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Analytic Study of Multistorey Steel Building's Reserve Capacity by Pushover Analysis

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Abstract- *As India seeks to attain the status of a developed country, its per capita steel consumption is on the rise. Already steel structural systems are being used for many buildings in preference to conventional RCC construction. This project work envisages the study of behaviour of steel moment resistant frame through response spectrum analysis (elastic analysis) and pushover analysis (non-linear static analysis). As far as steel structures are concerned, they have inherent better ductility compared to RCC structures. Therefore, employing only elastic analysis and arriving at the design values with associated partial safety factors will not provide a clear picture of the reserve capacity of the structure while under the action of seismic loads. Since collapse prevention is the prime attribute of an earthquake resistant structure, determination of collapse loads vis-a-vis the design loads assumes special significance. This project work makes an attempt to arrive at the reserve capacities of the moment resistant steel frames for heights ranging from G+3 to G+15. The structure is initially designed and optimized for response spectrum loads under elastic conditions. For structural members either Indian standard steel sections or built-up steel sections are used. The results of the analyses indicates that the reserve capacity ratio in terms of base shear reduces from 10.7 to 9.0 as the storey height increases from G+3 to G+6; however beyond G+6 up to G+15 it increases steadily reaching a maximum value of 22.0 for G+15; this observation leads to a significant conclusion that at lesser heights the structures are more vulnerable to collapse than at increased heights.*

Keywords: *Steel structure; RCC Construction; response spectrum analysis; earthquake resistant structure; pushover analysis; etc.*

Introduction

Nonlinear static analysis, or pushover analysis, has been developed over the past twenty years and has become the well-liked analysis procedure for design and seismic performance evaluation purposes because the procedure is comparatively simple and considers post elastic behaviour. However, the procedure involves certain approximations and simplifications that some amount of variation is usually expected to exist in seismic demand prediction of pushover analysis.

As traditional pushover analysis is widely used for design and seismic performance evaluation purposes, its limitations, weaknesses and therefore the accuracy of its predictions in routine application should be identified by studying the factors affecting the pushover predictions. In other words, the applicability of pushover analysis in predicting seismic demands should be investigated for low, mid and high-rise structures by identifying certain issues like modelling nonlinear member behaviour, computational scheme of the procedure, variations within the predictions of varied lateral load patterns utilized in traditional pushover analysis, efficiency of invariant lateral load patterns in representing higher mode effects and accurate estimation of target displacement at which seismic demand prediction of pushover procedure is performed.

Although, in literature, pushover analysis has been shown to capture essential structural response characteristics under seismic action, the accuracy and therefore the reliability of pushover analysis in predicting global and native seismic demands for all structures are a topic of dialogue and improved pushover procedures are proposed to beat the certain limitations of traditional pushover procedures.

1.1 Objectives of the study:

This dissertation aims to study the dynamic behaviour of multi storey steel buildings under seismic loads using STAAD pro V8i.The principle objectives of the study are as follows:

a. Generation of 3 dimensional models of different storey buildings to analyze dynamic and pushover analysis using STAAD pro V8i.

b. To study the dynamic behaviour of elastic analysis and inelastic analysis.

c. Comparison of response spectrum analysis and pushover analysis.

d. Comparison of displacement profiles, base shear and first mode behaviour for response spectrum analysis and pushover analysis and to find the reserve capacity.

II. Literature Review

Structures are expected to deform in elastically when subjected to severe earthquakes, so seismic performance evaluation of structures should be conducted considering post-elastic behaviour. Therefore, a nonlinear analysis procedure must be used for evaluation purpose as post-elastic behaviour cannot be determined directly by an elastic analysis. Moreover, maximum inelastic displacement demand of structures should be determined to adequately estimate the seismically induced demands on structures that exhibit inelastic behaviour.

Krawinkler and Seneviratna (1998) conducted a detailed study that discusses the advantages, disadvantages and the applicability of pushover analysis by considering various aspects of the procedure. The basic concepts and main assumptions on which the pushover analysis is based, target displacement estimation of MDOF structure through equivalent SDOF domain and the applied modification factors, importance of lateral load pattern on pushover predictions, the conditions under which pushover predictions are adequate or not and the information obtained from pushover analysis were identified. The accuracy of pushover predictions were evaluated on a 4-story steel perimeter frame damaged in 1994 Northridge earthquake. The frame was subjected to nine ground motion records. Local and global seismic demands were calculated from pushover analysis results at the target displacement associated with the individual records. The comparison of pushover and nonlinear dynamic analysis results showed that pushover analysis provides good predictions of seismic demands for low-rise structures having uniform distribution of inelastic behaviour over the height. It was also recommended to implement pushover analysis with caution and judgment considering its many limitations since the method is approximate in nature and it contains many unresolved issues that need to be investigated.

Mwafy and Elnashai (2001) performed a series of pushover analyses and incremental dynamic collapse analyses to investigate the validity and the applicability of pushover analysis. Twelve reinforced concrete buildings with different structural systems (four 8-story irregular frame, four 12-story regular frame and four 8-story dual frame-wall), with different design accelerations (0.15g and 0.30g) and with different design ductility levels (low, medium and high) were utilized for the study. Nonlinear dynamic analysis using four natural and four artificial earthquake records scaled to peak ground accelerations of 0.15g and 0.30g were performed on detailed 2D models of the structures considering predefined local and global collapse limits. Also, pushover analyses using uniform, triangular and multimodal load patterns were conducted and pushover curves were obtained. The results showed that the triangular load pattern outcomes were in good correlation with dynamic analysis results and a conservative prediction of capacity and a reasonable estimation of deformation were obtained using triangular load pattern. It was also noted that pushover analysis is more appropriate for low-rise and short period structures and triangular loading is adequate to predict the response of such structures. Further developments on accounting the inelasticity of lateral load patterns which would enable more accurate analysis of high-rise and highly irregular structures were recommended.

Inel, Tjhin and Aschheim (2003) conducted a study to evaluate the accuracy of various lateral load patterns used in current pushover analysis procedures. First mode, inverted triangular, rectangular, "code", adaptive lateral load patterns and multimode pushover analysis were studied. Pushover analyses using the indicated lateral load patterns were performed on four buildings consisting of 3- and 9-story regular steel moment resisting frames designed as a part of SAC joint venture (FEMA-355C) and modified versions of these buildings with a weak first story. Peak values of story displacement, understory drift, story shear and overturning moment obtained from pushover analyses at different values of peak roof drifts representing elastic and various degrees of nonlinear response were compared to those obtained from nonlinear dynamic analysis. Nonlinear dynamic analyses were performed using 11 ground motion records selected from Pacific Earthquake Research Center (PEER) strong motion database. Approximate upper bounds of error for each lateral load pattern with respect to mean dynamic response were reported to illustrate the trends in the accuracy of load patterns. Simplified inelastic procedures were found to provide very good estimates of peak displacement response for both regular and weak-story buildings. However, the estimates of understory drift, story shear and overturning moment were generally improved when multiple modes were considered. The results also indicated that simplifications in the first mode lateral load pattern can be made without an appreciable loss of accuracy

Viviane Warnotte (1998) summarized basic concepts on which the seismic pounding effect occurs between adjacent buildings. He identified the conditions under which the seismic pounding will occur between buildings and adequate information and, perhaps more importantly, pounding situation analyzed. From his research it was found that an elastic model cannot predict correctly the behaviors of the structure due to seismic pounding. Therefore non-elastic analysis is to be done to predict the required seismic gap between buildings.

III. Methods of Seismic Analysis

For seismic performance evaluation, a structural analysis of the mathematical model of the structure is required to work out force and displacement demands in various components of the structure. The seismic performance of the structures can be determined in elastic manner or inelastic manner by the help of various methods.

3.1 Methods of Seismic Analysis of a Structure

The two main systems currently used for this analysis are,

- Elastic analysis / Linear Dynamic Analysis such as Response spectrum analysis
- Inelastic analysis / Non-Linear Dynamic Analysis such as pushover analysis

Elastic analysis methods include code static lateral force procedure, code dynamic procedure and elastic procedure using demand-capacity ratios. These methods also are referred to as force-based procedures which assume that structures respond elastically to earthquakes.

In code dynamic procedure, force demands on various components are determined by an elastic dynamic analysis. The dynamic analysis could also be either a response spectroscopy or an elastic time history analysis. Sufficient number of modes must be considered to have a mass participation of at least 90% for response spectrum analysis. Any effects of upper modes are automatically included in time history analysis. In demand/capacity ratio (DCR) procedure, the force actions are evaluated to corresponding capacities as the ratio of demand vs. capacity. Demands for DCR calculations must include gravity effects. While code static lateral force and code dynamic procedures reduce the complete earthquake demand by an R-factor, the DCR approach takes the complete earthquake demand without reduction and adds it to the gravity demands. If DCR has a value higher than 1.0, it indicates deficiencies.

The response spectrum technique is basically a simplified special case of modal analysis. The modes of vibration are determined in period and shape in the usual way and the maximum response magnitudes corresponding to each mode are found by reference to a response spectrum. The response spectrum method has the good virtues of speed and cheapness. The basic mode superposition method, which is restricted to linearly elastic analysis, produces the entire time history response of joint displacements and member forces thanks to a selected ground motion loading. There are two major drawbacks of this approach. First, the tactic produces an outsized amount of output information which will require a huge amount of computational effort to conduct all possible design checks as a function of your time. Second, the analysis must be repeated for several different earthquake motions so as to assure that each one the many modes are excited, since a response spectrum for one earthquake, in a specified direction, isn't a smooth function.

The inelastic time history analysis is that the most accurate method to predict the force and deformation demands at various components of the structure. However, the utilization of inelastic time history analysis is restricted because the dynamic response is extremely sensitive to modeling and ground motion characteristics. It requires proper modelling of cyclic load-deformation characteristics considering deterioration properties of all important components. Also, it requires the availability of a group of representative ground motion records that accounts for uncertainties and differences in severity, frequency, and duration characteristics. Moreover, computation time, the time required for input preparation and interpreting voluminous output makes the utilization of inelastic time history analysis impractical for seismic performance evaluation.

The non-linear static procedure or just push-over analysis may be a simple option for estimating the strength capacity within the post-elastic range. This procedure involves applying a predefined lateral load pattern which is distributed along with the building height. The lateral forces are then monotonically increased in constant proportion with a displacement control node of the building until a specific level of deformation is reached. The applied base shear and therefore the associated lateral displacement at each load increment are plotted. Based on the capacity curve, a target displacement which is an estimate of the displacement that the planning earthquake will produce on the building is decided. The extent of injury experienced by the building at this target displacement is taken into account as a representative of the damage experienced by the building when subjected to style-level ground shaking.

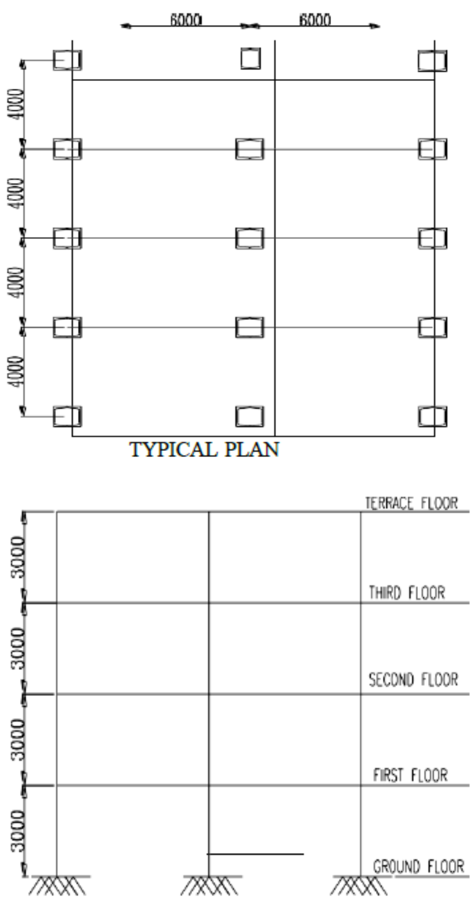
IV. Structural Modeling and Analysis

In order to evaluate the reserve capacity of buildings with different stories the analysis software, STAAD Pro is utilized to create 3D model and run all analyses. The software is in a position to predict the geometric nonlinear behaviour of space frames under static or dynamic loadings, taking into consideration both geometric nonlinearity and material inelasticity. This software takes static as well as dynamic load variables along with performing Eigen values, nonlinear pushover and dynamic linear analysis.

4.1 Details of the Models

In the present dissertation, we have considered five different models. The structural framing system consists of moment-resistant steel frames with Indian standard sections or built-up sections. The joints are rigid and the columns are treated as fixed at the foundation. The type of building considered is that which can be used as an administrative building.

1. Three storey (G+3) buildings
2. Six storey (G+6) buildings
3. Nine storey (G+9) buildings
4. Twelve storey (G+12) buildings
5. Fifteen storey (G+15) buildings



ELEVATION G+3

Fig-1: Plan and elevation of the G+3 model buildings.

Table-1: Properties of Structural Members for Beams and Columns

No of storeys	Column member	Beam members		
		6 m span	4 m span Inner	4 m span outer
G+3	ISMB 600	ISMB 500	ISMB 400	ISMB 300
G+6	ISMB 600A	ISWB 500	ISWB 400	ISMB 350
G+9	W14 X 120	ISWB 550	ISWB 450	ISWB 400
G+12	W14 X 159	ISWB 550	ISWB 450	ISWB 400
G+15	W14 X 211	ISWB 550	ISWB 450	ISWB 400

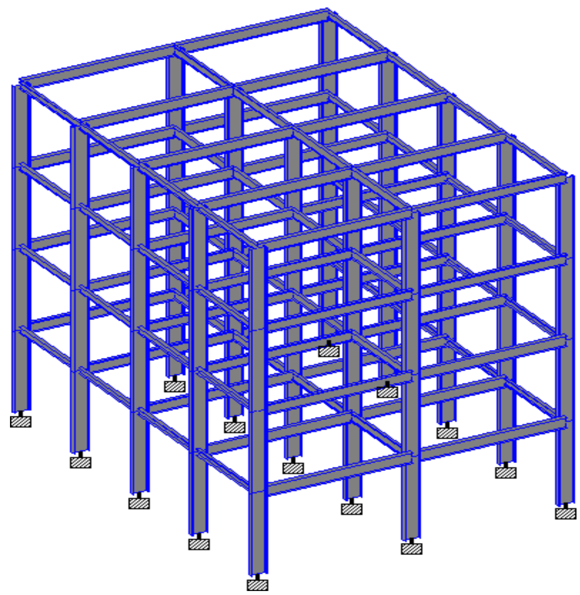


Fig-2: 3-D View of the Three Storey (G+3) Building Created in STAAD.

4.2 Assigning Loads

- Dead load of the structure is applied, which includes self weight of all members (beams, columns, slabs, walls).
- The wall loads are applied as continuous loads or UDLs on the beams.
- Live loads are taken as per IS code, for floors 2kN/m2 and for roof 1.5 kN/m2.

- The wind pressure shall be calculated based on the basic wind speed and other provisions laid in IS 875 (part 3) – 1987. The basic wind speed being considered for the specified location is 50 m/sec.
- Chennai is classified under earthquake zone III (moderate) as per the latest classification. The loading due to earthquake is assessed based on the provisions of IS 1893-2002

4.3 Analysis of Structure
The analysis procedures have been carried out for determining the various structural parameters of the model. The focus is to determine the behaviour of the structure under the effect of ground motion and dynamic excitations such as earthquakes in elastic range and inelastic range.
The analyses carried out are as follows:
➢ Response Spectrum Analysis
➢ Pushover Analysis.

Response Spectrum Analysis: Base shear for response spectrum analysis as per IS 1893:2002, the total design lateral force or design seismic base shear (V_b) is determined by the following expression.

$$V_b = A_h W$$

Where A_h is the design horizontal acceleration spectrum value using fundamental natural period and W is the seismic weight of the building. The seismic weight is estimated based on full dead load + 25% live load.

The fundamental natural period of vibration (Ta) in seconds of steel buildings estimated by using the following formula for infill walls

$$T_a = 0.09h/ (\sqrt{d})$$

Where 'h' is that the height of the building in meters, d is that the base dimension of the building at the plinth level in m, along the considered direction of the lateral force.

Pushover Analysis: Pushover analysis in STAAD may be a static, non-linear procedure in accordance with FEMA 356 specification. In this method, the lateral push load magnitude is actually increased according to a predefined loading pattern until either loading or the deflection reaches the described level.

General steps to be followed for performing Pushover Analysis in STAAD are described below:
➢ Define Steel Moment and Braced Frames
➢ Define Gravity Loading
➢ Define Lateral (Push) Loading
- Static load pattern
- Base shear to be distributed vertically

➢ Define Primary elements
➢ Define Pushover Hinges Properties and Acceptance Criteria
➢ Define Pushover Analysis Solution Control
➢ Define Input for Demand Spectrum
➢ Performance

V. Results and Discussion

STAAD PRO V8i is used to compute the response of a three storey (G+3), six storey (G+6), nine storey (G+9), twelve storey (G+12) and fifteen storey (G+15) for linear dynamic analysis (response spectrum), and nonlinear analysis (static push over analysis). Results from response Spectrum analysis are observed for the base shear, fundamental time period and maximum lateral displacement. Results from pushover analysis have been used to observe and compare the floor responses of different models. Pushover curves and capacity spectrum curves results have been used to observe and compare the displacement and base shear of the different storey buildings.

5.1 Analysis and Results of Three Storey Buildings (G+3)
The Analysis for G+3 building has been carried out for response spectrum and Pushover analysis which is detailed below:

RESPONSE SPECTRUM ANALYSIS
Response spectroscopy has been administered as per the response spectra mentioned in IS 1893(part1) 2002. The displacements for a particular joint at the top floor and base shear are given in Tables 5.1 and 5.2

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Table-2: Displacement Values for Lateral Loads for G+3

Direction	Node	Load case	X	Y	Z	Resultantmm
Max X	73	1.0 (DL+LL+EX)	11.3	-1.5	0.0	11.4
Min X	75	1.0 (DL+LL-EX)	-11.3	-1.8	0.0	11.4
Max Y	134	SEISMIC Z-DIR	0.9	0.2	57.3	57.3
Min Y	104	1.0 (DL+LL-EZ)	0.0	-2.7	-57.3	57.3
Max Z	13	1.0 (DL+LL+EZ)	1.0	-1.0	60.9	61.0
Min Z	133	1.0 (DL+LL-EZ)	-0.9	-1.3	-60.9	61.0
Max Rst	133	1.0 (DL+LL-EZ)	-0.9	-1.3	-60.9	61.0

Table-3: Maximum Displacement and Base shear for G+3

Storey	Maximum Displacement	Base shear
G + 3	60 mm	318 kN

PUSHOVER ANALYSIS
Pushover analysis has been carried out as per the FEMA 356: 2000 and ATC 40. The displacements and base shear are given in Table 5.3

Table-4: Displacement and Base Shear for G+3

Load Step	Displacement mm	Base Shear kN
1	0	0
2	108.03	2241.575
3	144.644	3001.174
4	160.861	3283.955
5	168.326	3404.94
6	168.545	3408.489

Pushover Curves for Three Storey Building
Pushover curve may be a plot of base shear vs. roof displacement (V vs. D). It is also known as capacity curve. This curve gives idea about the bottom shear induced within the structure at performance point. The pushover curves for different lateral load cases for G + 3 storey buildings are plotted and are shown in Figure. 5.1.

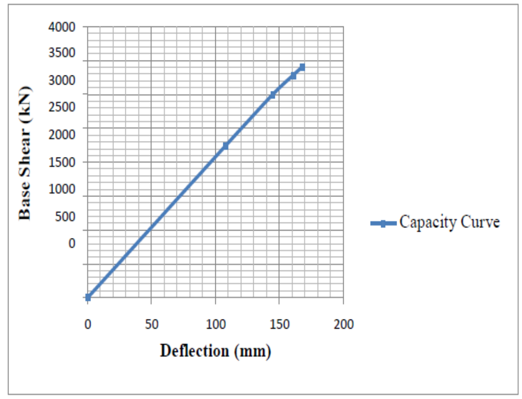

Fig-3: Pushover Curve for G+3 Storey Building.

5.2 Analysis & Results of Six Storey Buildings (G+6)
The Analysis for G+6 building has been carried out for response spectrum and Pushover analysis which is detailed below:

RESPONSE SPECTRUM ANALYSIS
Response spectrum analysis has been carried out as per the response spectra mentioned in IS 1893 (part1) 2002. The displacements for a particular joint at the top floor and base shear are given in Table 5.4 and 5.5

Table-5: Max Displacement Values for Lateral Loads G+6

Direction	Node	Load case	X	Y	Z	Resultant mm
Max X	172	1.0(D.L+L.L-W.L.X)	18.5	-3.5	0.0	18.9
Min X	174	1.0(DL+LL-EX)	-15.0	-4.1	0.0	15.6
Max Y	167	SEISMIC Z-DIR	4.9	0.5	79.2	79.4
Min Y	176	1.0(DL+LL-EZ)	-1.3	-5.1	-79.2	79.4
Max Z	168	1.0(DL+LL+EZ)	4.8	-2.4	75.2	75.2
Min Z	180	1.0(DL+LL-EZ)	-4.9	-3.2	-75.2	-75.2
Max Rst	180	1.0(DL+LL-EZ)	-4.9	-3.2	-75.2	-75.2

Table-6: Displacement and Base Shear G+6

Storey	Maximum Displacement	Base shear
G + 6	75 mm	562 kN

PUSHOVER ANALYSIS

Pushover analysis has been carried out as per the FEMA 356: 2000 and ATC 40. The displacements and base shear are given in Table 5.6

Table-7: Displacement and Base Shear for G+6

Load Step	Displacement mm	Base Shear kN
1	0	0
2	85.892	1994.651
3	131.624	3056.657
4	185.788	4306.665
5	207.266	4798.593
6	207.266	560.556

Pushover Curves for Six Storey Building

Pushover curve may be a plot of base shear vs. roof displacement (V vs. D). It is also known as capacity curve. This curve gives idea about the bottom shear induced within the structure at performance point. The pushover curves for different lateral load cases for rigid floor idealization for G +6 storey buildings are plotted and are shown in Figure. 5.2

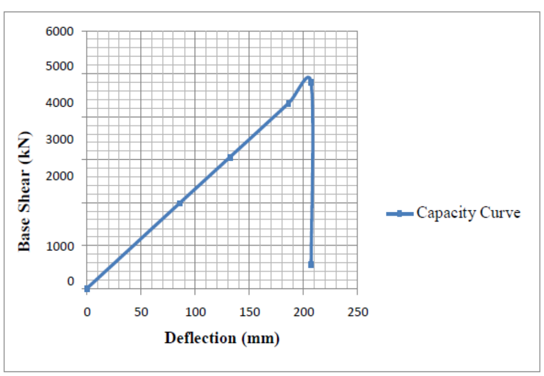

Fig-4: Pushover Curve for G+6 Storey Building.

5.3 Analysis & Results Of Nine Storey Buildings (G+9)

The Analysis for G+9 building has been carried out for response spectrum and Pushover analysis which is detailed below

RESPONSE SPECTRUM ANALYSIS

Response spectrum analysis has been carried out as per the response spectra mentioned in IS 1893(part1) 2002. The displacements for a particular joint at the top floor and base shear are given in Table 5.7 and 5.8

Table-8: Max Displacement Values for Lateral Loads for G+9

Direction	Node	Load case	X	Y	Z	Resultant mm
Max X	217	1.0(D.L+L.L-W.L.X)	40.0	-6.3	0.0	40.5
Min X	222	1.0(DL+LL-EX)	-20.8	-7.4	-1.0	22.1
Max Y	217	WIND LOAD -IVE X	40.0	0.8	0.0	40.0
Min Y	218	1.0(DL+LL-EZ)	0.0	-9.3	-33.4	34.7
Max Z	224	WIND LOAD -IVE Z	0.0	-0.7	50.3	50.3
Min Z	223	1.0(DL+LL-EZ)	-2.3	-5.9	-35.4	36.0
Max Rst	218	1.0(D.L+L.L-W.L.Z)	0.0	-9.3	50.3	51.1

Table-9: Displacement and Base Shear for G+9

Storey	Maximum Displacement	Base shear
G + 9	50 mm	577 kN

PUSHOVER ANALYSIS

Pushover analysis has been carried out as per the FEMA 356: 2000 and ATC 40. The displacements and base shear have been tabulated as below-

Table-10: Displacement and Base Shear for G+9

Load Step	Displacement mm	Base Shear kN
1	0	0
2	30.748	1445.05
3	101.717	4780.412
4	164.186	7715.757
5	168.07	7895.873
6	168.07	3958.711

Pushover Curves for Nine Storey Building

Pushover curve is a plot of base shear vs. roof displacement (V vs. D). It is also known as capacity curve. This curve gives idea about the base shear induced in thestructure at performance point. The pushover curves for different lateral load cases for rigid floor idealization for G +9 storey buildings are plotted and are shown in Figure. 5.3.

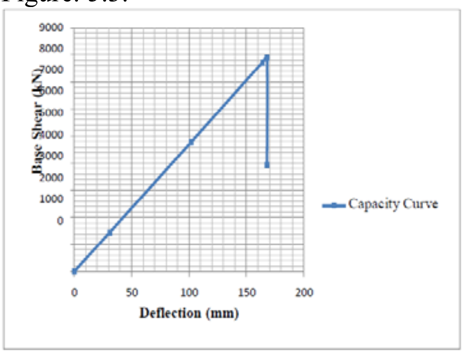

Fig-5: Pushover Curve for G+9 Storey Building.

5.4 Analysis & Results of Twelve Storey Buildings (G+12)

The Analysis for G+12 building has been carried out for response spectrum and Pushover analysis which is detailed below

RESPONSE SPECTRUM ANALYSIS

Response spectrum analysis has been carried out as per the response spectra mentioned in IS 1893(part1) 2002. The displacements for a particular joint at the top floor and base shear are given in Table 5.10 and 5.11

Table-11: Max Displacement Values for Lateral Loads for G+12

Direction	Node	Load case	X	Y	Z	Resultant mm
Max X	262	1.0(D.L+L.L-W.L.X)	56.8	-8.0	0.0	57.4
Min X	264	1.0(DL+LL-EX)	-26.1	-9.7	-0.7	27.8
Max Y	262	WIND LOAD -IVE X	56.7	1.1	0.0	56.7
Min Y	263	1.0(DL+LL-EX)	-26.0	11.4	0.0	28.4
Max Z	257	1.0(D.L+L.L-W.L.Z)	0.0	-8.4	65.2	65.7
Min Z	270	1.0(DL+LL-EZ)	-3.8	-7.8	-42.8	43.7
Max Rst	263	1.0(D.L+L.L-W.L.Z)	0.0	11.4	65.1	66.1

Table-12: Displacement and Base Shear for G+12

Storey	Maximum Displacement	Base shear
G + 12	65 mm	586 kN

PUSHOVER ANALYSIS

Pushover analysis has been carried out as per the FEMA 356: 2000 and ATC 40. The displacements and base shear are given in Table 5.12

Table-13: Displacement and Base Shear for G+12

Load Step	Displacement mm	Base Shear kN
1	0	0
2	43.297	1885.452
3	209.673	9130.662
4	231.368	10012.655
5	242.527	10448.718
6	242.612	10452.037

Pushover Curves for Twelve Storey Building

Pushover curve is a plot of base shear vs. roof displacement (V vs. D). It is also known as capacity curve. This curve gives idea about the base shear induced in thestructure at performance point. The pushover curves for different lateral load cases for rigid floor idealization for G +12 storey buildings are plotted and are shown in Figure. 5.4

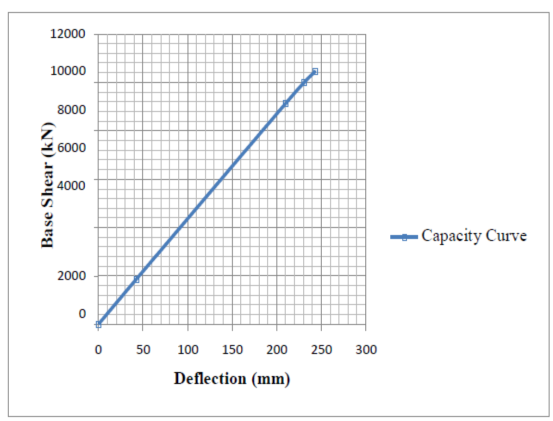

Fig-6: Pushover Curve for G+12 Storey Building.

5.5 Analysis & Results of Fifteen Storey Buildings (G+15)

The Analysis for G+15 building has been carried out for response spectrum and Pushover analysis which is detailed below

RESPONSE SPECTRUM ANALYSIS

Response spectrum analysis has been carried out as per the response spectra mentioned in IS 1893(part1) 2002. The displacements for a particular joint at the top floor and base shear are given in Table 5.13 and 5.14

Table-14: Max Displacement Values for Lateral Loads for G+15

Direction	Node	Load case	X	Y	Z	Resultant mm
Max X	307	1.0(D.L+L.L-W.L.X)	88.1	-8.8	0.0	88.6
Min X	306	1.0(DL+LL-EX)	-33.1	11.0	-1.1	34.9
Max Y	307	WIND LOAD -IVE X	88.1	1.7	0.0	88.1
Min Y	308	1.0(DL+LL-EZ)	0.0	12.8	-47.8	49.4
Max Z	302	1.0(D.L+L.L-W.L.Z)	0.0	-9.2	97.5	98.0
Min Z	315	1.0(DL+LL-EZ)	-9.4	-9.2	-56.9	58.4
Max Rst	308	1.0(D.L+L.L-W.L.Z)	0.0	12.7	97.5	98.3

Table-15: Displacement and Base Shear for G+15

Storey	Maximum Displacement	Base shear
G + 15	98 mm	625 kN

PUSHOVER ANALYSIS

Pushover analysis has been carried out as per the FEMA 356: 2000 and ATC 40. The displacements and base shear is given in Table 5.15

Table-16: Displacement and Base Shear

Load Step	Displacement mm	Base Shear kN
1	0	0
2	58.228	2327.585
3	155.643	6221.617
4	258.337	10320.246
5	273.876	10895.868
6	289.616	11435.126
7	306.277	11933.794
8	364.797	13682.785
9	364.797	587.136

Pushover Curves for Fifteen Storey Building

Pushover curve is a plot of base shear vs. roof displacement (V vs. D). It is also known as capacity curve.

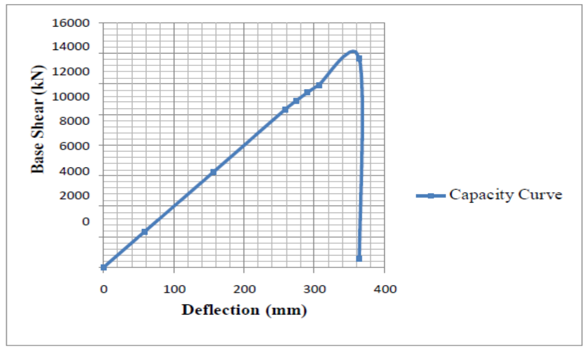

Fig-7: Pushover Curve for G+15 Storey Building.

5.6 Comparison of Response Spectrum and Pushover Analysis

Figure 5.6 represents that comparison of variation of base shear as obtained for response spectrum analysis and push over analysis. While the push over analysis yields a variation which steeply increases as the height increases, the base shear obtained from response spectrum analysis does not display such steep variation.

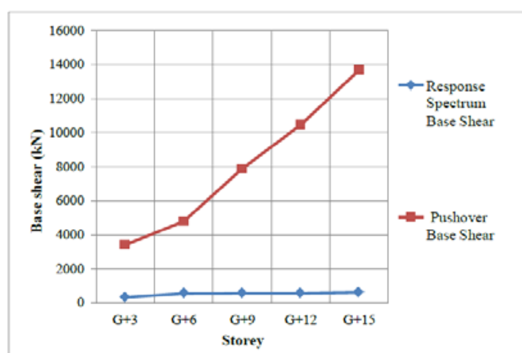

Fig-8: Comparison of Base Shear for Response Spectrum Vs Pushover Analysis.

Figure 5.7 represents the deflection pattern obtained in both the cases. Interestingly there is a reduction in lateral displacements the height increases from G+6 to G+9. which can be attributable, in the case of elastic analysis, to the phenomena that in this range of height, there is a sudden drop in base shear characterized by the drooping portion of the response spectrum curve; however in the case of push over analysis the different stages of plastic hinge formation cause this drop in displacement.

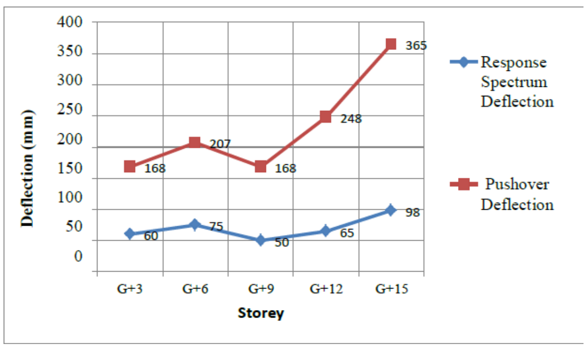

Fig-9: Comparison of Deflection for Response Spectrum Vs Pushover Analysis.

The reserve capacities of the structure are shown in Figure.5.8. The Figure indicates that the reserve capacity increases with height of the building except between G+3 and G+6 which leads to the inference that at lesser heights the structures are more vulnerable to collapse than at increased heights.

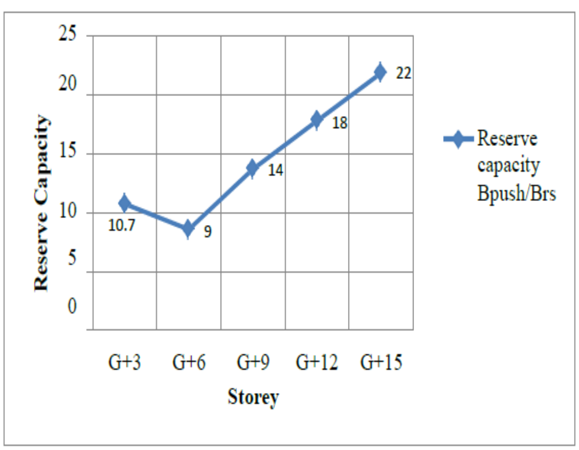

Fig-10: Reserve Capacity for Base shear.

5.7 Stage-Wise Plastic Hinge Formation and Mechanism

Table-17: Plastic Hinge Location and Status

Beam	Status	Dir (Local)	Section	Status	Section	Status
83	Non linear	Y	0	LS - CP		
158	Non linear	Y	0	IO - LS	4	IO - LS
233	Non linear	Y	0	IO - LS	4	IO - LS
308	Non linear	Y	0	IO - LS		

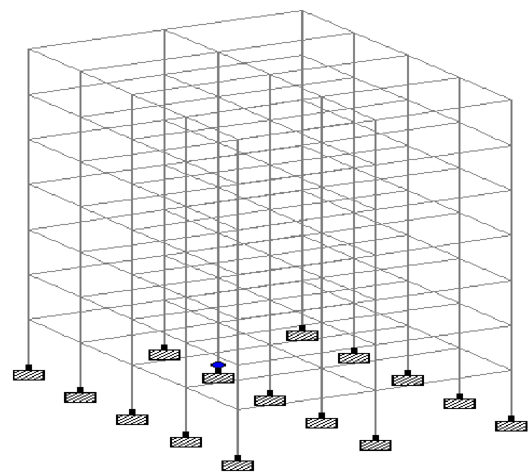

Fig-11: Plastic Hinge Formations at Immediate Occupancy.

VI. Conclusion

In this dissertation work moment resistant steel frames forming part of the structural system of a typical institutional building of height ranging from G+3 to G+15 have been analysed by response spectrum method (elastic analysis) and pushover analysis (non-linear static analysis). The structural behaviour under these analyses has been studied through the parameters of base shear, fundamental time period, displacement and reserve capacity.

- The reserve capacity ratio in terms of base shear reduces from 10.7 to 9.0 as the storey height increases from G+3 to G+6. However beyond G+6 upto G+15 it increases steadily reaching a maximum value of 22.0 for G+15. This observation leads to a significant conclusion that at lesser heights the structures are more vulnerable to collapse than at increased heights. Similar trend is observed for maximum displacement also; but the slope of the curve is milder compared to that of base shear reserve capacity.

- Between G+3 and G+6 the moment resistant frames behave predominantly in shear mode with reduced overall structure ductility resulting in reduced reserve capacity.
- The push over analysis shows the step by step plastic hinge formation till mechanism is attained which help selective strengthening of identified members for further increasing the collapse load.

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