**Effect of Stiffness Modifier on Predicted Seismic Performance of RC Buildings**

Nishant Kumar1, Ramanand Dubey2

1Student, Department of Earthquake Engineering, IIT Roorkee, Roorkee

*[nishant\_k@eq.iitr.ac.in](mailto:nishant_k@eq.iitr.ac.in)*

2Professor, Department of Earthquake Engineering, IIT Roorkee, Roorkee  
[rn.dubey@eq.iitr.ac.in](mailto:rn.dubey@eq.iitr.ac.in)

**Abstract.** In current practices, concrete cracking is considered to reduce the stiffness of the member. Stiffness modifiers are used in linear analysis to account for the cracking of concrete and the slipping of bonding between steel and concrete. Before the inception of the stiffness modifier, designers used the entire stiffness section, which yielded more reinforcement in members, resulting in an uneconomical design and sometimes contradicting the strong column and weak beam concepts. Therefore, the structure's performance must be verified using newly advanced techniques such as performance-based design based on FEMA. The introduction of stiffness modifiers makes the structure more flexible, increasing the structure's time period and attracting less seismic forces in the member. Many researchers and codes provide the recommended modified stiffness modifiers for RC Structures, but none have provided the value according to the height. The main objective of the present work is to compare the seismic performance and damage of different heights of structures with and without stiffness modifier. Pushover analysis has been carried out for the nonlinear static analysis. Fragility analysis was used to check the damage to the structures. The percentage change in drift decreases with the increase in storeys. Increasing the number of storeys also results in a 25-percentage improvement in performance points. Fragility curve and damage probability matrices have given that, after using the stiffness modifier, the performance of the building improves and experiences lesser damage from complete to extensive in high stories and extensive to slight in low stories. The study recommends classifying stiffness modifiers according to the structure's height.

**Keywords:** Stiffness modifier; Seismic performance; Pushover curve; Performance point; Fragility curve.

# Introduction

Reinforced concrete has been used and is still used as the primary building material in most engineered structures to gain strength and durability. Concrete is used for its workability, ease of handling, long life and low maintenance. Plain concrete is strong in compression but weak in tension [1]. Application of an even smaller load results in failure of the tension side before the compression side achieves its full strength. This failure problem is solved by placing reinforcements in concrete members on the tension side of the members. However, these reinforcements only start taking loads after the concrete cracking begins. The cracking of concrete results in stiffness reduction and the moment-carrying capacity of the member [2]. It also redistributes the moment to the uncracked concrete member. This effect of concrete is tackled by using the stiffness modifiers as mentioned in the various codes around the world.

Stiffness modifiers are the factors that minimize the stiffness in concrete sections and simulate the cracking pattern of concrete. Branson defines the effective moment of inertia for cracked concrete [3]. Designing any member or structure is mainly based on the forces (moment, shear, axial) obtained by analysing the structure. These forces depend upon the stiffness of the member [4]. The stiffer the member, the more forces will be attracted. For this reason, we need to design the members for added reinforcement, which results in higher costs. The extent of the cracking is still being determined, and it is also impossible to detail the load distribution after cracking. The increment in loading increases the cracking and reduces the moment of inertia of the section [5]. The change in the moment of inertia concerning loading makes the nonlinear deflection equation. A nonlinear deflection equation prohibits using the superposition method for load combination. Nonlinear analysis is required to predict the extent of cracking, which is time-consuming for linear analysis [6].

There are two methods to consider the extent of cracking in the analysis and design of the structure. One is the advanced nonlinear theory, and the other is a direct approach using the precise values of reduced stiffness of cracked members. The effective moment of inertia proposed by different researchers [6] is used for the member-wise design. IS 1893 (Part 1):2016 provided the values for stiffness modifiers in clause 6.4.3.1. The global lateral response used the reduced modifier according to different country codes. In past practices, it has been seen that uncracked stiffness increases the time period, design moment and shear of structure.

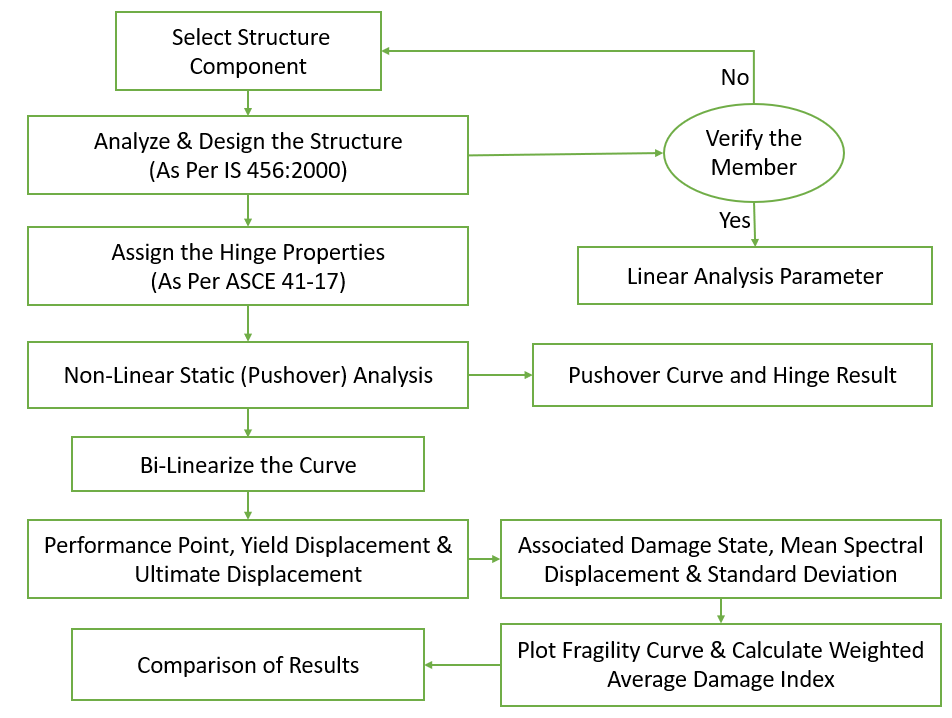
Many researchers compare the performance of structures with and without stiffness modifiers [7-10]. None of them were studied by varying the Height of the Structure. Nonlinear static analysis, particularly pushover analysis, has become the preferred method for assessing the seismic performance of structures with stiffness modifiers. This approach allows for evaluating a structure's performance point, which is critical for performance-based design. [11] investigated the efficacy of a building according to its seismic performance, economic loss, and seismic resilience in terms of the decrement in column stiffness. [12] investigated the seismic performance of the building using the SBC code. The author also compared the performance of three different methods: capacity spectrum, displacement method and modified displacement method.

Fragility analysis is another methodology that has gained prominence in recent years. This probabilistic approach assesses the likelihood of different levels of damage occurring under varying seismic intensities. [13] briefly describe seismic vulnerability, damage and risk evaluation. The capacity spectrum method and how to obtain fragility curves are briefly discussed. [14] studied the damaged state of gravity load and designed multi-level frame building by developing the capacity curve, fragility curve, and damage probability matrices.

In the present study, the effect of the Stiffness modifier on the seismic performance and seismic damage is seen for different structure heights. Different storey models have been analysed and designed according to IS 456:2000. Pushover analysis was carried out to obtain the structure's seismic performance and capacity curve. Further, the capacity curve has been bilinearized as per FEMA 356. The performance point of the structure is obtained by the Displacement method as per FEMA 356. Fragility analysis was used to develop the fragility curve based on HAZUS methodology. Damage probability matrices are formed at the performance point to obtain the damage state of the structure.

# Methodology

The methodology adopted in this study includes a combination of nonlinear static analysis (pushover analysis) and fragility analysis to assess the seismic performance of RC buildings with varying heights. Linear analysis methods, such as response spectrum analysis, help assess structures' fundamental periods and mode shapes but fall short in capturing the nonlinear behaviour during seismic events.



**Fig. 1**  Methodology Flow Chart

Pushover analysis has emerged as a preferred method for nonlinear assessment, allowing engineers to evaluate a structure's performance point—a critical factor in performance-based seismic design. Research by [15] demonstrated the efficiency of pushover analysis in predicting RC buildings' seismic responses, mainly when stiffness modifiers are applied.

Fragility analysis provides a probabilistic approach to assessing damage risks under different seismic intensities. Studies by [16] showed that fragility curves could effectively predict potential damage in structures with modified stiffness, offering valuable insights for risk assessment and mitigation. Fig. 1 shows the Methodology flow chart.

## Pushover Analysis

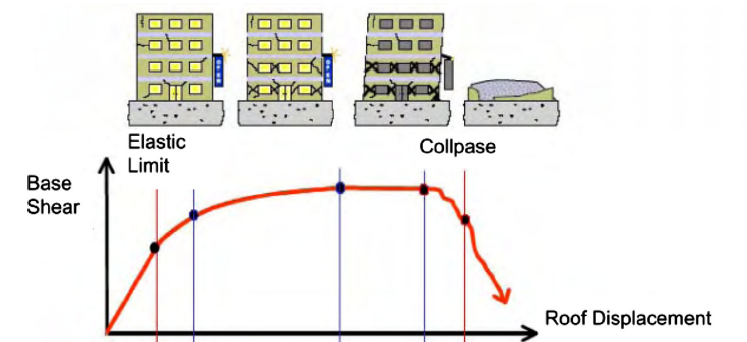
The linear analysis method is the simplest and most common structure analysis approach. This method applies a response reduction factor to lower the seismic forces according to the limit state design. Structural deflection is managed through the ductility provided by ductile reinforcement detailing. However, this elastic analysis has limitations, as it often fails to account for stiffness changes, P-delta effects, and sometimes the failure mechanisms of structures, particularly when inelastic behaviour, need to be accurately considered.

On the other hand, Pushover analysis assumes that the structural response can be approximated by a single-degree-of-freedom (SDOF) system, where a single mode dominates the response, and the shape remains constant. While this assumption is inaccurate, studies have shown that a single mode can often dominate in a multi-degree-of-freedom (MDOF) system, leading to more reliable results. Pushover analysis directly predicted the amount and location of plastic yielding in the structure. This method is generally used to obtain the ultimate building capacity of the structure. It estimates the nonlinear response of the structure by demonstrating the progressive failure in the structure. It also finds potential weak areas in the structure by obtaining the sequential damages in each member, called hinges.

## Nonlinear Plastic Hinges

The performance of a building is analyzed using two parameters: hazard level and performance level. It also provides the force deformation curve for critical sections of members in the structure, as depicted in Fig. 2. The points A, B, C, D, and E on the pushover curve describe the deflected action of hinges. Point A is an unloading condition that moves as a linear response towards point B, which is effective yielding. Then, from B, the structure's stiffness reduces till point C, followed by a sudden drop in lateral resistance till point D, with a final loss of resistance at point E. Line CD depicts the initial failure of the structure and line DE shows the residual strength in the structure. These points specified according to FEMA and at the intermediate level are described as Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) between points B and C. The Performance Level of structure concerning IO, LS, and CP is given in Fig. 3.

A diagram of a triangle with lines and arrows

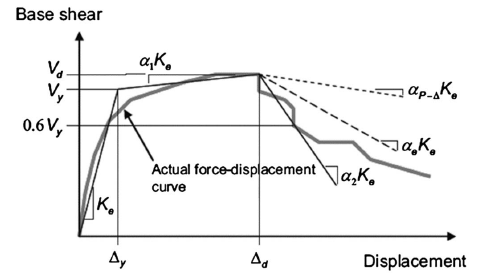
Description automatically generated

**Fig. 2** FEMA 356 Performance Level (Source: Hakim, 2014, p. 7692)

**Fig. 3** Component or Element Acceptance Criteria (Source: FEMA 356, p. 2-15)

## Bilinearization of Pushover Curve

The pushover curve is the plot of roof displacement vs base shear. This nonlinear curve is replaced with the bilinear curve with yield displacement, yield strength, ultimate strength and ultimate displacement. This conversion process is according to section 7.4.3.2.4 of ASCE 41-17. A bilinear curve is given in Fig. 4. The first line segment has a slope of effective lateral stiffness Ke and originates from the origin. Ke shall be taken as secant stiffness at 60% of effective yield strength Vy. Vy should be less than the maximum base shear force at any point in the curve. The second line segment represents the positive post-yield slope determined by Vd. Vd and point of intersection with the first line are such that the area above and below the actual curve is the same. A third line segment is a negative post-yield slope between Vd and the point at which the base shear degrades to 60% of effective yield strength.



**Fig. 4** Idealized Force Deformation Curve (Source: FEMA 356, p. 3-20)

## Building Performance point

Building Performance is obtained using the target displacement for a specific hazard level. The structure is pushed to calculate target displacement. There are three methods to obtain target Displacement. These methods are the capacity spectrum method (ATC 40), Displacement method (FEMA 356) and displacement modification method (FEMA 440). In the current study, we use the displacement method.

## Capacity Spectrum Method

This method is also called the ATC 40 method. This method changes the capacity and demand curve into spectral acceleration (Sa) vs spectral displacement (Sd), which is acceleration displacement response spectrum (ADRS) format. First, calculate the modal participation factor (Г1) and modal mass coefficient (α1), provided in equations 1 and 2. Fig. 5 is a graphical representation of the capacity spectrum method.

Г1 = (1)

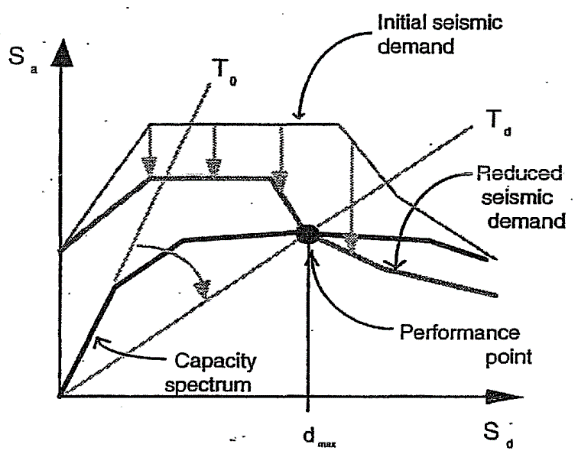
α1 = (2)

Then spectral acceleration (Sa) and spectral displacement (Sd) for every point on the curve are calculated, and the Sa v/s T curve is converted into the Sa v/s Sd curve using given equations (3), (4), and (5). A bilinear capacity curve estimates effective damping and reduction in demand. First, assume the trial performance point. If the reduced spectrum curve intersects the capacity curve at the trial performance point, then that point is our result; else, repeat the same exercise with new trial values.

(3)

(4)

(5)



**Fig. 5** Performance Point Calculation as per Capacity Spectrum Method (Source: ATC 40, p. 2-18)

## Displacement Method

This method is given by FEMA 356. It modifies the elastic response from the SDOF system into coefficients C0, C1, C2, and C3 and obtains the target displacement as equation 6.

Target Displacement = (6)

Where C0 is Modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building, C1 is Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response, C2 is Modification factor representing the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response, C3 is Modification factor to represent increased displacements due to dynamic P-Δ effects, Te is Effective fundamental period of the building in the direction under consideration. The values of C0 and C2 are available in FEMA 356, and C1 and C3 calculate from equations 7-9.

C1 = 1.0 for Te ≥ Ts

= for Ts < Te (7)

C3 = (8)

R = (9)

Ts is the characteristic period of the response spectrum, R is the ratio of elastic strength demand to calculated yield strength coefficient, Sa is response spectrum acceleration at the building's effective fundamental period and damping ratio in the direction under consideration., *g* is the acceleration of gravity, Vy is Yield strength of the buildings, W is Effective seismic weight, and Cm is Effective mass factor.

## Displacement modification factor

It is a modified displacement method produced by the FEMA 440. The equation of target displacement is similar to that used in the displacement method except for changes in C1 and C2 factors as per equations 10 and 11.

C1 = (10)

C2 = (11)

R is the ratio of elastic strength demand v/s calculated yield strength coefficient (130, 90, and 60) for site categories B, C, and D, respectively.

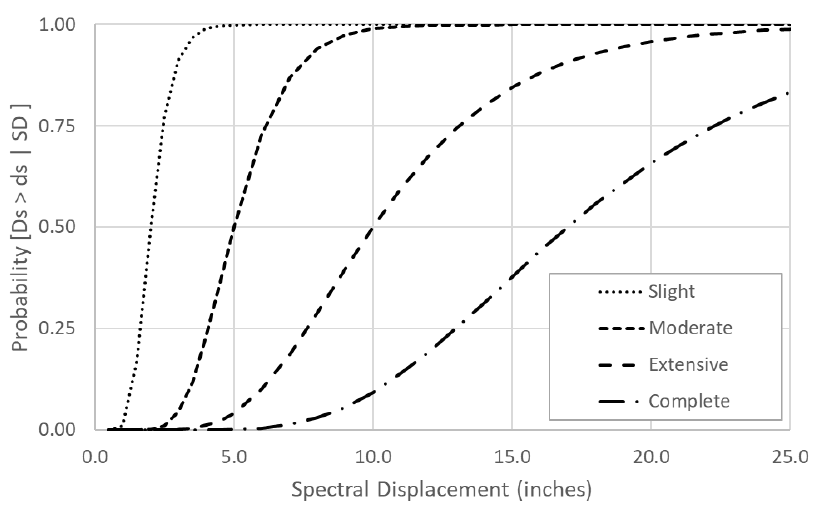
## Fragility Analysis

The main aim of fragility analysis is to obtain the probability of a given level of damage due to an earthquake. This probabilistic relationship is formed between structural damage and ground motion parameters (peak ground acceleration or spectral displacement). The vulnerability function and damage probability matrices represent the probability of damage.

The fragility curve gives the probability of the structure limiting or exceeding a given damage state as a function of a given seismic parameter and follows the lognormal cumulative distribution. The damage state has a logarithmic standard deviation and median value of the seismic parameter. In the present study, spectral displacement is taken as a seismic parameter. HAZUS manual has been used to assess the damaged state of the RC structural building. The damage state is divided into four parts: slight, moderate, extensive, and complete. Probability is given as per equation 12.

(12)

Sd is Spectral Displacement, and ds is damaged state, is Standard normal cumulative distribution function, and is the Median value of spectral displacement. An example of a fragility curve is given in Fig***.*** 6.



**Fig. 6** Example Fragility Curves for Slight, Moderate, Extensive and Complete Damage. (Source: HAZUS 4.2 SP3, p. 5-3)

Estimation of the fragility curve requires a wide range of statistical data. These data were presented in the HAZUS (chapter 5) manual for low-rise, mid-rise and high-rise structures. Damage states taken are based on the structure's performance and as guided by the HAZUS manual. [13] proposed the damage state based on spectral yield displacement and spectral ultimate displacement, presented in Table 1. A weighted average damage index (Dsm) is calculated to map the single parameter's damage distribution as per equation 13.

(13)

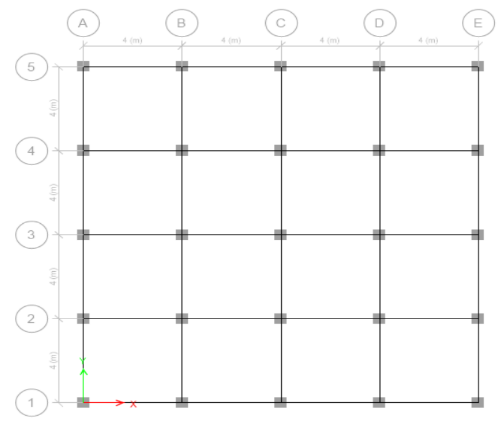
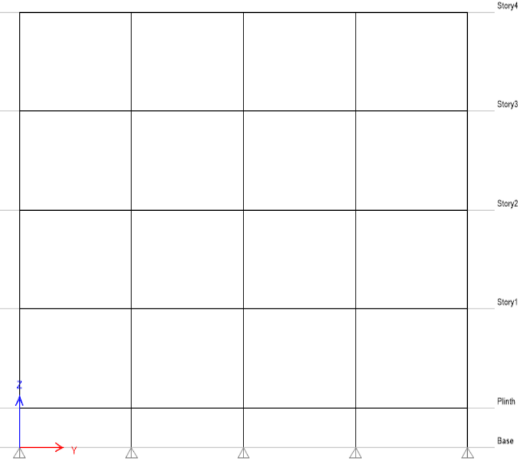
dsi is the damage state that takes values 1,2,3,4 respectively, and P[dsi] is the corresponding damage probability.

**Table 1** Damage State and Threshold (Source: Barbet, 2008, p. 853)

|  |  |  |
| --- | --- | --- |
| **Most Probable Damage State** | **Damage State Threshold** | **Average Damage index** |
| Slight | 0.7 Sdy | 0.5 – 1.5 |
| Moderate | Sdy | 1.5 – 2.5 |
| Extensive | Sdy + 0.25(Sdu – Sdy) | 2.5 – 3.5 |
| Complete | Sdu | 3.5 – 4.0 |

# Description of Models

The study selects 10 RC frame structures with 4, 6. 8, 10 and 12 stories and similar plan arrangements with five bays (4m each) in the x direction and five (4m each) in the y direction. The plan dimensions in the x direction are 20m, and in the y direction is 20m. The storey height is 3m in all four models, with a plinth height of 1.2m. Fig. 7. shows the plan and elevation of the models. Table 3.1 shows the RC section and design details used.



**Fig. 7** Plan (left) and Elevation (right) of the Model (G+10)

All models are analyzed for zone 4 and soil type 2. The response reduction factor is 5, the Importance Factor of the structure is 1.2, the Exterior and interior walls are of thickness 230 mm and 150 mm, respectively, the live load on the roof is 2 KN/m2, the live load at the floor is 4 KN/m2, Floor finish is 1 KN/m2, Roof Treatment is KN/m2. The materials used for the slab, beam, and column are M30 and Fe 415 for the rebar. Table Table 2 Section details for all models shows the RC section and design details for selected models.

**Table 2** Section details for all models

|  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- |
| Storey No | Floor No | Section | Width  (mm) | Depth  (mm) | Reinforcement used |
| Storey 4 | 1-4 | Beam (Exterior) | 400 | 400 | 4-Ø16 (top), 2-Ø16 (bottom) |
| Beam (Interior) | 350 | 350 | 4-Ø16 (top), 2-Ø16 (bottom) |
| Column | 500 | 500 | 12-Ø16 |
| Storey 6 | 1-6 | Beam (Exterior) | 400 | 400 | 6-Ø16 (top), 3-Ø16 (bottom) |
| Beam (Interior) | 350 | 350 | 4-Ø16 (top), 2-Ø16 (bottom) |
| Column | 500 | 500 | 12-Ø20 |
| Storey 8 | 1-6 | Beam (Exterior) | 400 | 400 | 6-Ø20 (top), 3-Ø20 (bottom) |
| Beam (Interior) | 350 | 350 | 6-Ø20 (top), 3-Ø20 (bottom) |
| Column | 500 | 500 | 16-Ø20 |
| 7-8 | Beam (Exterior) | 400 | 400 | 4-Ø16 (top), 2-Ø16 (bottom) |
| Beam (Interior) | 350 | 350 | 4-Ø16 (top), 2-Ø16 (bottom) |
| Column | 450 | 450 | 12-Ø22 |
| Storey 10 | 1-6 | Beam (Exterior) | 400 | 400 | 6-Ø20 (top), 3-Ø20 (bottom) |
| Beam (Interior) | 350 | 350 | 6-Ø20 (top), 3-Ø20 (bottom) |
| Column | 500 | 500 | 16-Ø22 |
| 7-10 | Beam (Exterior) | 400 | 400 | 4-Ø16 (top), 2-Ø16 (bottom) |
| Beam (Interior) | 350 | 350 | 4-Ø16 (top), 2-Ø16 (bottom) |
| Column | 450 | 450 | 16-Ø20 |
| Storey 12 | 1-6 | Beam (Exterior) | 400 | 400 | 6-Ø24 (top), 3-Ø20 (bottom) |
| Beam (Interior) | 350 | 350 | 6-Ø20 (top), 3-Ø16 (bottom) |
| Column | 500 | 500 | 12-Ø22 |
| 7-12 | Beam (Exterior) | 400 | 400 | 6-Ø20 (top), 3-Ø20 (bottom) |
| Beam (Interior) | 350 | 350 | 6-Ø16 (top), 3-Ø16 (bottom) |
| Column | 450 | 450 | 16-Ø24 |

## Modelling of structure

The structure is modeled using ETABS. Five different-storey height models were prepared. Each height storey consists of two cases, one without the stiffness modifier and the other with the stiffness modifier as per IS 1893 part 1:2016. Different load combinations are taken as per IS 1893 Part 1:2016. The structure is designed and analyzed to verify that all the members are passed. Design results may vary depending on the designer's choices. The nonlinear model is prepared by applying the hinge property at both ends of the beam and both ends of the column. This nonlinear model is based on the physical admissible plastic hinge mechanism. ETABS provide auto hinges to the beam and column, which consist of M3 hinges and PMM hinges, respectively, as per ASCE 41-17. Displacement controlled parameter is used for analysis.

# Results and Discussions

The time period and base shear of the models are given in Table 3. The time period of structure increases slightly on using the stiffness modifier and with the increase in stories.

**Table 3** Fundamental Period and Base Shear of Models

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
| Parameter | With Stiffness Modifier | | | Without Stiffness Modifier | | |
| Time Period (Sec) | Weight Used (KN) | Base Shear (KN) | Time Period (Sec) | Weight Used (KN) | Base Shear (KN) |
| 4 Storey Model | 0.82 | 16523.32 | 991.40 | 0.66 | 16523.32 | 991.40 |
| 6 Storey Model | 1.23 | 24074.77 | 1444.49 | 0.89 | 24074.77 | 1444.49 |
| 8 Storey Model | 1.65 | 31544.05 | 1815.87 | 1.19 | 31544.05 | 1815.87 |
| 10 Storey Model | 2.07 | 38931.17 | 1810.13 | 1.50 | 38931.17 | 1810.13 |
| 12 Storey Model | 2.52 | 46318.29 | 1806.24 | 1.81 | 46318.29 | 1806.24 |

After the implication of the stiffness modifier in each storey, a decrement in storey shear and overturning moment is observed. The decrease in low-storey height is about 20%, and high-storey height is about 40% in the case of storey shear. In the case of overturning moment, decrement in low storey height is 14% and in high storey structure is about 35%. This percentage change shows that the same stiffness modifier results vary for storey shear, and moment variation differs for each storey.

For the result of inter-storey drift, the G+4 storey structure, storey drift on the top floor, increased by approximately 46%, and in the G+12 storey, it was 26% for the linear static case. In the nonlinear case, the G+4 Storey structure, storey Drift on the top floor is increased by approximately 37% and in the G+12 storey, it is 17%. The percentage change in drift decreases as we increase the number of storeys. Comparison charts for inter-storey drift are shown in Fig***.*** 8 (a, b, c, d, e).

The result for Storey Displacement increased after using the stiffness modifier. In the linear static case, the G+4 Storey structure, storey displacement on the top floor is increased by approximately 37.5% and in the G+12 storey, it is 34.1%. In the nonlinear case, the G+4 Storey structure, storey Drift on the top floor is increased by approximately 78% and in the G+12 storey, it is almost negligible. The percentage change in drift decreases as we increase the number of storeys.

A graph of different colored lines

Description automatically generated (a) A graph of different colored lines

Description automatically generatedA graph of different colored lines

Description automatically generated(b)

A graph of a graph of drift

Description automatically generated with medium confidenceA graph of different colored lines

Description automatically generated  
 (c) (d)

(e)

**Fig. 8** (a, b, c, d, e) Inter-Storey Drift Comparison for all Storey Model

## Pushover analysis and Target Displacement

The pushover curve highlights the global behaviour of the frame with ductility and stiffness, and their slopes are gradually decreased with an increase in the lateral displacement of the building. The reduction in slope is due to the progressive formation of plastic hinges in beams and columns. Table4 gives the models' spectral yield, ultimate, and target displacement. There is a significant increase in target displacement after using the stiffness modifier. The pushover curve and their hinges at the performance point are shown in Fig***.*** 9 and Fig***.*** 10 for G+4 without and with modifier.

**Table 4** Spectral yield, spectral ultimate and Target displacement for all models

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
|  |  | Storey 4 | Storey 6 | Storey 8 | Storey 10 | Storey 12 |
| With Modifier | Spectral Yield Displacement (Sdy)(mm) | 74.63 | 98.91 | 110.79 | 138.4 | 174.13 |
| Spectral Ultimate Displacement (Sdu)(mm) | 88.20 | 123.72 | 155.70 | 190.1 | 229.32 |
| Target Displacement (mm) | 28.16 | 44.37 | 60.84 | 79.35 | 94.16 |
| Without Modifier | Spectral Yield Displacement (Sdy)(mm) | 23.31 | 50.24 | 41.14 | 79.94 | 67.77 |
| Spectral Ultimate Displacement (Sdu)(mm) | 52.01 | 70.28 | 89.65 | 114.98 | 140.57 |
| Target Displacement (mm) | 38.67 | 62.84 | 84.24 | 108.37 | 130.69 |

.

**Fig. 9** G+4 without modifier Pushover curve and Hinge formation at target displacement

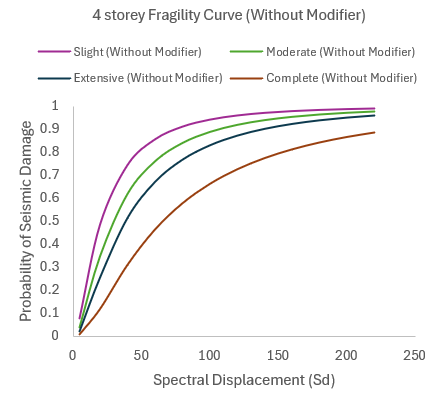
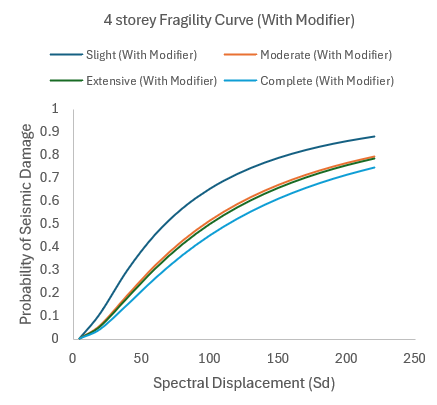
A graph of lines and dots

Description automatically generated

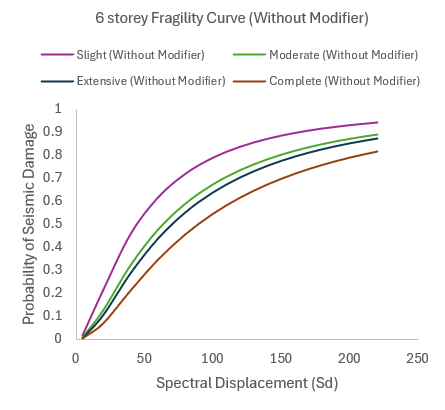
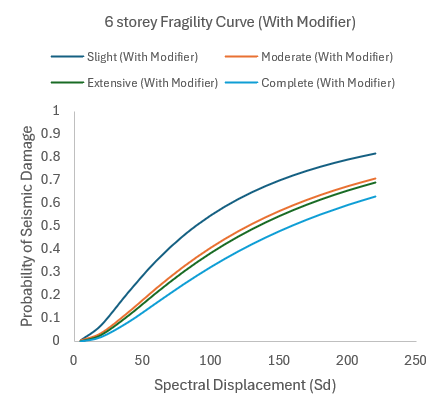
**Fig. 10** G+4 with modifier Pushover curve and Hinge formation at target displacement

## Fragility analysis

The Fragility Curve provides the probability of structural damage to the ground motion parameter (Spectral Displacement in our case). Fig***.*** 11 to Fig*.*15 show the different Fragility curves for all considered models. The weighted mean average index and probable damage state of the structure are provided in Table5 and Table6. The table shows that the weighted mean average damage index decreases after using the stiffness modifier. Percentage change variation is around 50% in Storey 4 and Storey 6. Percentage change variation is around 20 % in G+8 and above storey.



**Fig. 11** Fragility Curves for G+4 Storey Model



**Fig. 12** Fragility Curves for G+6 Storey Model

A graph of different colors and sizes

Description automatically generatedA graph of different colors and sizes

Description automatically generated

**Fig. 13** Fragility Curves for G+8 Storey Model

A graph of different colors and sizes

Description automatically generatedA graph of different colors and lines

Description automatically generated

**Fig. 14** Fragility Curves for G+10 Storey Model

**A graph of different colors

Description automatically generatedA graph of different colors and sizes

Description automatically generated**

**Fig. 15** Fragility Curves for G+12 Storey Model

**Table 5** Calculations of Damage State Probability & Weighted Mean Damage (Without Modifier)

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
| Storey No | Damage State Probability | | | | Weighted Mean Damage index (Dsm) | Probable Damage State |
| Slight | Moderate | Extensive | Complete |
| 4 Storey | 0.49 | 0.35 | 0.29 | 0.19 | 2.87 | Extensive |
| 6 Storey | 0.48 | 0.35 | 0.31 | 0.23 | 3.09 | Extensive |
| 8 Storey | 0.51 | 0.37 | 0.33 | 0.25 | 3.30 | Extensive |
| 10 Storey | 0.54 | 0.40 | 0.36 | 0.27 | 3.50 | Extensive |
| 12 Storey | 0.67 | 0.53 | 0.44 | 0.26 | 4.12 | Complete |

**Table 6** Calculations of Damage State Probability & Weighted Mean Damage (With Modifier)

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
| Storey Number | Damage State Probability | | | | Weighted Mean Damage index (Dsm) | Probable Damage State |
| Slight | Moderate | Extensive | Complete |
| 4 Storey | 0.21 | 0.13 | 0.12 | 0.09 | 1.26 | Slight |
| 6 Storey | 0.24 | 0.14 | 0.13 | 0.10 | 1.33 | Slight |
| 8 Storey | 0.43 | 0.30 | 0.27 | 0.19 | 2.64 | Extensive |
| 10 Storey | 0.44 | 0.29 | 0.28 | 0.21 | 2.72 | Extensive |
| 12 Storey | 0.52 | 0.38 | 0.35 | 0.28 | 3.47 | Extensive |

# Conclusion

The present study includes models at different stories, and these models comprise two cases, one without a stiffness modifier and the other with a stiffness modifier based on IS 1893 (Part 1): 2016. IS 456:2000 is utilized for the analysis and design of the structure. Nonlinear static analysis is used to obtain the pushover curve. Bilinearization of the pushover curve is done as per ASCE 41-17. The target displacement of structures is obtained using the displacement method as per FEMA 356, and the fragility analysis obtains the fragility curves. The damage state of the structure is determined using the damage probability matrices.

On including the stiffness modifier, the time period increases, the storey shear and overturning moment decrease, and storey drift and displacement increase. Performance point increases after using stiffness modifier. Fragility curve and damage probability matrices have given that a building experienced lesser damage after using the stiffness modifier. The weighted mean average damage index decreases after using the stiffness modifier. The larger displacements observed in buildings with stiffness modifiers are consistent with the expected increase in flexibility due to reduced stiffness. However, the results demonstrate that these displacements do not compromise the building's safety or serviceability. Instead, they contribute to a more efficient design by allowing for lower seismic forces, leading to potential cost savings in construction.

The findings support the hypothesis that stiffness modifiers enhance the seismic performance of RC buildings by reducing the likelihood of severe damage and improving overall structural resilience. The study recommends classifying stiffness modifiers according to the structure's height.

# References

1. Nilson AH, Darwin D, Dolan CW (2016) Design of Concrete Structures. McGraw-Hill Education.
2. Ahmed M, Khan MKD, Wamiq M (2008) Effect of Concrete Cracking on the Lateral Response of RCC Buildings. Asian Journal of Civil Engineering (Building and Housing). 9(1):25-34.
3. Branson DE (1963) Instantaneous and time-dependent deflections of simple and continuous reinforced concrete beams. HPR Publication No.7, Part 1, AHD, U.S.B. of Public Roads l-78.
4. Pique JR, Burgos M (2008) Effective Rigidity of Reinforced Concrete Elements in Seismic Analysis and Design. 14th World Conference on Earthquake Engineering, Beijing, China, October 12-17.
5. Paulay T, Priestley MJN (1992) Seismic Design of Reinforced Concrete and Masonry Buildings. John Wiley & Sons.
6. Elwood KJ, Eberhard MO (2006) Effective Stiffness of Reinforced Concrete Columns. Research Digest No. 2006-1, Pacific Earthquake Engineering Research Centre (PEER), University of California at Berkeley, U.S.A.
7. Bhavsar PV, Jamani A (2021) Analytical Study on Effect of Stiffness Modifiers on the Performance of RCC Building Subjected to Seismic Force as per IS 1893:(2016). International Research Journal of Engineering and Technology. 8(5):373-377.
8. Jesal M, Patel SB, George E (2022) Stiffness Modifier in RC and PT Beams and their Impact on Structural Behaviour. International Advanced Research Journal in Science, Engineering and Technology. 9(5):690-697. doi: [10.17148/IARJSET.2022.9598](https://iarjset.com/papers/stiffness-modifiers-in-rc-and-pt-beams-and-their-impact-on-structural-behavior/)

1. Kontoni DPN, Farghaly AA (2018) Stiffness Effects of Structural Elements on the Seismic Response of RC High-Rise Buildings. Archives of Civil Engineering. 64(1):3-20.  
   doi: [10.2478/ace-2018-0001](https://bibliotekanauki.pl/articles/231212)
2. Mithaiwala ME, Patil AA, Khadake NV (2020) A Review of Different Sets of Stiffness Modifiers Varying Through Height of Structure on Analysis of Multi-Story RCC Structure. International Research Journal of Engineering and Technology. 7(8):845-849.
3. Prasanth S, Ghosh G (2022) Effect of Reduction in Column Stiffness on the Resilience of a Building. Materials Today: Proceedings. 55(2):354-358.

doi: [doi.org/10.1016/j.matpr.2021.09.555](https://www.sciencedirect.com/science/article/abs/pii/S2214785321066037?via%3Dihub)

1. Hakim RA, Alama MS, Ashour SA (2014) Seismic Assessment of RC Building According to ATC 40, FEMA 356 and FEMA 440. Arabian Journal for Science & Engineering. 39:7691-7699. doi: [doi.org/10.1007/s13369-014-1395-x](https://link.springer.com/article/10.1007/s13369-014-1395-x)
2. Barbet AH, Pujadas LG, Lantada N (2008) Seismic Damage Evaluation in Urban Area Using the Capacity Spectrum Method: Application to Barcelona. Soil Dynamics and Earthquake Engineering. 28(10):851-865. doi: [doi.org/10.1016/j.soildyn.2007.10.006](https://www.sciencedirect.com/science/article/abs/pii/S0267726107001376?via%3Dihub)
3. Haldar L, Paul S (2016) Seismic Damage Evaluation of Gravity Load Designed Low Rise RC Building using Nonlinear Static Method. Procedia Engineering. 144:1373-1380.   
   doi: [doi.org/10.1016/j.proeng.2016.05.167](https://www.sciencedirect.com/science/article/pii/S1877705816303861?via%3Dihub)
4. Chopra AK, Goel RK (2002) A modal pushover analysis procedure to estimate seismic demands for buildings. Earthquake Engineering and Structural Dynamics. 31:561–582.  
   doi: [doi.org/10.1002/eqe.144](https://onlinelibrary.wiley.com/doi/10.1002/eqe.144)
5. **Porter KA, Kennedy RP, Bachman RE (2007)** Creating Fragility Functions for Performance-Based Earthquake Engineering. Earthquake Spectra. 23(2):471-489.

doi: [doi.org/10.1193/1.2720892](https://journals.sagepub.com/doi/10.1193/1.2720892)

1. Bureau of Indian Standards (2013) IS 15988: Seismic Evaluation and Strengthening of Existing Reinforced Concrete Buildings-Guidelines. New Delhi, India.
2. Bureau of Indian Standards (2016) IS 1893 Part 1: Criteria for Earthquake Resistant Design of Structure Part 1 General Provisions and Buildings. New Delhi, India.
3. Bureau of Indian Standards (2023) IS 16700: Criteria for Structural Safety of Tall Buildings. New Delhi, India.
4. American Society of Civil Engineers (2017) ASCE 41-17: Seismic Evaluation and Retrofit of Existing Buildings Virginia, US.
5. Federal Emergency Management Agency (2000) FEMA 356: Pre-standard and Commentary for the Seismic Rehabilitation of Buildings. Washington DC, US.
6. Federal Emergency Management Agency (2020) HAZUS 4.2 SP3: Hazus Earthquake Model Technical Manual. Washington DC, US.
7. Federal Emergency Management Agency (2005) FEMA-440: Improvement of Nonlinear Static Seismic Analysis Procedures. Washington DC, US.
8. California Seismic Safety Commission (1996) ATC-40: Seismic evaluation and retrofit of concrete buildings volume 1. Report: SSC 96-01. Redwood City, California.