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A STUDY ON PUSHOVER ANALISIS OF REINFORCED CONCRETE FRAME STRUCTURES

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ABSTRACT

An earthquake resistant building is one that has been deliberately designed in such a way that it remains safe and suffers no appreciable damage during destructive earthquake. However, it has been that during past earthquakes many of the buildings were collapsed due to failure of members. To model the complex behavior of reinforced concrete analytically in its non-linear zone is difficult. This has led engineers in the past to rely heavily on empirical formulas which were derived from numerous experiments for the design of reinforced concrete structures. For structural design and assessment of reinforced concrete members, the non-linear analysis has become an important tool. The method can be used to study the behavior of reinforced concrete structures including force redistribution.

During severe earthquakes, the structures undergo deformations well in excess of their yield deformations and consequently suffer damage at certain locations. The location and degree of damage reflects the performance of the structure as well as its non-structural components. Consequently a deformation-based analysis and design is more rational and meaningful than a force-based analysis and design. Such an analysis helps in evaluating the location and degree of damage and thus evaluates the performance of the structure. This analysis of the nonlinear response of RC structures to be carried out in a routine fashion. It helps in the investigation of the behavior of the structure under different loading conditions, its load deflection behavior and the cracks pattern.

In the present study, the non-linear response of RCC frame using SAP2000 under the loading has been carried out with the intention to investigate the relative importance of several factors in the non-linear analysis of RCC frames.

I. INTRODUCTION

The structures experience deformations during strong earthquakes that are far greater than their yield deformations, which causes damage in some areas. The area and severity of damage are indicators of both the structural and non-structural components of the structure's performance. As a result, evaluation and design based on deformation are more logical and significant than evaluation and design according to force. Such an examination assists in determining the extent and location of damage, which evaluates the structure's functionality.

Recent earthquakes, such as the 2001 Bhuj earthquake, in which numerous concrete structures suffered extensive harm and collapsed, have highlighted the necessity to assess the seismic suitability of existing structures. Growing concern is being expressed in particular over the seismic rehabilitation of existing concrete structures in seismically active regions since it is necessary to identify vulnerable structures and establish an acceptable degree of safety. Simplified linear-elastic approaches are insufficient for such an assessment.

The non-linear static examination, formerly called as the Pushover analysis (POA), sometimes called the collapse analysis, is looked at a practical way to evaluate performance even though other methods are feasible. The technique is employed in this study to assess the effectiveness of RC plane frames. Due to population growth, reinforced concrete (RC) frame structures are turning into a growing trend in metropolitan India, where safety is of utmost importance.

The POA, also known as nonlinear static analysis, has been established over the last twenty years and has since replaced other analysis techniques as the method of choice for seismic and design performance assessment since it is post-elastic behavior-aware and reasonably straightforward. However, because of the technique's approximations and simplified concepts, some variation is always anticipated in the POA seismic demand estimate.

1.1 ORGANIZATION OF THE THESIS

In the first chapter, a brief description on introduction to the project, Organization of the Thesis is presented. Second chapter describe on literature review. Third chapter addresses with the Objectives, Scope and modeling of RC plane frames, the designs of RC plane frames as per IS codal provisions, loads were considered. In the fourth chapter, contains POA procedure for RC frames. Fifth chapter contains evaluation of results and the last chapter contains the summary and conclusions. At the end of the report, references are presented.

II. REVIEW OF LITERATURE

A thorough study by Krawinkler and Seneviratna, which takes into account many different facets of the process, examines the benefits, drawbacks, and application of POA. The fundamental ideas and fundamental premises upon which the POA is predicated have been determined along with the target displacement forecasting of the MDOF structure through equivalent SDOF domain and the utilised modifying factors. The significance of the lateral load pattern on POA, the circumstances under which POA are adequate or not, and the data gleaned from the POA.

A four-story steel border frame was damaged by the Northridge earthquake in 1994 was used to test the precision of pushover forecasts. a total of 9 ground motion records were applied to the frame. Pushover outcomes at the target displacement linked with the individual records were used to compute both global and local seismic demands. POA offers accurate forecasts of seismic demands for low-rise structures with a homogeneous distribution of inelastic behaviour over the height, according to the results of a comparison between pushover and nonlinear dynamical evaluation. Furthermore, given the method's numerous limitations, including its approximate nature and a number of unresolved problems, it was advised that POA be used with caution and good judgment.

In order to examine the relevance and validity of POA, Mwafy and Elnashai conducted a incremental dynamic collapse and number of POA investigations. For the study, 12 reinforced concrete structures with various structural systems such as 4 (eight-story irregular frames), 4 (Twelve-story regular frames), and eight-story dual frame-wall with various design accelerations (0.15g, and 0.30g), and various design ductility stages (low rise, medium rise, and high rise) were used. On intricate 2D models of the buildings, a nonlinear dynamical study was conducted using 4 natural and 4 artificial earthquake recordings scaled to peak ground accelerations of 0.15g and 0.30g while taking into account specified global and local collapse limits. Then, utilising the outcomes of the nonlinear dynamic studies, regression analyses were carried out toprovide whole pushover-like load-displacement curves for every structure in the form of both the lower and upper response enclosures as well as the finest fit. Pushover curves were also developed after POA utilizing triangular, multimodal load, and uniform patterns. The findings demonstrated that the triangular load pattern outputs had a strong association with the findings of the dynamic evaluation and that utilising the triangular load pattern, conservative predictions of capability and appropriate estimates of deformation could be made. Additionally, it was discovered that triangular loading is sufficient to forecast the reactivity of low-rise, short-period buildings while POA is more suitable for these types of structures. It was suggested that additional research be done on accounting for the inelasticity of lateral load patterns so that high-rise and extremely irregular constructions might be analyzed with greater accuracy.

Several researchers have proposed adaptive load patterns because unchanging lateral load patterns cannot forecast greater mode effects in the post-elastic range and cannot account for the redistribution of inertial forces. Eberhard and Sozen recommended utilizing load patterns based on mode shapes obtained from secant stiffness at every load phase.

The accurateness of different lateral load patterns employed in POA methodologies was examined in a study by Nel, Tjhin, and Aschheim. The "first mode, rectangular, and inverted triangular, "code ", adaptable multiple modes and lateral load patterns in POA were investigated. Four buildings with three and nine story unvarying steel moment-resistant frames that were part of the SAC joint venture and adapted versions of these buildings with a weak 1 story were subjected to POA utilizing the indicated lateral load patterns. Peak values of peak roof drift reflecting elastic and different degrees of non-linear reactivity were related to those derived from nonlinear dynamic investigation for inter-story drift, story displacement, overturning moment, and story shear.

To pinpoint failure mechanisms brought on by higher modes, Sasaki et al, developed the Multi-Mode Pushover (MMP) approach. In addition to the load pattern based on basic mode, the technique also uses separate patterns of load based on higher modes. For each load pattern, a POA is done, and a capacity curve is created while taking the modes of interest into consideration. Employing the capacity of a structure and Capacity Spectrum method for each of the modes is compared to the demand for earthquakes. The points at the intersection of the capacity spectra and the response spectra, which are presented on the same graph in ADRS format, represent the seismic demand on the building's structure. Using MMP, the 1994Northridge earthquake-damaged 17-story steel frame and the 1989 Loma Prieta earthquake- damaged 12-story steel frame were both assessed. POA using only the first mode load pattern for both frames was unable to determine the real damage. However, the distribution of real damage was more accurately matched by the pushover results of higher modes or the influence of the combined type such as first mode and higher modes. According to the study's findings, MMP can be effective in locating failure causes resulting from higher modes in buildings that exhibit considerable higher-order modal responses.

Although MMP is highly helpful for qualitatively identifying the influence of higher modes, it is unable to estimate seismic reactions and distribute them throughout the structure. To measure the consequences of greater mode responses in tall structures, Moghadam suggested a method. Structures are subjected to a series of POA employing elastic mode shapes as the load pattern. The combined responses from the individual POA are used to estimate the highest seismic responses. The suggested combining rule states that each mode's reaction is multiplied by the mode's mass involvement factor, and each mode's response is then added together.

To evaluate the technique's accuracy, it was used on a twenty-story steel moment- resistant frame. The frame was exposed to 6 earthquake ground movements, and in a total of six investigations, it was determined the mean of the greatest displacements and the inter-story drift ratios of each story. Additionally, pushover studies for the frame's first three modes were run, and the estimates for each mode's responses were merged. There was a high connection between calculated inter-story drifts and displacements and the average of the highest responses obtained from 6 non-linear dynamic analyses.

The contrast of the data showed that POA for all load patterns significantly underestimating the requirements for story drift and causes significant inaccuracies in the rotation of plastic hinges. When determining floor displacements, plastic hinge position, narrative drifts, and plastic hinge rotations, the MPOA performed better than all POA. Based on the outcomes from El Centro ground motion scaled by factors range from 0.25 to 3.0, MPOA results were also demonstrated to be minimally reliant on ground motion intensity. It was determined that the height-wise variance in responses estimated by MPOA is often comparable to the 'precise' outcomes from nonlinear dynamic analysis by taking into account the contributions of an adequate amount of modes (two or three).

III. METHODOLOGY

3.1 GENERAL

A structure's frame model is an assembly of structural parts, such as columns, and beams, that depicts the structural characteristics of a typical structure's frame and shows how it responds to loading from the outside. The ideal analytical model should capture the strength, mass distribution, deformability, and stiffness of the object under a variety of global and local displacements. This chapter covers the modeling of 1 to 4-story RC space frames.

3.2 OBJECTIVES

1. To investigate the effectiveness of RC space frames when subjected to lateral loads during earthquakes.
2. POA will be used to examine the inelastic response of RC Space frames.
3. To investigate the variation in peak response, capacity curves, and hinge locations of the structure using the capacity spectrum approach on various RC frames under identical loading.

POA on various space frames was done with the above goals in mind. Following space frame patterns were taken into consideration.

No-bracing space frame.

1) A space frame with bracing..

i) Pattern of bracing frames –A.

ii) Pattern of bracing pattern –B.

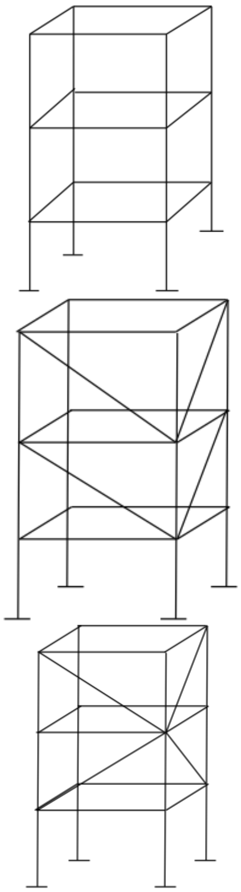


Fig 3.1 Different patterns of frames
Space frame without bracing Space frame bracing pattern-A Space frame bracing pattern-B

The study also takes into account the number of floors (i.e., up to the fourth floor). One, two, three, and four levels of RC space frames, both with and without bracing, were created. Using SAP2000, the sizes of various pieces were designed in accordance with IS 456: 2000. The beam's dimensions were 230x300, and 25 mm were used as the reinforcement's cover. The column's dimensions were 300x300 mm, with 40 mm serving as the cover for the reinforcement. only at the top of the structure are lateral loads imposed. These dimensions were modified to fit the POA space frame.

3.1 SCOPE

1. Only multi-story Space frames were taken into account.
2. Structures with shear walls or unconventional plans are not taken into account.
3. A non-linear static method called POA is utilized to forecast the way the RC frames would actually perform when subjected to lateral loads.

Using SAP2000, one, two, three, and four storeys of RC plane ground frames—without bracing and with bracing—were modelled and analysed. All elements that influence the strength, stiffness and mass, of the frame, are represented by the numerical model.

3.1 MATERIALS

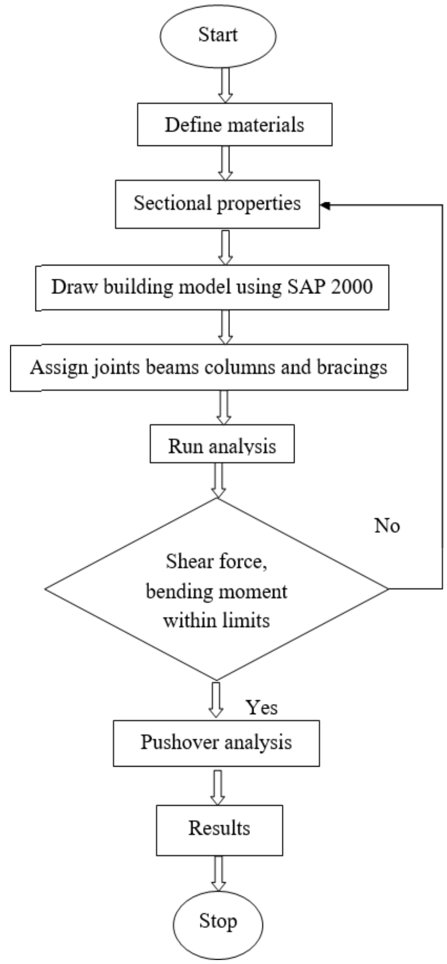
According to IS 456:2000, reinforced concrete's elasticity modulus is given by

$$E_c = 5000\sqrt{f_{ck}}$$

The data needed for steel rebar includes elastic modulus, ultimate strength, and yield stress. High-yield strength deformed bars (HYSD) with a yield strength of 415 N/mm2 were chosen for the current investigation since they are commonly employed in design practice.

3.6 METHODOLOGY

With and without bracing, one-story, two-story, three-story, and four-story RC space frames of the ground were evaluated and developed for gravity loads in accordance with IS 456:2000 and lateral loads (seismic loads) in accordance with IS 1893 (part-1):2002. To determine the response for the largest considered earthquake in Zone V, the frames were subjected to POA. Initially, the frames were analysed as plane frames.



PUSHOVER ANALYSIS OF FRAMES

4.1 GENERAL

In the constructions in order to prevent earthquake damages and strengthen the stiffness and lateral strength of the members additional arrangements must be implemented. According to IS 1893 (part-1): 2002, dynamic analysis (linear or non-linear) of buildings is done, and this analysis takes into account the effects of stiffness and lateral strength as well as the inelastic deformations in the parts. According to IS 1893 (part-1): 2002, the lateral stresses brought on by earthquakes were calculated using the Response spectrum approach.

4.1 CALCULATION OF BASE SHEAR

According to clause 7.5.3 of IS 1893:2002 (referred to as the Code), the total design seismic base shear or design lateral force (VB) is determined.

The total Base shear

$$V_B = A_h W$$

where Ah denotes the horizontal seismic design coefficient

$$A_h = \left(\frac{Z}{2}\right) \frac{I}{R} \frac{S_a}{g}$$

Here

R = Response Reduction Factor, I = Importance Factor,Z = Zone Factor

As per IS 1893 (part-1):2002, Tables 2, 6, and 7 provide the values for Z, I, and R, respectively. Sa/g = Coefficient of Spectral acceleration. It is calculated in accordance with Clause 6.4.5 of the Code and is provided as follows in seconds for the basic time period Ta.

For without infill, a Moment Resisting Frame

$$T_a = 0.075\ h^{0.75}$$

For a Resisting Frame with brick infill panels

$$T_a = \frac{0.09\ h}{\sqrt{d}}$$

Here

h denotes building Height a Building Frame

d = Base measurement of the structure in metres, measured along the lateral load's intended direction. Since it was built using ordinary moment resisting frames (OMRF) and the IS456- 2000 standard, factor 3 was taken into account while calculating the base force seismic zone.

IV. RESULTS

EVALUATION RESULTS

The POA outputs include the deflected shape, the development of the hinge performance, and hinges with increasing load levels at the performance point. At the performance point (or the point where the POA ends), the number of hinges produced in the columns and beams is calculated, together with their performance levels.

The demand and capacity spectra curves, pushover curve, and their values are among the other outcomes of the pushover study. The inelastic displacement and the base shear capacity of the roof are revealed by the pushover curve. The roof displacement at ultimate base shear to the roof displacement at the start of yielding can be used to calculate a material's global ductility. The performance point's existence was established using the demand and capacity spectrum curves. The structure is unable to reach the desired performance level if the performance point is missing.

Pushover results for Ground floor space frame

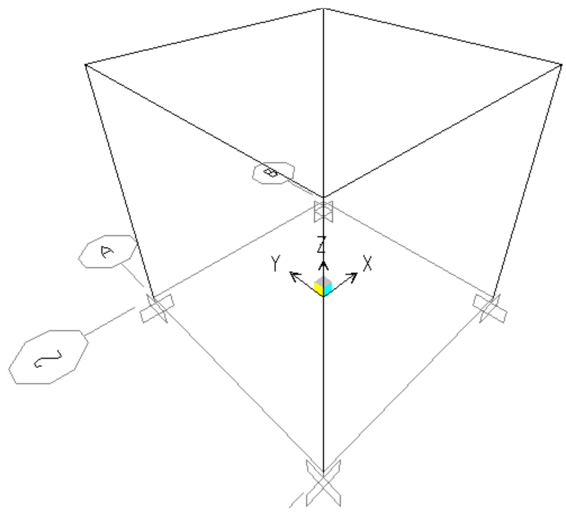


Fig 5.1 Ground floor space frame structure

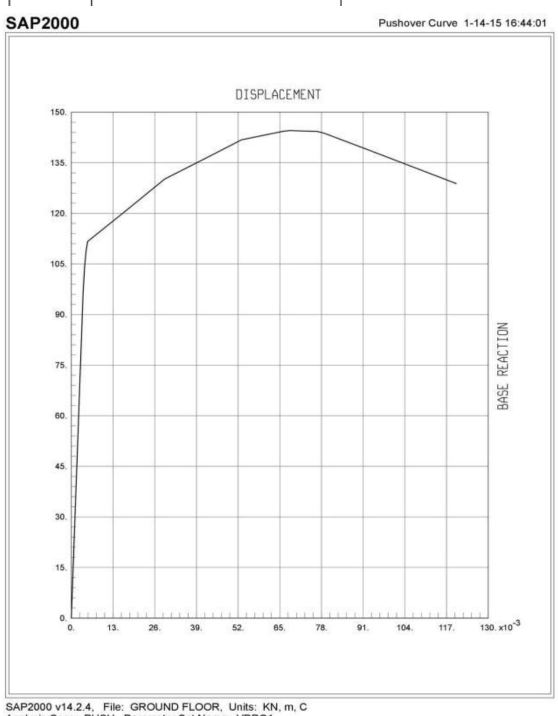
Results of the ground floor space frame POA are displayed in the tables and graphs below. The displacement versus baseforce values for lateral loads applied at the top of the ground floor space frame are shown in Table 1. The graph in Graph 1 compares displacement to base force.

Maximum displacement was 0.068 m at maximum base force of 144.53 KN, and maximum displacement was 0.12 m at maximum base force of 128.82 KN. The graph in Figure 2 shows the performance point-awarded relationship between spectral acceleration and spectral displacement.

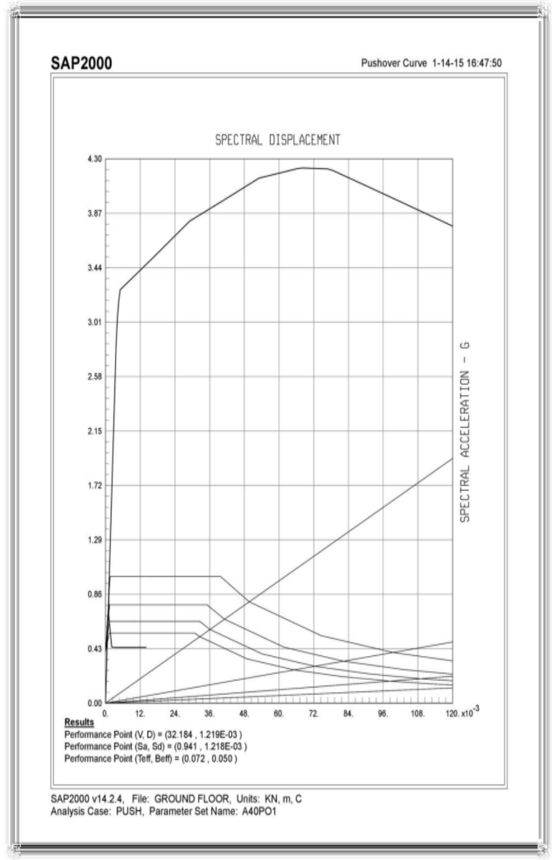
The performance point's spectral displacement was 0.00121 m and its spectral acceleration was 0.941 m/s2.

TABLE: 1 Base force vs Displacement for Ground floor.

Pushover Curve Demand - FEMA356 – PUSH		
Step	Displacement(d)	Base Force(KN)
0	0	0
1	0.003652	96.475
2	0.004112	103.863
3	0.004582	108.801
4	0.005089	111.651
5	0.017089	120.811
6	0.029089	130.189
7	0.041089	135.952
8	0.053089	141.795
9	0.065089	144.122
10	0.066293	144.352
11	0.068033	144.530
12	0.076658	144.281
13	0.078571	143.848
14	0.090571	139.551
15	0.102571	135.194
16	0.114571	130.826
17	0.12	128.828



Graph 1 Pushover Curve for Ground floor



Graph 2 Pushover Curve Demand Capacity - ATC40 –Ground floor

Pushover results for First floor space frame

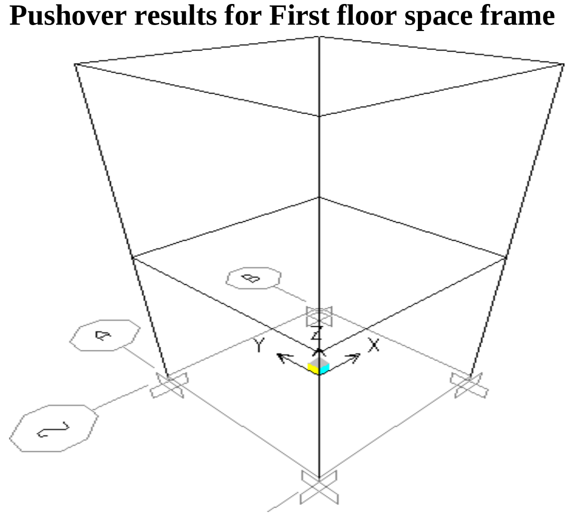


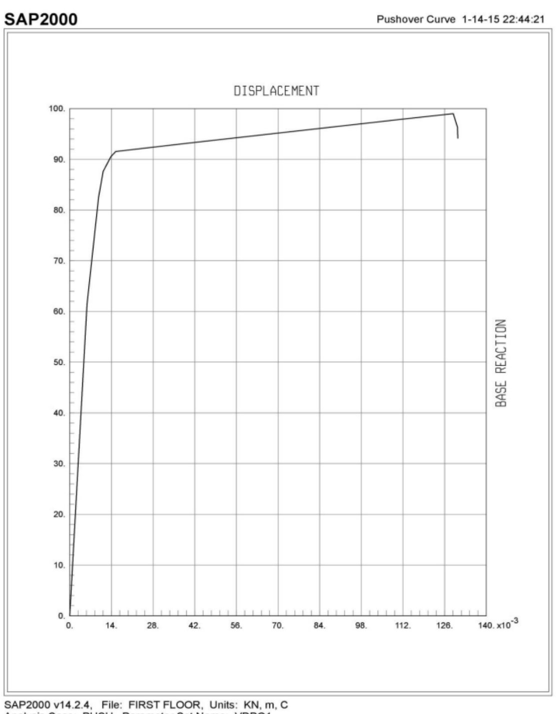
Fig 5.2 First floor frame structure

Results of the first floor space frame POA are displayed in the tables and graphs below. When the lateral loads are applied at the top of the first

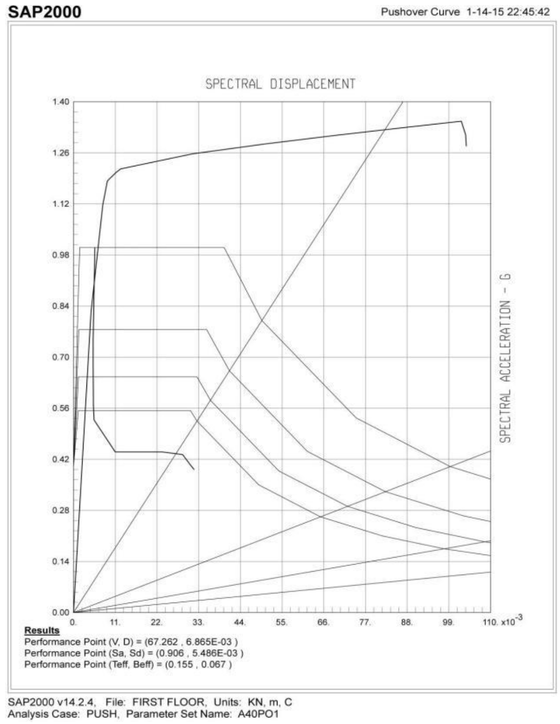
floor space frame, displacement against baseforce values are shown in Table 2. The graph in Graph 3 compares displacement to base force. Maximum displacement was 0.13 m at maximum base force of 94.155 KN and highest displacement was 0.128 m at maximum base force of 99 KN. The graph in Figure 4 shows the performance point-awarded relationship between spectral acceleration and spectral displacement
n. The performance point's spectral displacement was 0.0054 m and its spectral acceleration was 0.906 m/s2.

Pushover Curve Demand - FEMA356 – PUSH		
Step	**Displacement(d)**	**BaseForce(KN)**
0	0	0
1	0.005776	61.37
2	0.009693	82.559
3	0.011209	87.629
4	0.013874	90.567
5	0.015473	91.525
6	0.039473	93.137
7	0.063473	94.733
8	0.087473	96.312
9	0.111473	97.877
10	0.128883	99
11	0.130284	96.362
12	0.13045	94.155

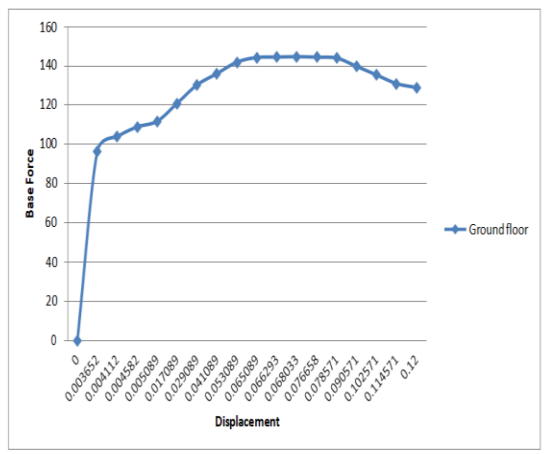
TABLE: 2 Base force vs Displacement for First floor



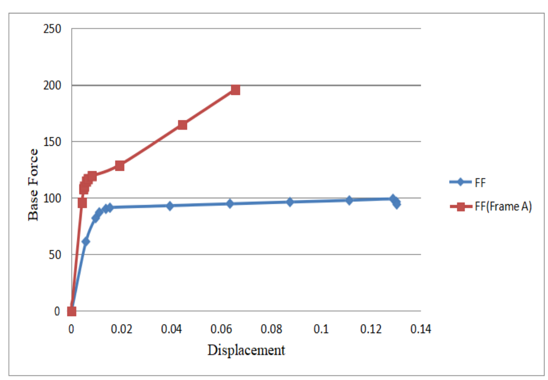
Graph 3 Pushover Curve for First floor



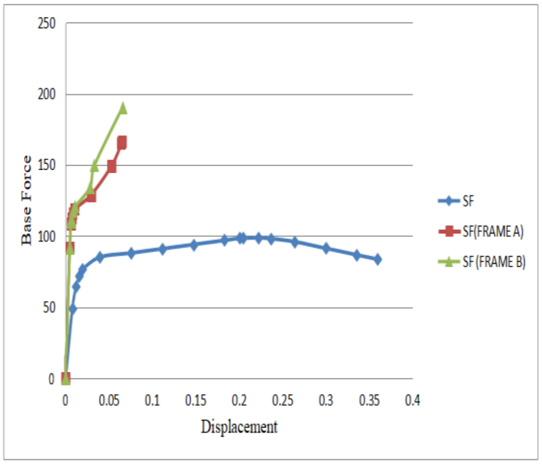
Graph 4 Pushover Curve Demand Capacity - ATC40 – for First floor



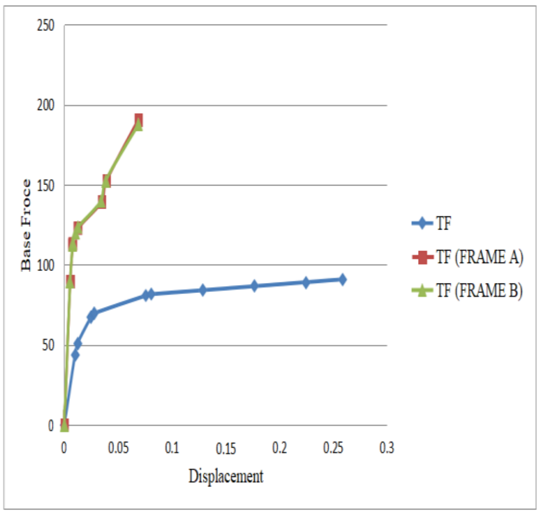
Graph 25 GROUND FLOOR



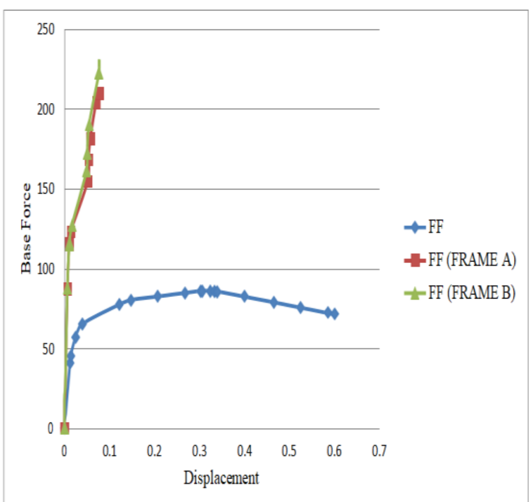
Graph 26 FIRST FLOOR



Graph 27 SECOND FLOOR



Graph 28 THIRD FLOOR



Graph 29 FOURTH FLOOR

Table 13 Performance Pionts Corresponding to Different Floors

V. RESULTS AND CONCLUSIONS

DISCUSSION OF THE RESULTS

WITHOUT BRACING FRAME STRUCTURE

- As the number of stories rises, thus rises the number of hinges.
- As the number of stories rises, the displacement at the top of the frame also rises.
- As the number of stories rises, the performance point shifts to the right and moves higher to a certain point before declining.

FRAME STRUCTURE WITH BRACING (FRAME A)

- The amount of hinges created when the floor is raised remains approximately the same.
- The base force keeps rising as the number of levels rises.
- As the number of floors rises, displacement at the top of the frame continues to rise.
- Increasing the number of stories caused the performance point to shift to the right and upward.

CONCLUSIONS

For frames without a bracing structure, the number of hinges rises as the number of stories increases.

- The amount of hinges created by raising the floor is the same for frames with bracing.
- Base Force rises as the proportion of floors with and without bracing patterns of frames rises.
- Peak pushover load reaches its maximum value when the base reaction abruptly decreases and then increases in displacement without increasing base reaction.
- Compared to frames without bracing, displacement is quite low.
- As the number of stories for frames with bracing structure increases, there is a rightward movement higher in the performance point.

- Due to their lower displacements at their maximum Base Force, Frame B structures are superior to other frame structures.

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