, 80.92% less compared to as model-1.

Table 1:-Comparison of time period between IS Code method and analysis using ETABS software for various models.

Model No.	**IS Code 1893-2002**		**ETABS Analysis**	
	longitudinal	**transverse**	**longitudinal**	**transverse**
1	2.2097	2.2097	4.5383	4.5383
2	2.2097	2.2097	4.6267	4.6267
3	2.2097	2.2097	4.6206	4.6206
4	2.2097	2.2097	3.5186	3.5186
5	2.2097	2.2097	3.2449	3.2449
6	2.2097	2.2097	3.6345	3.6345
7	2.2097	2.2097	0.8666	0.8666

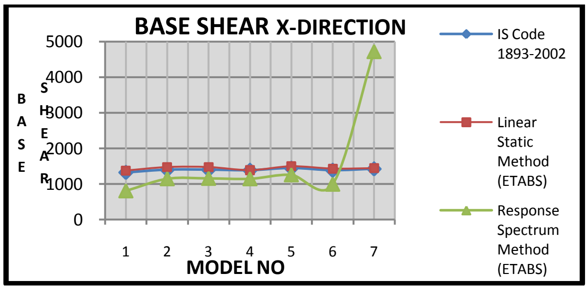


Graph 1:-Model Vs Time period for different models along longitudinal and transverse direction

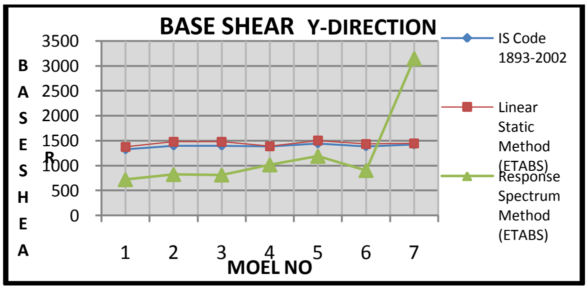
DESIGN SEISMIC BASE SHEAR:-From the below Table 2 Represents The Seismic Base Shear For Various Models. From the Table it can be Observed that the seismic base shear for all the models except model 1 has smaller values compared to others models. The reduced percentages from model 2 to model 7 are 6.91%, 6.91%, 0.92%, 8.30%, 3.95% and 4.80% respectively. It can be observed that the Response Spectrum Analysis Yields Lesser Values Of Base Shear as compared to that of equivalent static analysis as the higher modes are given due consideration. Table 2 represents the comparison of base shear obtained from IS Code method, ESM and RSM .From the above table, it is clearly identified that the values obtained from the IS Code method are the least as compared to the ESM. Whereas ESM yields the largest values and further the curves for IS Code lies in between that of ESM and RSM method. Apart from the bare frame model the values for the rest of the models lies almost in a straight horizontal line obtained from IS Code and ESM where as in case of RSM the base shear for each model fluctuates very significantly as shown in the below Graph 2 and 3. It has been found that calculation of earthquake forces by treating the buildings as ordinary frames results in an underestimation of base shear.

TABLE 2:-Comparison of Base Shear by IS code method, Equivalent Static method(ESM) and Response Spectrum method(RSM) for various models

Base shear (KN)						
Model No.	**IS Code 1893-2002**		**Linear Static Method (ETABS)**		**Response Spectrum Method (ETABS)**	
	longitudinal	**transverse**	**longitudinal**	**transverse**	**Longitudinal**	**transverse**
1	1325	1325	1374	1374	805.1	723.1
2	1399	1399	1476	1476	1145.1	825
3	1399	1399	1476	1476	1155.6	815.5
4	1384	1384	1387	1387	1144.3	1017
5	1444	1444	1499	1499	1255.1	1189
6	1384	1384	1431	1431	990.3	906.1
7	1423	1423	1444	1444	4719.1	3145



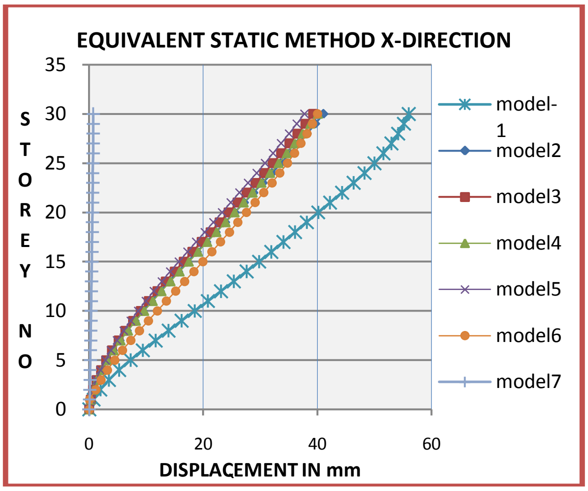
Graph 2:-Comparison of Base Shear by IS code method, ESM and RSM for various models a long longitudinal direction



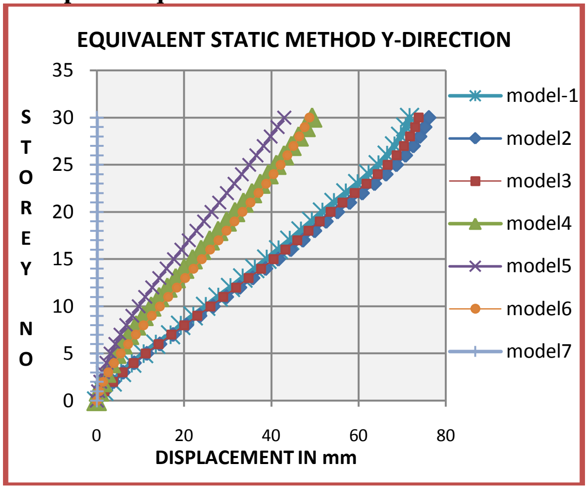
Graphs 3:-Comparison of base shear by IS Code method, ESM and RSM for various models along transverse direction

LATERAL DISPLACEMENTS:-The maximum displacements at each floor level with respect to ground for equivalent static response spectrum and pushover analysis. For better comparability the displacement for each model along the two directions

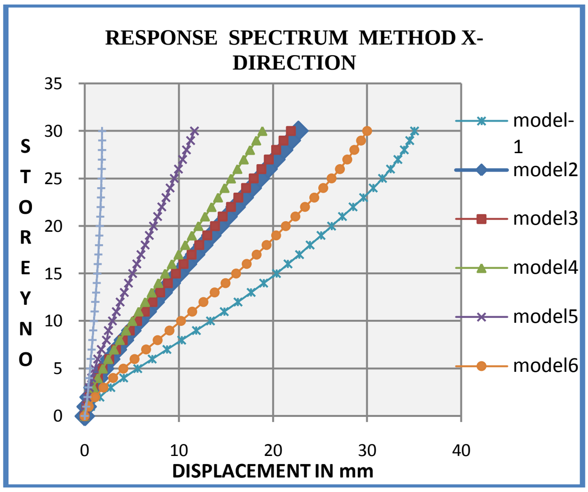
of ground motion are plotted in as shown in Graphs from 4 to 9. In the three dimensional model, however, there are six degrees of freedom with the two translational degree of freedom along X, Y-axes and rotation degree of freedom about Z (vertical)-axis playing significant role in the deformation of the structure. Apart from the translation motion in a particular direction, there is always an additional displacement due to the rotation of floor. Due to this the maximum displacement at floor levels obtained by three-dimensional analysis are always greater then the corresponding values obtained by one-dimensional analysis



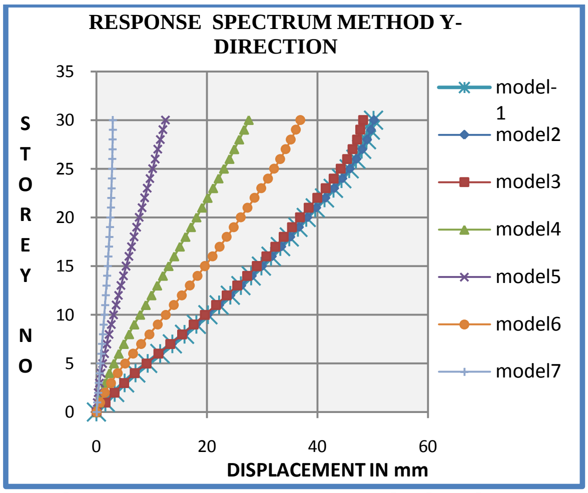
Graph 4:-Equivalent Static method X-direction



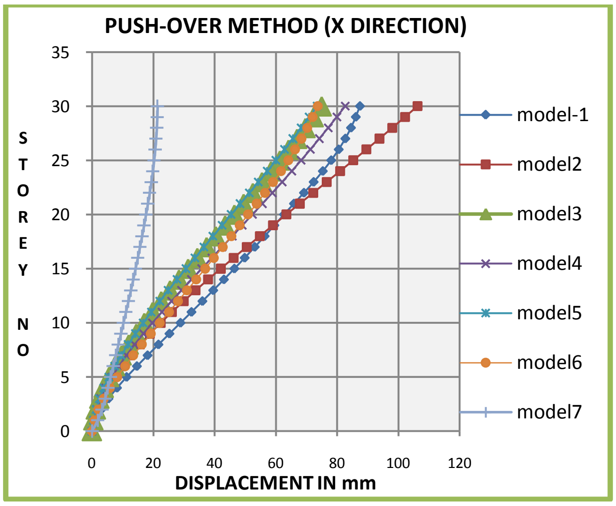
Graph 4:-Equivalent Static method Y-direction



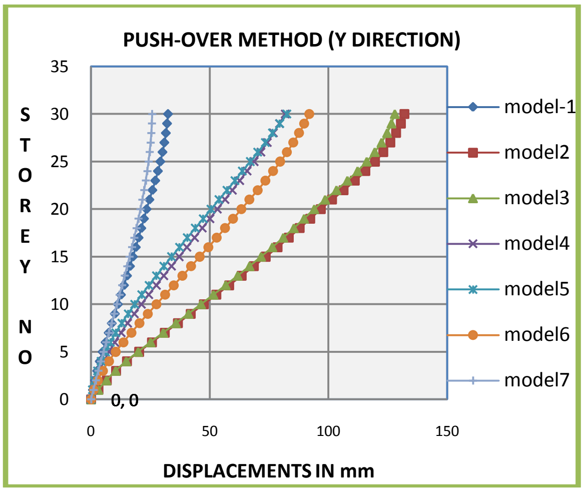
Graph 6:-Response Spectrum method X-direction



Graph 7:-Response Spectrum method Y-direction



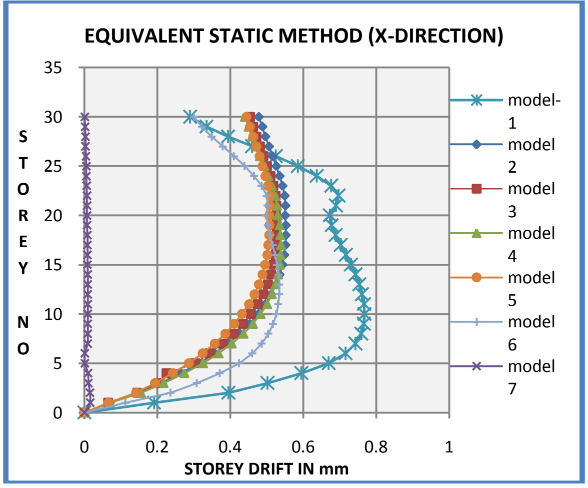
Graph 8:- Push-Over method X- direction



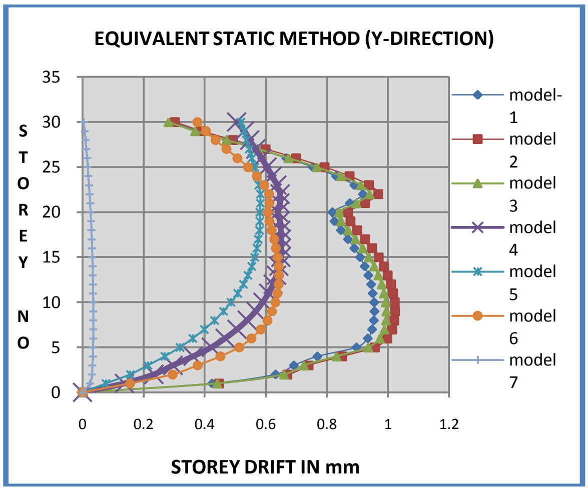
Graph 9:- Push-Over method Y- direction

Moreover, the floor rotation is maximum at the top floor, gradually reducing down the height of the building to an almost negligible rotation at the lowest basement floor. In equivalent static analysis it has been found that model -2, model-3, model-4, model-5, model-6 and model-7 has 26.71%, 29.93%, 28.44%, 32.66%, 28.55% and 28.72% respectively less displacement as compared to the model-1 in longitudinal direction and in transverse direction model-4, model-4, model-6, and model-7, has 31.14%, 39.89%, 31.95%, and 99.94% respectively less displacement compared to model-1. In response spectrum analysis it has been found that model -2, model-3, model-4, model-5, model-6 and model-7 has 35.18%, 37.44%, 46.15%, 66.74%, 14.38% and 94.77% respectively less displacement as compared to the model-1 in longitudinal direction and in transverse direction model-3, model-4, model-5, model-6, and model-7 has 3.75%, 45.03%, 75.11%, 26.42% and 94.11% respectively less displacement compared to model-1. In pushover analysis it can be seen that it has been found that model -3, model-4, model-5, model-6, and model-7 has 14.30%, 5.51%, 15.93%, 15.76% and 75.52% respectively less displacement as compared to the model-1 in longitudinal direction, and transverse direction model-7 has 20.58% less displacement as compare to model-1

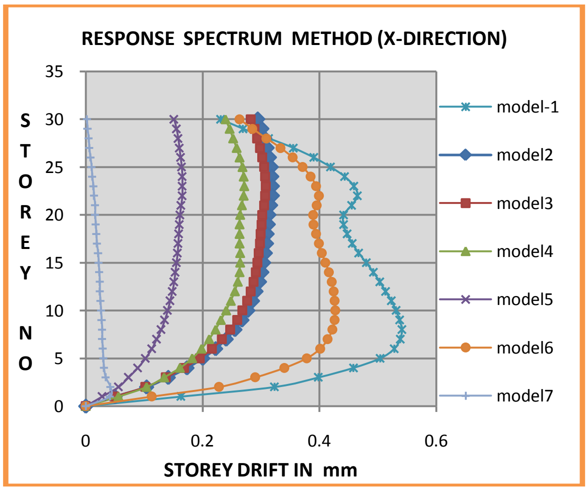
STOREY DRIFTS:-The permissible inter-storey drift is limited to 0.004 times the storey height, so that minimum damage would take place during earthquake and pose less psychological fear in the minds of people. The storey drifts for all models of descending building along longitudinal and transverse directions are shown in Graph from 10 to 15. From the below Graph it can be seen that, all storey drifts are within the permissible limit (0.004*h=12mm) and the storey drifts in lower stories are larger than that in top stories.



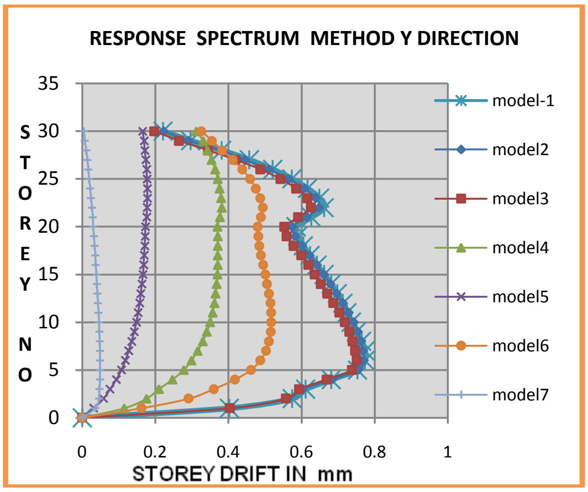
Graph 10:-Equivalent Static method X-direction



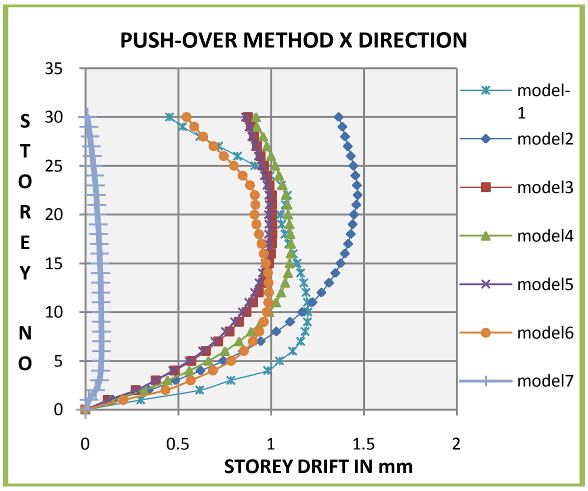
Graph 11:-Equivalent Static method Y-direction



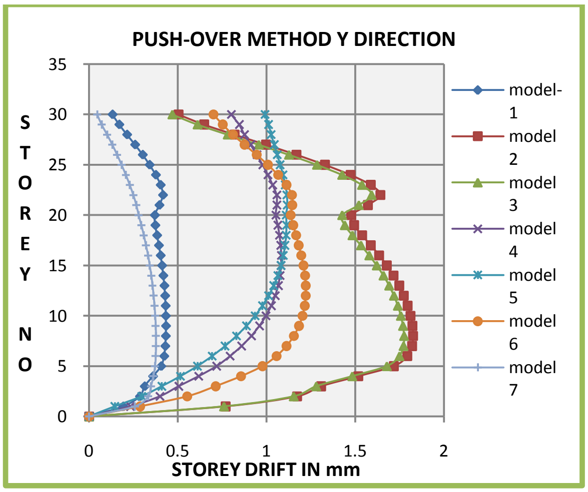
Graph 12:-Response Spectrum method X-direction



Graph 13:-Response Spectrum method Y-direction



Graph 14:- Push-Over method X- direction



Graph 15:- Push-Over method Y- direction

Table-3: Response reduction factor and ductility ratio along longitudinal direction

	M-1	M-2	M-3	M-4	M-5	M-6	M-7
Yield displacement (U_{Yield}) (mm)	87.40	106.2	74.9	82.6	73.5	73.7	21.4
Ultimate displacement $U_{ultimate}$ (mm)	1039	805.4	659	790	716	556	135
Ductility ratio μ	11.89	7.60	8.80	9.56	9.74	7.55	6.34
R	4.77	3.76	4.08	4.26	4.30	3.76	3.42

	M-1	M-2	M-3	M-4	M-5	M-6	M-7
Yield displacement (U_{Yield}) (mm)	32.4	132	128	81.7	82.5	91.9	25.8
Ultimate displacement $U_{ultimate}$ (mm)	100	163	535	838	742	664	385
Ductility ratio μ	30.95	1.241	4.18	10.2	8.99	7.23	14.9
R	7.805	1.22	2.71	4.42	4.12	3.67	5.37

Table-4: Response reduction factor and ductility ratio along transverse direction

PERFORMANCE POINT:- The values of performance point parameters such as structural acceleration (Sa), structural displacement (Sd), base shear (V) and roof displacement (D) are shown in Table 3 and 4 along longitudinal and transverse direction for all the building models. It can be noted that the structural displacement (Sd) and roof displacement (D) has smaller value for model 7 as compared to other models, it can also be seen that for structural acceleration (Sa) is maximum for model-7 and base shear (v) is almost max. for model 7 as compared to other models.

Table 5:-Performance point parameter for building models along longitudinal direction

Model No.	**1**	**2**	**3**	**4**	**5**	**6**	**7**
Spectral Acceleration (Sa)	0.019	0.017	0.013	0.028	0.032	0.022	0.178
Spectral Displacement (Sd) mm	164	199	170	147	145	127	39
Base Shear (KN)	2843	2953.1	2048.8	3912.6	4584.4	3353.9	29765.72
Roof Displacement (mm)	222	11	70	221	221	174	52
A-B	7576	7026	7582	6777	7313	7397	6496
B-IO	241	544	275	448	588	432	1005
IO-LS	308	470	165	498	581	943	575
LS-CP	922	416	430	851	536	270	10
CP-C	0	0	0	0	0	0	11
C-D	0	4	8	0	2	0	1
D-E	13	0	0	6	8	18	2
>E	0	0	0	0	0	0	0
Total no. of hinges	9060	8460	8460	8580	9028	9060	8100

Model No.	**1**	**2**	**3**	**4**	**5**	**6**	**7**
Structural acceleration Sa (m/sec2)	0.021	0.03	0.03	0.03	0.034	0.024	0.29
Structural Displacement Sd (mm)	140	127	124	124	121	118	23
Base shear V (KN)	3098.631	4112.829	4570.966	3929.130	4880.009	3583.732	50080.335
Roof Displacement D (mm)	188	224	191	177	185	162	26
A-B	7261	6911	6166	6670	7254	7267	6441
B-IO	127	242	1044	429	579	420	1385
IO-LS	431	330	325	439	423	993	212
LS-CP	1229	960	909	1038	756	370	57
CP-C	0	0	0	0	0	0	4
C-D	0	0	2	1	2	0	1
D-E	12	17	14	3	14	10	0
>E	0	0	0	0	0	0	0

Table 6:-Performance point parameter for building models along Transverse direction

IV. CONCLUSION

1. Fundamental natural period decreases when effect concrete core wall is considered.
2. Storey drifts are found within the limit as specified by code (IS 1893-2002 Part-1) in both linear and dynamic and non-linear static analysis.
3. Bay wide and storey height shear wall can be effectively used in reducing the dynamic response of a structure.
4. The addition of shear walls for lateral strength increases the structural stiffness which in turn increases the spectral acceleration sa/g value in models of building.
5. The behaviour of properly detailed reinforced concrete frame building is adequate of demand and capacity curves and the distribution of hinges in the beams and the columns. Most of hinges developed in the beams and few in the columns but with limited damage.
6. The result obtained in terms of performance point and plastic hinges gave on insight into the real behaviour of structures.
7. Base shear at first hinge is less and displacement at first hinge is more for bare frame model and vice versa for other models.
8. Ductility ratio is maximum bare frame structure and it get reduced when the effect of shear wall is considered. It indicates that these structure will show adequate warning before collapse.
9. Bare frame structure are having highest response reduction factor as compared to other models. It indicates that bare frame structure are capable of resisting the forces still after first hinges.
10. In case of core-wall structure it can be seen that almost all hinges are formed in link-beams. To function properly under severe earthquake loading, the core-wall requires ductile link beams that can undergo large in-elastic deformation.

11. For the above study we conclude that model 7 i.e., bare frame with shear wall at corner plus lift core wall shows better performance among the others for given seismic parameters.

V. SCOPE FOR FURTHER STUDY

Further studies can be conducted that on sky scrappers, composite structures, Studies can be conducted by providing dual system, which consists of shear wall (or braced frame) and moment resisting frame such that the two systems are designed to resist the total design force in proportion to their lateral stiffness considering the interaction of dual system at all floor levels. The moment resisting frames may be designed to independently resist at least 25% of design seismic base shear. For better ductility beam-column junction study can also be made. Various damping mechanisms and its applications on structures can also be studied. Studies also on existing building can be considered for evaluation. Where, a preliminary investigation using FEMA-273 can be done before evaluation of the existing building using mathematical modeling with the help of FEA package and further it can be evaluated using Non-Linear Dynamic Analysis. Conventional approach to earthquake resistant design of buildings depends upon providing the building with strength, stiffness and inelastic deformation capacity. But the new techniques like Energy Dissipation and Active Control Devices are a lot more efficient and better.

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