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Study on the Behaviour of the Setback Rc Frame Building Using Pushover Analysis

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Abstract:- Multi-story framed structures are highly sensitive to the distribution of mass, stiffness, and strength within the horizontal and vertical dimensions of a building, as well as to severe seismic activity. When lateral load-resisting frames in multistory structures have structural flaws, damage usually starts during earthquakes. Furthermore, the structural deterioration brought on by plastification is frequently made worse by these flaws, which might eventually result in total failure. These shortcomings could result from differences in mass, stiffness, or strength between floors. These floor-to-floor variances are often ascribed to sudden changes in the geometry of the frame along its height. In past earthquakes, vertical discontinuities have played a significant role in the collapse of several buildings. It has frequently been observed that abnormal configurations, regardless of plan or height, played a major role in the failures of previous seismic disasters. A setback building is one of the most prevalent vertical geometric anomalies in building designs. It is a structure with a significant drop in its lateral dimension at specific heights. Much research has been done to better understand the behavior of setback and irregular structures and to develop strategies to improve their performance. Pushover analysis is one kind of nonlinear static analysis that is frequently used to assess the seismic response of framed buildings. However, the conventional pushover analysis outlined in FEMA 356:2000 and ATC 40:1996 has limits for regular-shaped buildings. The findings demonstrate that mass-proportional uniform load patterns for pushover analysis and suggested enhancements to the target displacement computation can yield reliable forecasts.

Keywords*:* setback building, pushover analysis, irregularity, target displacement.

1. Introduction

The distribution of mass, stiffness, and strength both vertically and horizontally affects how stable multi-story framed buildings are during seismic events. Usually, structural weak spots in the frames that withstand lateral forces are when damage starts. These defects, which are often brought on by abrupt changes in the geometry of the frame as the building rises, can be the consequence of differences in mass, strength, or stiffness among the stories. Several structural collapses in previous earthquakes have been linked to these kinds of vertical anomalies. When designing buildings with reasonably equal distributions of mass, stiffness, and strength, structural engineers typically feel more confident. However, they have more challenges in ensuring seismic resistance in buildings with non-standard geometric designs. A prevalent type of vertical geometric irregularity is the setback structure, characterized by an abrupt decrease in lateral dimensions at specific elevations. Because of its practical and beautiful features, this architectural style is becoming more and more common in contemporary multi-story buildings. Furthermore, this type of building shape complies with the restrictions placed by building bye-laws on the "floor area ratio" (an Indian custom).

Because of variations in mass and stiffness at different heights, these constructions have different dynamic qualities than regular buildings. Athanassiadou (2008) pointed out that the modal participation of these structures varies significantly throughout the research that is currently in publication. Furthermore, inter-story drifts in setback structures are anticipated to be larger in the upper levels and smaller in the lower levels as compared to conventional buildings without setbacks. Through a study involving three two-story and ten-story planar frames, the seismic behavior of multi-story reinforced concrete (R/C) frame buildings with uneven elevation profiles was investigated. While the second frame saw two significant disturbances in its upper floors, the first frame was erected consistently and without any difficulties.

There were four notable setbacks at the high echelons of the third frame. The 2004 Eurocode 8 (EC8) guidelines were followed in the design of all three frames. An inelastic static pushover study was performed on both the setback and regular frames using specific seismic input motions. It was discovered that the budget of the buildings was unaffected by the ductility class. Although the irregularities created challenges, these frames fared well seismically. They even performed as well as conventional frames when exposed to twice as strong ground motions as those predicted by the design. This mismatch meant that traditional pushover analysis frequently underestimated the responses in the upper levels of irregular frames. Multimode elastic analysis was used to evaluate Eurocode EC-8's seismic design guidelines. "Setback buildings" had to follow the same design guidelines as ordinary buildings, according to EC-8, and both setback and conventional structures developed in line with EC-8 performed similarly under seismic loads.

The inelastic seismic response of planar steel moment-resisting frames with setbacks was examined in a study by Karavasilis et al. (2008). To investigate different limit states, thirty different ground motion scenarios were applied to a total of 120 frames, all of which were constructed in compliance with European seismic and structural requirements and scaled to vary in severity. The primary findings of the study underscored the importance of inelastic deformation and geometric configuration by demonstrating that deformation demands tended to increase with height. The core "tower" portions of the tower-like constructions had the highest deformation needs, whereas the setback areas showed higher demands for different geometric configurations. The goal of Shahrooz and Moehle's (1990) comprehensive study was to improve design techniques specifically for setback structures by examining the effects of setbacks in multi-story buildings during seismic events. Their primary goal was to construct a six-story reinforced concrete space frame with a 50% setback at mid-height in one direction and the ability to support transient loads.

The displacement profiles seemed to be rather smooth across the building's height. But there were evident inter-story drifts and moderate deterioration at the connection between the tower and the base. The displacement and inertia force profiles suggest that the basic mode largely shaped the overall translational response parallel to the setback. The same test structure remained the focus of their inquiry, and the lateral stress distribution pattern complied with Uniform Building Code (UBC) specifications. Interestingly, there were no discernible variations in the dynamic response. Additionally, they examined six typical reinforced concrete setback frames in greater detail. Soni and Mistry (2006) conducted a similar study in which they compiled data from several sources, including published literature and construction codes, to investigate seismic wave behavior. Their objective was to understand how unevenly aligned building frames will react to seismic activity. Building regulations provided classification criteria for vertical abnormalities, and recommendations for dynamic analysis were made to assess design lateral forces.

The drift in the setback structures led to an increase in drift needs in the tower portions. Buildings, according to seismic standards, must have a discontinuous distribution of mass, stiffness, and strength. When stiffness and strength problems occur at the same time, there is an increased seismic demand. By examining the available information on the fundamental periods of buildings—which was obtained from motion recordings made during eight earthquakes in California, from the Northridge earthquake of 1994 to the San Fernando earthquake of 1971—Goel and Chopra (1997) assessed the formulas found in the current U.S. standards. They created increasingly precise techniques for figuring out the fundamental periods of steel moment-resisting frames and reinforced concrete frames by using regression analysis on the observed period data. Additionally, the results of their investigation offered suggestions for restricting the time frame that might be determined by logical analysis.

The conventional nonlinear static (pushover) analysis as described in FEMA 356 (2000) and ATC 40 (1996) may not offer a comprehensive enough assessment of the seismic performance of setback buildings due to its limitations in effectively addressing irregular structures that are influenced by significant higher mode effects.

The literature contains numerous documented attempts to modify the pushover analysis method to fit various types of irregular structures.

The primary objective of this study is to determine whether modifying the conventional pushover analysis approach for setback structures is appropriate. This work provides important insights to enhance the accuracy and applicability of pushover analysis for these kinds of irregular structures.

"Displacement response" refers to how a structure moves or varies in response to loads or outside factors. It is essentially the movement of a structure due to applied loads such as earthquake, wind, or living loads. Recognizing dynamic deformations that may result in damage and stress is made simpler by having an understanding of the displacement response. It makes it possible for engineers to take the appropriate precautions to avoid failure. The displacement response might theoretically be obtained by twice integrating the measured acceleration data. In practical terms, this means integrating acceleration twice to yield displacement.

- First integration: Velocity (m/s) = ∫(Acceleration) dt
- Second integration: Displacement (m) = ∫(Velocity) dt

A structure's ductility is essential because it keeps the structure strong even when there is significant inelastic deformation. When structures have well defined yield-displacement and ultimate deformation zones, this is especially crucial. The ratio of a structure's maximum displacement to its yield displacement is used to measure its ductility. In essence, ductility is a reflection of a structure's capacity to release energy.

Ductility is essential while building earthquake-resistant structures. One important consideration in the construction of earthquake-resistant buildings is the structure's ductility. Understanding a structure's inelastic behavior is necessary to guarantee its safety during a seismic event.

Ductility, $\mu = \frac{\Delta u}{\Delta y}$

Where, Δu = Maximum displacement of the structure.

Δy = Yield displacement of the structure.

2. Objectives

i. To evaluate how different setback percentages affect high-rise building's reinforced concrete frame performance.
ii. To assess the planned building displacement using the application of pushover analysis, also referred to as nonlinear static analysis.
iii. To suggest enhancements to the pushover analysis methods already in use for setback structures.

3. Methods

i. By carefully assessing the literature to ascertain the objectives of the research.
ii. Selecting an assortment of building frame models with setbacks. These models will differ in height (8 to 20 stories), width (four to twelve bays), and other special features. We will concentrate on 48 different setback frame arrangements.
iii. Examine each of the 48 building models using the Non-linear Static (Pushover) Analysis technique.
iv. We will investigate possible improvements to the current pushover analysis process as part of our research, with an emphasis on improving target displacement estimation. Our intention is to modify these protocols so that they better fit the special needs of setback buildings.

4. Pushover Analysis

A kind of nonlinear static analysis known as pushover analysis involves progressively increasing the amplitude of the lateral loads. A preset distribution pattern is maintained throughout the building's height, as shown in Fig. 4.1(a). The structure is moved until the structure collapses or the "control node" reaches the predefined "target displacement". We keep an eye out for the development of cracks, plastic hinging, and structural component failure during this procedure. The relationship between base shear and control node displacement for each pushover inquiry is shown in Figure 4.1(b). An essential part of the analysis is the base shear vs. control node displacement curve, sometimes referred to as the pushover curve or capacity curve. This curve is a reference point for determining structural capacity and is utilized to calculate the "target displacement".

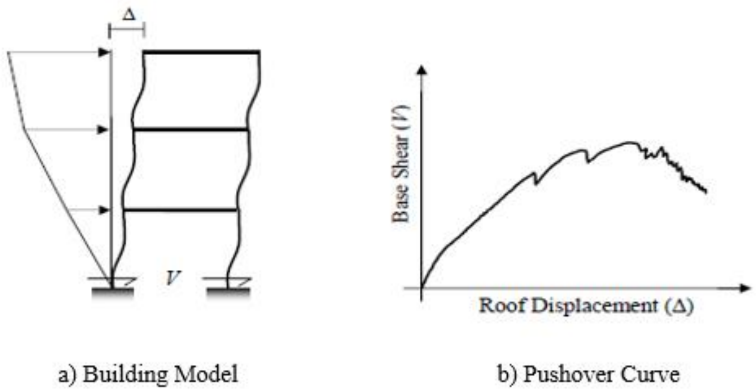

Fig 4.1: A diagram illustrating the pushover analysis process.

There are two pushover analysis options. Initially, the structure is pushed until the goal displacement can no longer be determined. Next, the seismic demand is computed up to the desired displacement. Seismic demands are evaluated at this objective displacement, including component deformations, forces, and narrative drifts. To assess the behavior of the structure, these demands are then compared to the applicable structural capacity or defined performance limit states. The two main orthogonal axes of the building may each undergo an individual examination; when appropriate, bidirectional impacts may be taken into consideration at the same time. Both the lateral load pattern and the control node selection impact the analysis's conclusions.

The control node, which is often found on the roof near the center of mass, is necessary for this operation. When choosing the lateral load pattern for pushover analysis, the FEMA 356 standards must be followed. To completely evaluate the structural behavior, lateral loads (i.e., dead and some live loads) are often applied in both positive and negative directions in addition to gravity loads. For pushover analysis, FEMA 356 allows the use of adaptive lateral load patterns rather than uniform distribution. Nevertheless, the standard offers no comprehensive instructions for the adaptive method. Although it requires a lot more analysis work, an adaptive strategy can produce findings that are consistent with the real behavior of the building. The typical lateral load pattern used in pushover analysis is shown in Fig. 4.2.

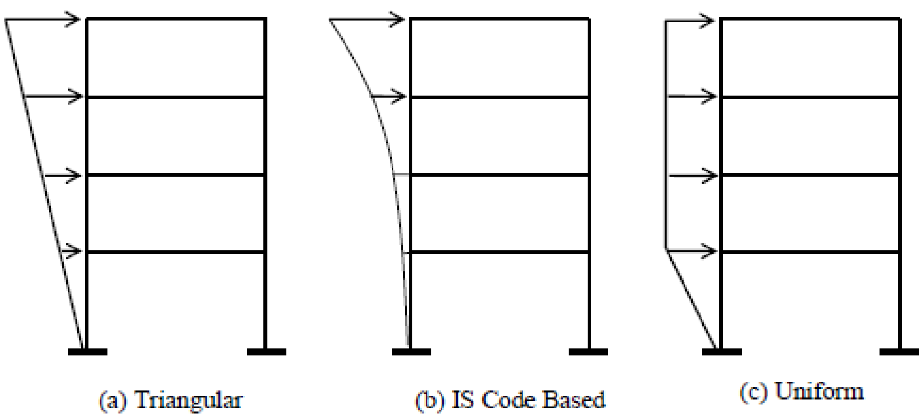

Fig 4.2: Lateral Load Pattern for Pushovers analysis.

The target displacement is the demand for displacement at the control node that the building makes in response to the specific ground motion that is being studied. Pushover analysis depends on the building's general and specific responses, or on its forces and displacements at this objective displacement. The overall effectiveness of the structure is ascertained by comparing these responses to the intended performance limit state. The efficacy of a pushover analysis depends on how precisely the planned displacement is estimated. There are two methods for figuring out the target displacement:

a) Displacement Coefficient Method (DCM) of FEMA 356.

b) Capacity Spectrum Method (CSM) of ATC 40.

Using the pushover curve, both approaches calculate the worldwide displacement demand for the structure. They examine the response of a comparable system with one degree of freedom (SDOF) in order to achieve this. The specific strategies each system employs are what set them apart from one another.

The first method uses initial linear characteristics and damping for the particular ground motion under study to compute the elastic displacement of a system known as SDOF. This is then multiplied by a variety of displacement coefficients to determine the building's overall maximum inelastic displacement response at the roof level.

The pushover curve, the first step in the investigation, is shown in Figure 4.3(a) and illustrates the relationship between base shear and roof displacement. The equivalent period (T_{eq}) is graphically calculated by starting with the original period (T_i). The linear stiffness of the related SDOF system is measured by T_{eq}. The peak elastic spectral displacement corresponding to T_{eq} may thus be readily found using the response spectrum. The response spectrum in Figure 4.3(b) illustrates the seismic ground motion that is being studied.

$$S_d = \frac{T_{eq}^2}{4\pi^2} S_a \tag{4.1}$$

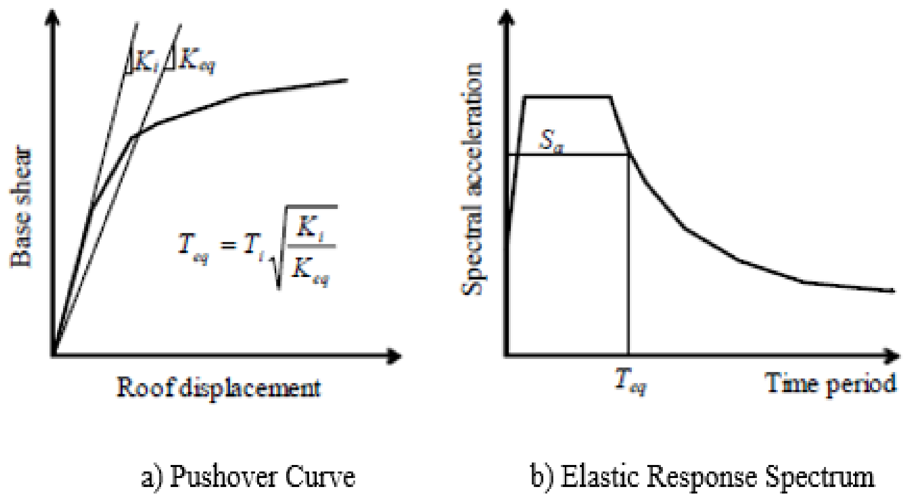

Fig 4.3: Diagrammatic illustration of the displacement coefficient approach.

The maximum anticipated movement of the building's roof (also known as the target displacement) in reaction to the chosen seismic ground motion is shown below:

$$\delta_t = C_0C_1C_2C_3S_d = C_0C_1C_2C_3\frac{T_{eq}^2}{4\pi^2} S_a \tag{4.2}$$

C_0 = shape factor to translate the spectral displacement (typically understood as the first mode participation factor) between the displacement at the building's roof and the similar SDOF system.

C_1 = the proportion between the expected displacement (elastic plus inelastic) for an inelastic system and the displacement of a linear system.

C_2 = a factor that takes into consideration the pinching impact on the load-deformation relationship caused by stiffness and strength deterioration.

C_3 = a factor to adjust geometric nonlinearity (P-Δ) effects.

5. Structural Modelling

This thesis examines the nonlinear analysis of many structural models that show vertically uneven multi-story setback buildings. An overview of the various parameters that define the computational models, the underlying assumptions, and the architectural geometries that are examined in this study are given in the first section of this chapter.

It is crucial to appropriately model the nonlinear properties of various structural elements in nonlinear analysis. In this study, a point plasticity model with inelastic flexural hinges was used to simulate the frame members. This chapter's second section explores the characteristics of these point plastic hinges, describes how they are made, and highlights the underlying presumptions.

Part of the modeling process involves creating and integrating the various load-bearing building framework components. Ideally, the strength, deformability, stiffness, and mass distribution should all be precisely represented in the model. An overview of the material properties and structural elements employed in this study is provided below.

M-40 concrete and Fe-550 grade reinforcing steel are used in all frame types in this investigation. These materials' elastic qualities meet the requirements of Indian Standard IS 456 (2000). The short-term modulus of elasticity (Ec) of the concrete can be calculated using the formula below:

$$E_c = 5000\sqrt{f_{ck}}\ MPa \quad (5.1)$$

where C is the characteristic compressive strength (in MPa, in this case 40 MPa) of the concrete cube at day 28. The steel rebar's modulus of elasticity (E_s) and yield stress (f_y) are calculated in accordance with IS 456 (2000).

Three-dimensional frame parts serve as the representation of beams and columns in this piece. We may compute the bending moments and forces at the interfaces between the beams and columns by adding end offsets to the frame elements that comprise the beam-column connections. It is believed that these beam-column couplings are stiff (Fig. 5.1). Furthermore, the underlying premise of every model is that the columns are set in place and reach the foundation. Additionally, we add nonlinear features to the frame elements at potential yielding places.

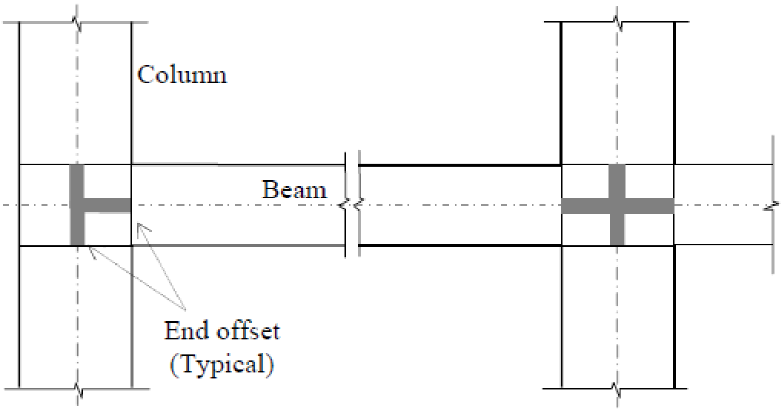

Fig 5.1: End offsets are used at beam-column joints.

In our analysis, we take into account the structural influence of slabs due to their in-plane stiffness by designating a "diaphragm" action at each floor level. In addition, we separately calculate the mass and weight contribution of the slab on the supporting beams.

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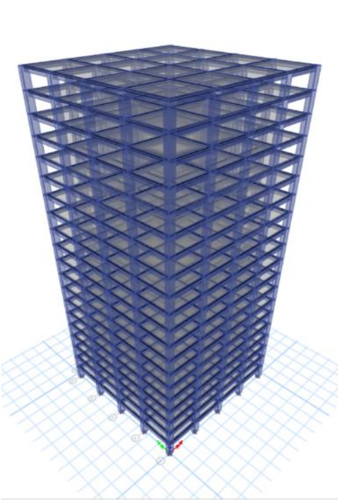
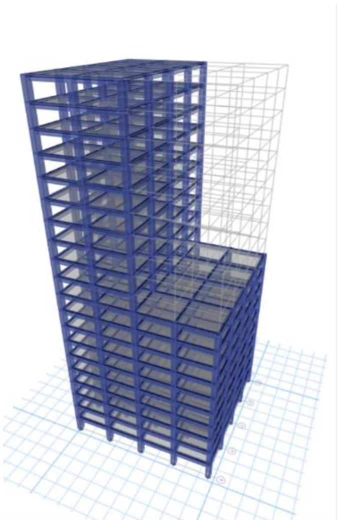


a) Regular Building



b) Setback Building

Fig 5.2: Typical structural models.

This study focuses on orthogonal frames that lay in a plane and have storey heights and bay widths. We examined a range of architectural shapes, each of which to some extent indicated an irregularity or setback. Specifically, we looked into three width categories that ranged from 4 to 12 bays (in the direction of the earthquake) and had a constant bay width of 6 meters. Remarkably, bay widths of four to six meters are common in Indian and European building techniques.

Four distinct height categories, with a uniform storey height of three meters, ranging from eight to twenty stories, were taken into consideration in our study. We chose a total of 48 building frames, all of which had different setback abnormalities that were attained by progressively reducing their height and width:

- Setback S1: 25% reduction in both width and height.
- Setback S2: 50% reduction in both width and height.
- Setback S3: 75% reduction in both width and height.

Furthermore, we included a standard frame in Fig. 5.2(a), where "R" is indicated, without any problems. The standard setback frame shown in Fig. 5.2(b) next to a comparable regular building.

- Modelling Software: We used ETABS (21) for structural modelling.
- Gravity Loads: The assumed floor loads (including dead and live loads) were 20.7 kN/m.
- Earthquake Ground Motion: In accordance with IS 1893:2016, we defined the ground motion utilizing the software's auto lateral load features.
- Column Sections: The column sections were engineered to satisfy standards for stiffness and strength.
- Materials: All of the selected models were designed using M-40 grade concrete and Fe-550 grade reinforcing steel in compliance with IS 456:2000.

The building structure, which features S1-type irregularity, is referred to as S1-X-Y; X stands for storey count and Y for bay count. Interestingly, the chosen frames include a broad range of fundamental times, ranging from 2.17 to 0.85 seconds. These frames correspond with the real relationships shown in Fig. 4.3 by Goel and Chopra (1997). Consequently, our chosen models faithfully capture the usual behavior of moment-resisting eight to twenty-story reinforced concrete (RC) frames, which is in line with the well-established findings of Goel and Chopra.

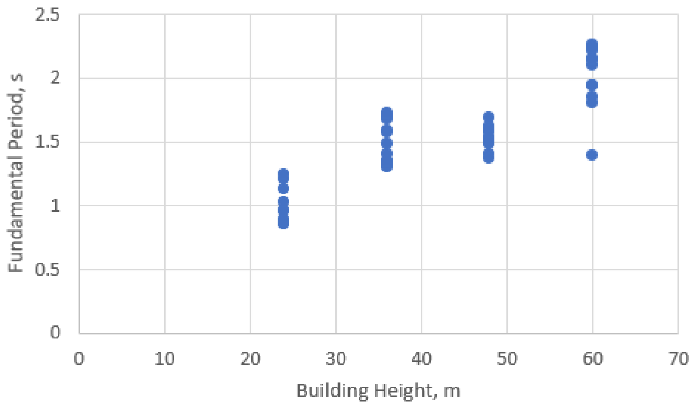


Fig 5.3: Fundamental Period vs Overall Height Variation of all the selected frames.

It is essential to assess the nonlinear behavior of structural elements in pushover analysis. We use a point-plasticity technique to represent this nonlinearity in the current study. Specifically, we assume that at that precise moment, the plastic hinge is enclosed within the frame component that is being considered. The main actions we took were as follows:

i. Hinge Modelling:

• Beams: We modelled beam elements that had flexural hinges (identified as M3) at the extremities of the beams where there may be plastic regions.

• Columns: In a manner similar to this, flexural hinges (P-M2-M3) at the plastic regions under lateral load were used to approximate column elements.

ii. Flexure Hinge Properties: These flexural hinges are intended to simulate the behavior of reinforced concrete components under lateral loads. We calculated the properties of the plastic hinge with ETABS (21).

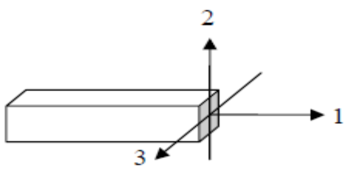


Fig 5.4: The coordinate system defining hinges are flexural and shear.

We look at flexural hinge characterization using moment-rotation curves. The cross-sectional features and reinforcing details at possible hinge positions are used to generate these curves. For each structural component, a moment-curvature analysis is carried out in order to assess the hinge quality. The dimensions of the plastic hinge within the structural member and the properties of the concrete and steel reinforcing elements are significant contributions. We use linked P-M2-M3 features for column elements and uncoupled moment (M3) hinges to describe beam flexural hinges. We look at flexural hinge characterization using moment-rotation curves. The cross-sectional features and reinforcing details at possible hinge positions are used to generate these curves. For each structural component, a moment-curvature analysis is carried out in order to assess the hinge quality. The dimensions of the plastic hinge within the structural member and the properties of the concrete and steel reinforcing elements are significant contributions. For beam flexural hinges, we define them using uncoupled moment (M3) hinges and linked P-M2-M3 characteristics for column elements. At the hinge point, these values take into consideration the interaction between biaxial bending moments and axial force. Remember that even though column flexural hinges allow axial forces to interact, rotation values are closely related to the axial force caused by gravity loads.

6. Results

Using ETABS, linear and nonlinear static tests were performed on all 48 building models with various setback anomalies, as demonstrated in Chapter 5. This chapter presents the findings from the analyses that were previously covered in great depth. The results fall into two main categories:

• Patterns of lateral loads appropriate for performing nonlinear static analysis.

• Target displacement estimation.

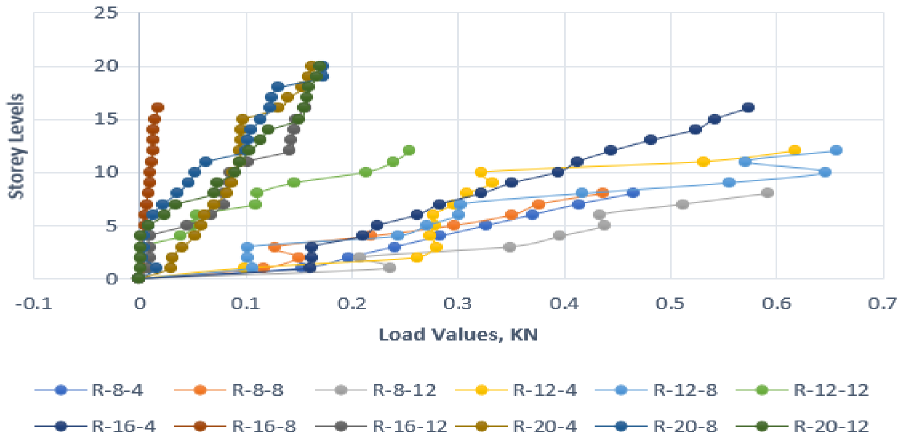


Fig 5.5: Storey Levels VS Load Values of Regular Buildings.

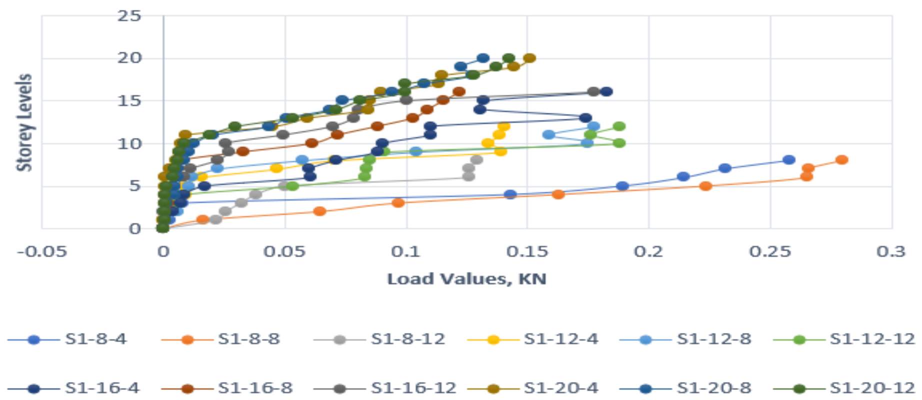


Fig 5.6: Storey Levels VS Load Values of S1 Setback Buildings (25%).

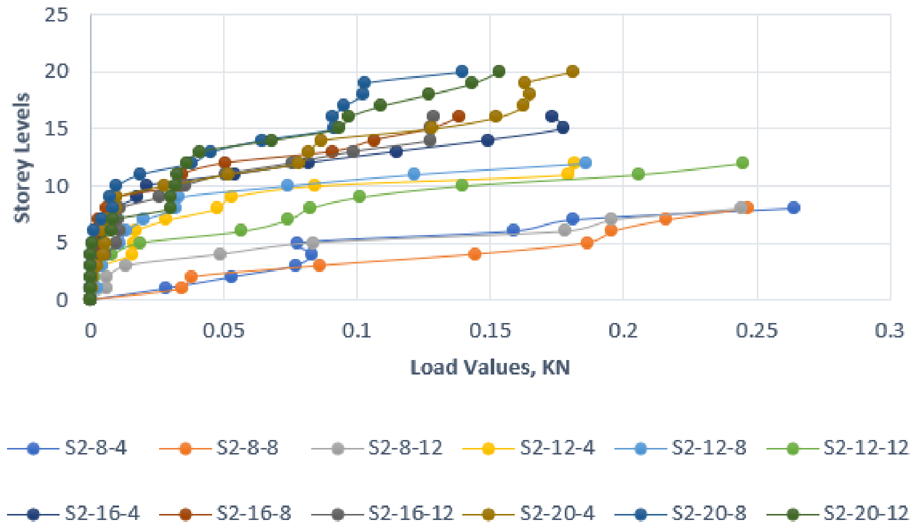


Fig 5.7: Storey Levels VS Load Values of S2 Setback Buildings (50%).

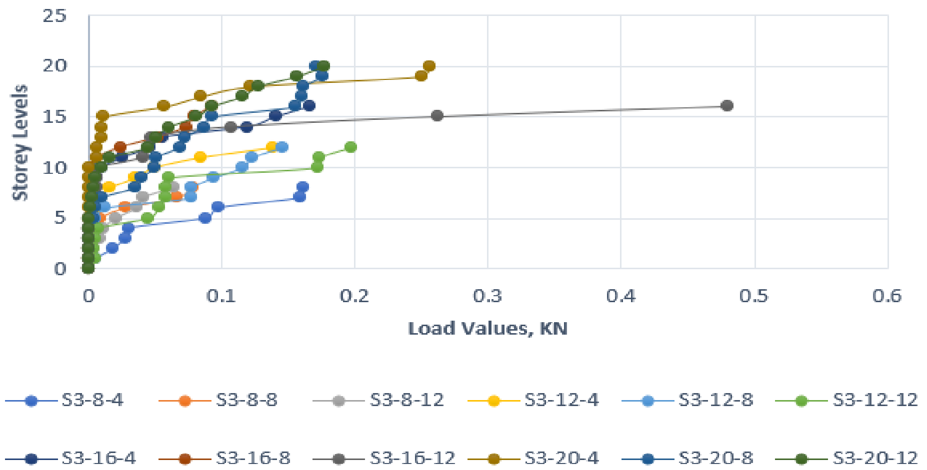


Fig 5.8: Storey Levels VS Load Values of S3 Setback Buildings (75%).

The findings imply that the base shear capabilities rise with the number of storeys and bays for all load patterns taken into consideration. This pattern is explained by the building's increasing seismic weight as more stories and bays are added.

For a given height category, the maximum roof displacement capacity ascertained using the IS 1893 load pattern falls within a predefined range. However, it's still unclear how adding stories will change the requirement for relocation. It is important to note that for all load patterns that have been studied, the maximum roof displacement capacity reduces as the number of bays grows; the precise cause of this variance is yet unknown.

The base shear demand derived from the pushover study of the IS 1893:2016 load pattern is consistent for both normal and S1 type setback frames. Conversely, there is a gradual decrease in the need for S2 and S3 type setback frames. It has been found that the displacement capacity is nearly constant for all pushover curves for a given height category and number of bays. Interestingly, this potential seems to be unaffected by an increase in setbacks.

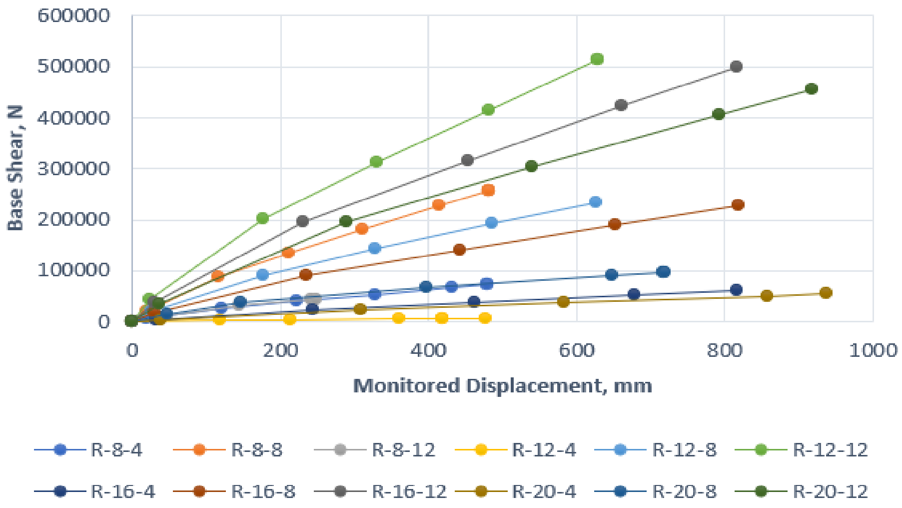


Fig 5.9: Base Shear VS Monitored Displacement of Regular Building.

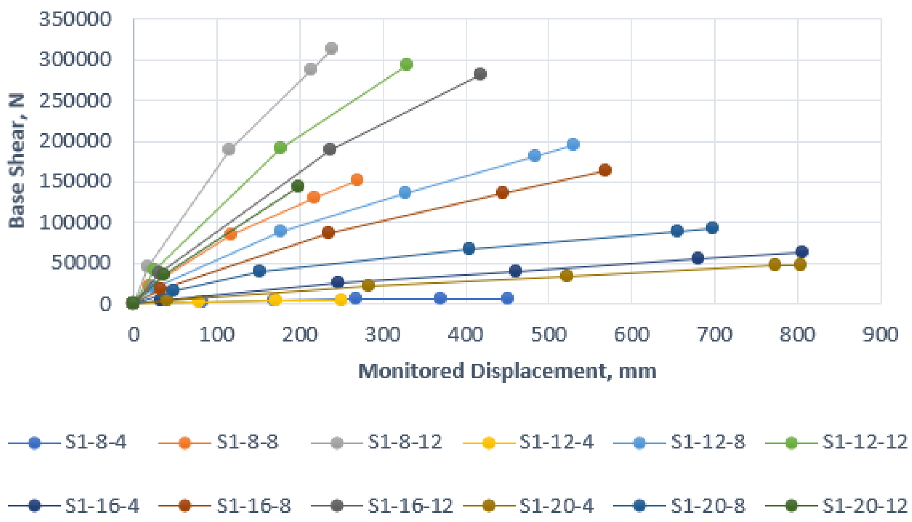


Fig 5.10: Base Shear VS Monitored Displacement of S1 Setback Buildings (25%).

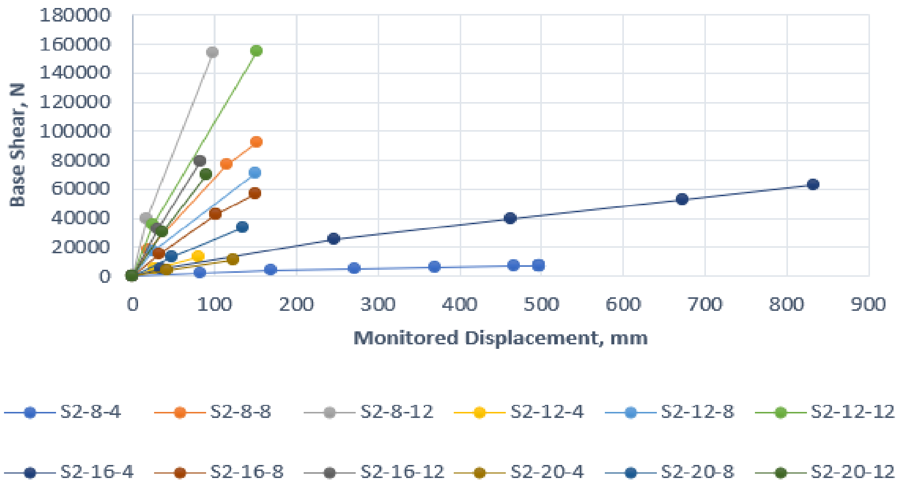


Fig 5.11: Base Shear VS Monitored Displacement of S2 Setback Buildings (50%).

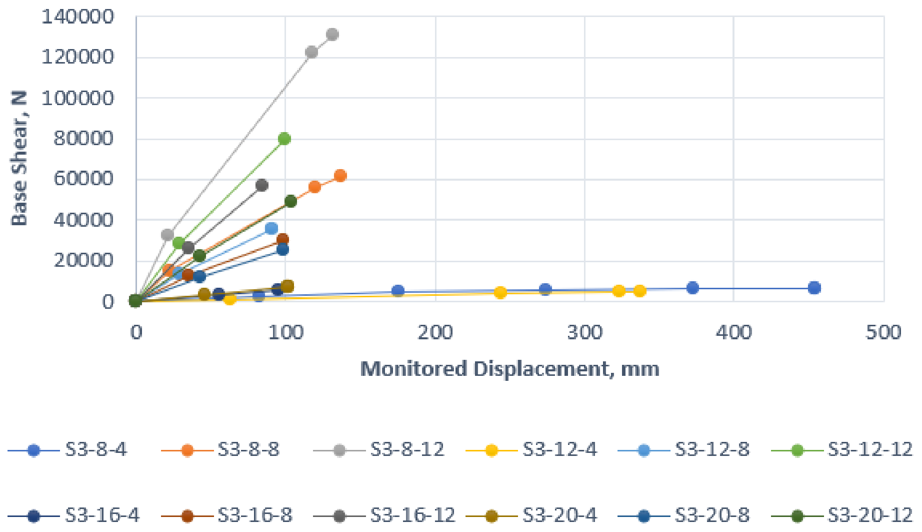


Fig 5.12: Base Shear VS Monitored Displacement of S3 Setback Buildings (75%).

For conventional buildings, pushover assessments using load patterns in accordance with FEMA 440 offer thorough insights into the connection between base shear and roof displacement. The findings for typical buildings, shown in Figures 5.9 through 5.12, corroborate this claim. However, setback structures are exempt from this restriction, particularly tall ones with more irregularities, such as S3-type setbacks.

Because of how widely used and easily applied the displacement coefficient approach is in design processes, it is particularly remarkable (FEMA 356:2000). However, our study has attempted to precisely modify this approach to target displacement estimation in setback constructions. The sections that follow provide specifics on the suggested changes along with their rationale. The displacement coefficient technique, as shown in FEMA 440, highlights the impact of building geometry on the C_0 factor but has little effect on C_1, C_2, and C_3.

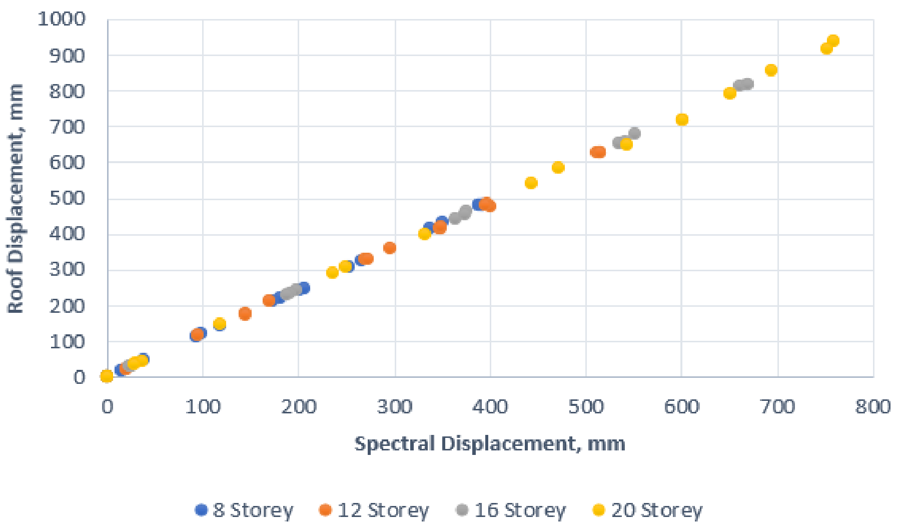


Fig 5.13: Regular Frame Roof Displacement VS Spectral Displacement.

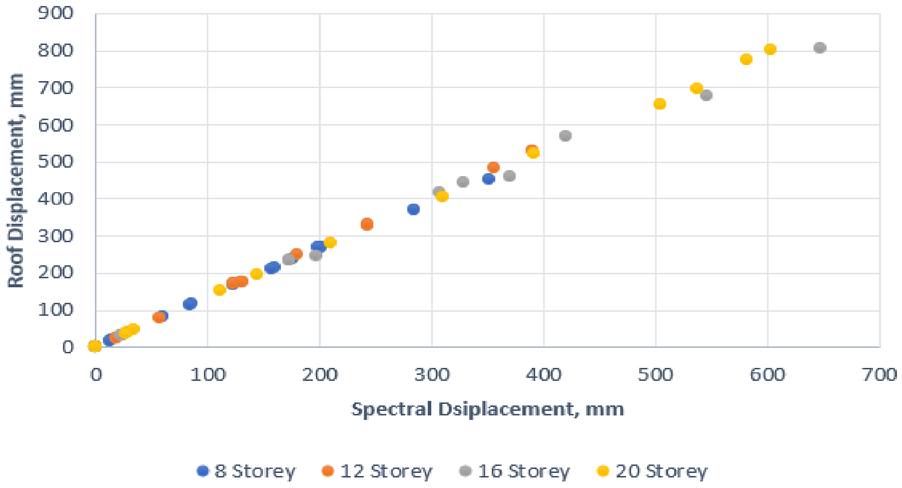


Fig 5.14: S1 Setback frame Roof displacement VS Spectral Displacement.

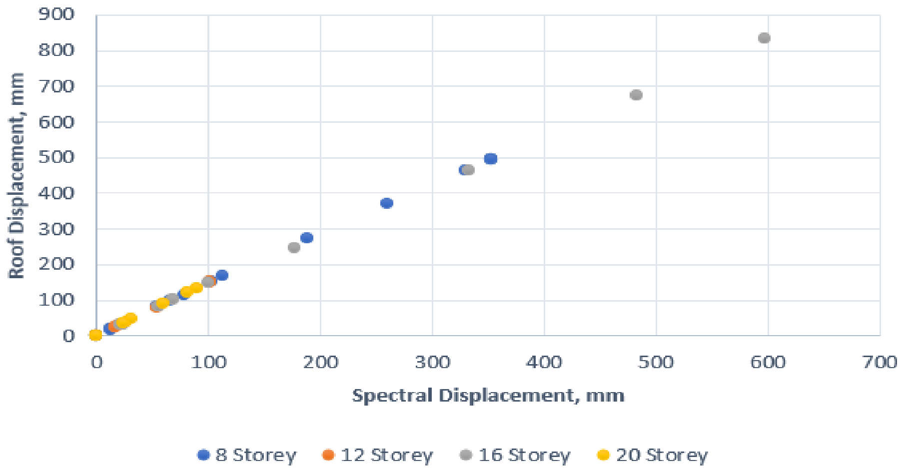


Fig 5.15: S2 Setback frame Roof Displacement VS Spectral Displacement.

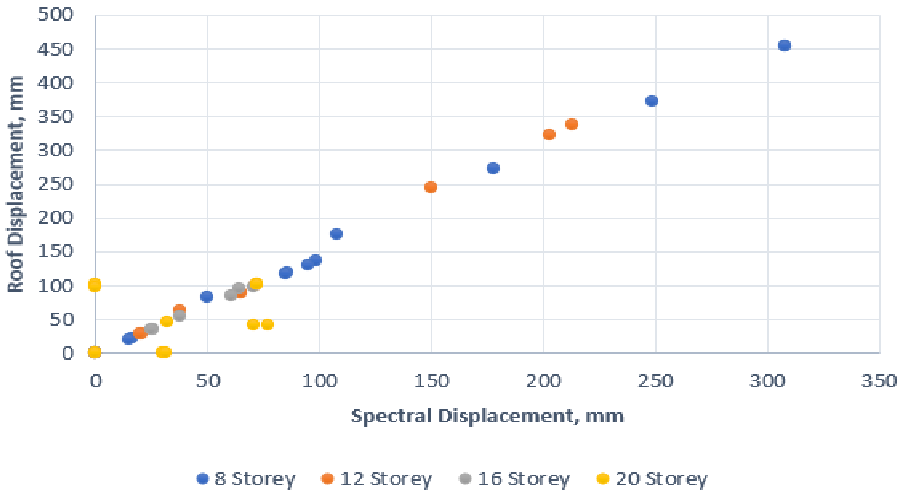


Fig 5.16: S3 Setback frame Roof Displacement VS Spectral Displacement.

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The average maximum roof displacement for each type of frame is compared to the average spectral displacement obtained from the corresponding comparable SDOF system in Figures 5.13, 5.14, 5.15, and 5.16. These evaluations include setback frames of the S1 type, S2 type, and S3 type in addition to conventional frames. The results for each type of frame are categorized based on its height. The C_0 values for the two reference lines, the uniform load pattern ($C_0 = 1.2$) and the triangular load pattern ($C_0 = 1.3$), are also displayed in these figures. These are from the FEMA 356 file. These figures clearly show that C_0 typically isn't within the FEMA-recommended ranges. The difference for setback frames is greater than the difference for standard frames (R). Surprisingly, the divergence from FEMA values is less pronounced in lower story frames than in narrative frames with more stories. The elastic roof displacement of a specific MDOF frame and the elastic spectral displacement of an analogous SDOF system change significantly with increasing irregularity and number of floors. For typical buildings, FEMA 440 provides recommended ranges for C_0 factors to aid in the estimate of inelastic displacement demand. However, the investigation's findings demonstrate that the actual C_0 values for setback buildings are more than the highest upper bounds set by FEMA. Notably, even for typical high-rise buildings, the C_0 values are not reliably predicted by FEMA standards. The effect of the bay count on the C_0 value was examined. As seen in Figures 5.17, 5.18, 5.19, and 5.20, the C_0 factor varies with the number of bays while maintaining a constant height and regularity index. It's noteworthy to notice that the data suggests the C_0 factor for mid-rise buildings is not significantly influenced by the number of bays. Figures 5.108 to 5.111 further illustrate how the C_0 factor fluctuates with the percentage of setback in a building frame, highlighting the significant influence that setback irregularities have on the C_0 value.

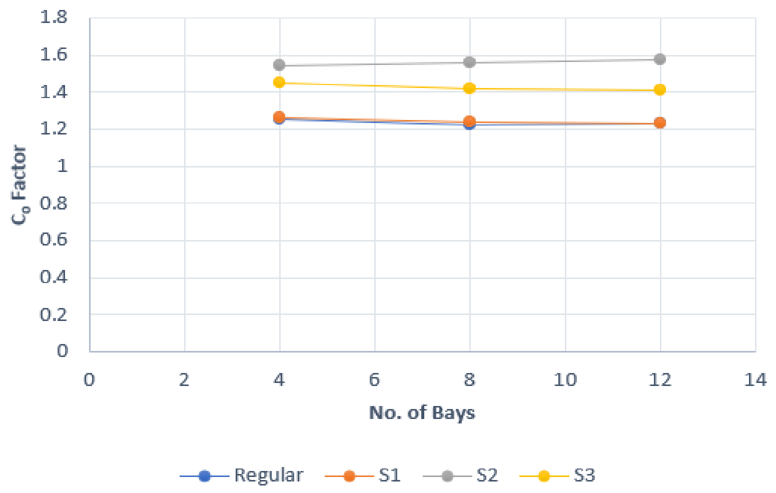

Fig 5.17: C_0 of 8 Storey RC Building.

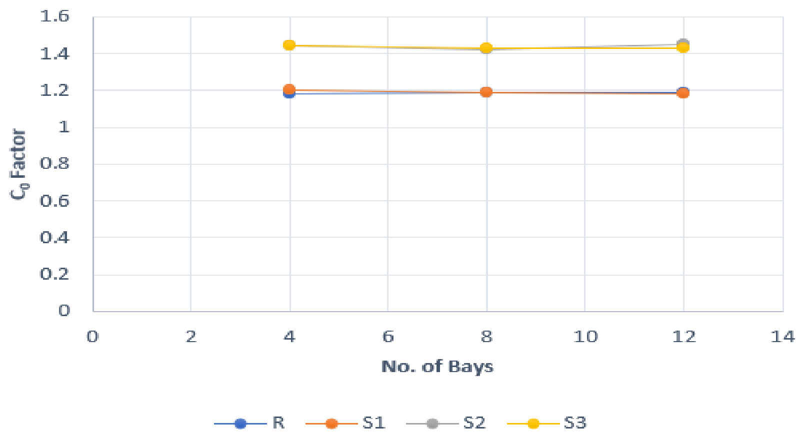

Fig 5.18: C_0 of 12 Storey RC Building.

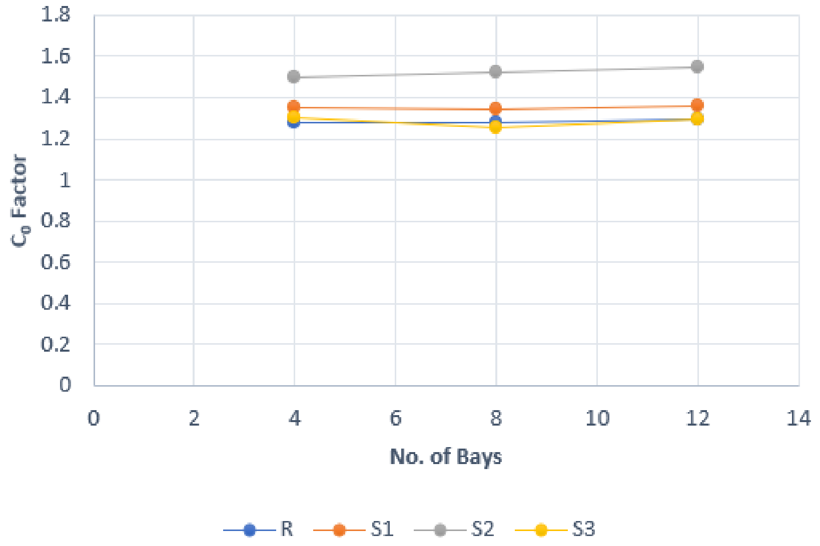

Fig 5.19: C_0 of 16 Storey RC Building.

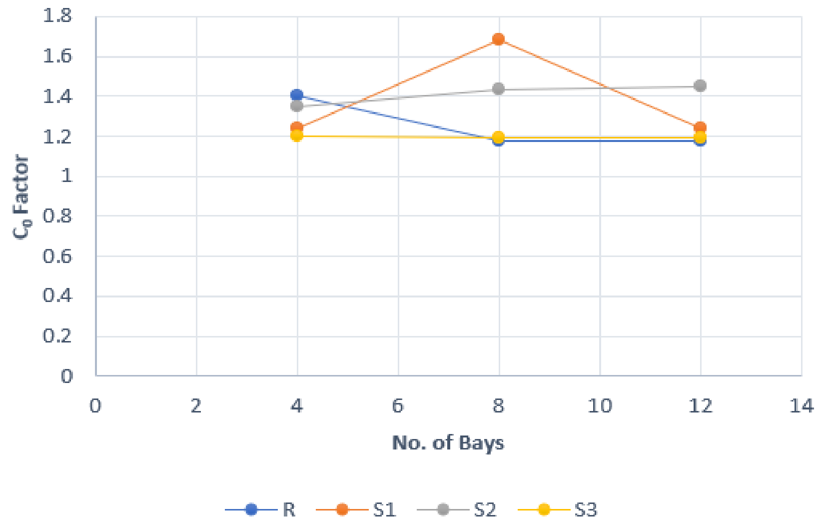

Fig 5.20: C_0 of 20 Storey RC Building.

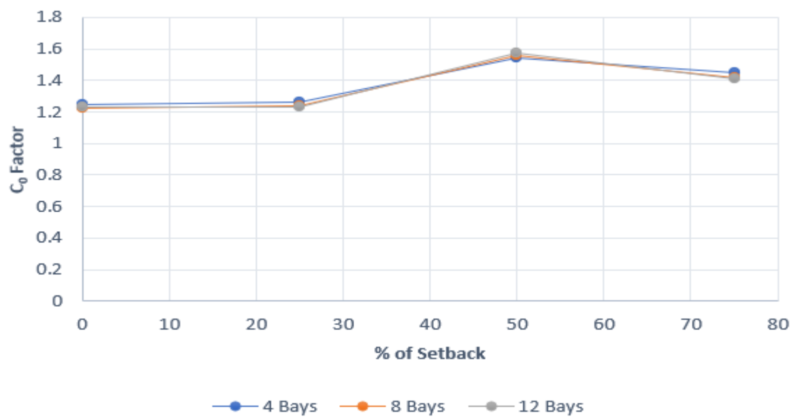

Fig 5.21: Percentage of Setback in 8 Storey RC Building.

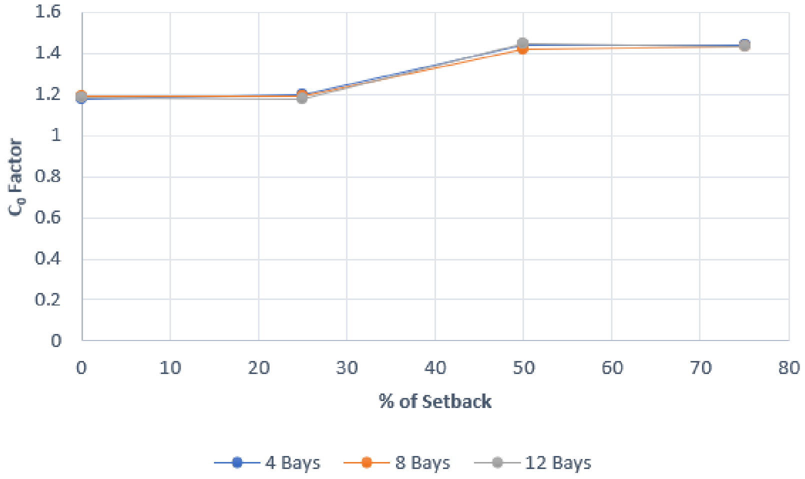

Fig 5.22: Percentage of Setback in 12 Storey RC Building.

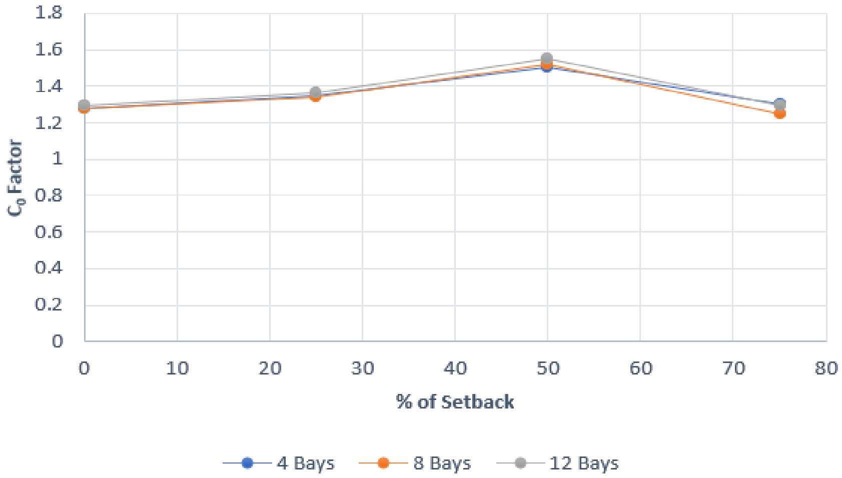

Fig 5.23: Percentage of Setback in 16 Storey RC Building.

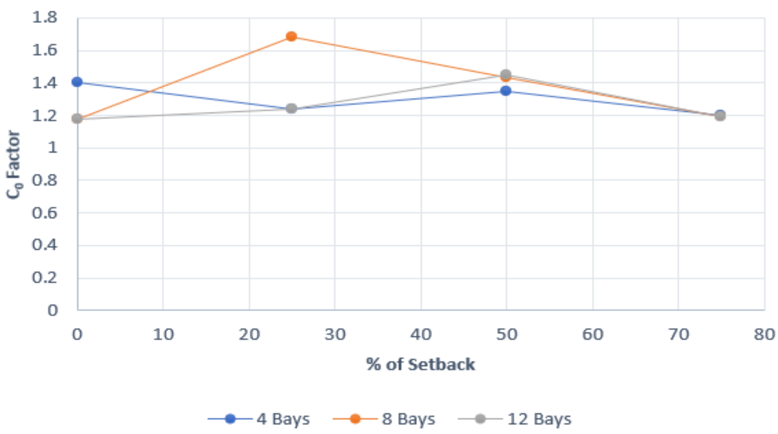

Fig 5.24: Percentage of Setback in 20 Storey RC Building.

The findings show that the C_0 factor is influenced by the building height, bay width, and structural irregularity percentage (% setback). However, the modest fluctuation found (particularly in mid-rise buildings) suggests that the C_0 factor is largely insensitive to building width and height. Regression analysis views the C_0 factor as a function of the building's irregularity, represented as a setback percentage. The following equation can be used to calculate the C_0 factor:

$$C_0 = 1.266 + (0.189 \times x) \qquad (6.1)$$

Where, x is the % of Setback (0 < x < 1.0).

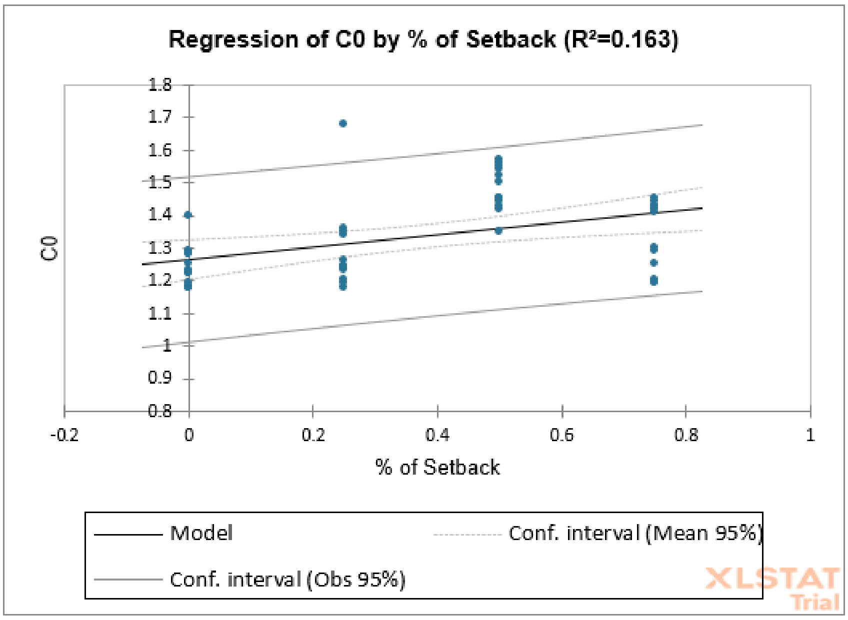

Fig 5.21: Comparison of C_0 factor obtained from the proposed function.

7. Conclusion

Reviewing a lot of the literature on setback buildings usually highlights a few key points. First off, the displacement demand in these structures is mainly concentrated around the setbacks and is intimately related to the frame's geometrical arrangement. Second, higher modes of vibration have a major impact on the overall structural response of setback buildings. Thirdly, the displacement demands are frequently underestimated by standard pushover assessments, especially on the top levels of irregularly framed buildings. Interestingly, high-rise and setback buildings frequently have characteristics with conventional mid-rise structures in terms of the first mode shape and the triangular load pattern. Consequently, many of the buildings under examination show striking parallels in the pushover curves used to assess structural behavior. According to FEMA 440, pushover equations with uniform and triangular load patterns may accurately represent how a typical building behaves in terms of foundation shear against roof displacement. However, our results offer a different perspective on setback buildings, particularly high-rise constructions with noticeable imperfections (such S3-type setbacks). The usual procedures frequently fail to adequately account for the seismic effects. This study demonstrates that the FEMA 440 limitations are not always met by the C_0 factor, which is frequently utilized in seismic design. Divergence is still very small in standard frames (marked as 'R'), but it is often much more evident in setback frames. It's noteworthy to see that fewer stories are related with less unpredictability in comparison to taller frames.

The study's findings indicate that various factors, including building height, bay width, and the level of irregularity (indicated as a percentage of setback), influence the C_0 factor. It is noteworthy that the C_0 factor appears largely unaffected by building width and height, given the limited range of variance, particularly in mid-rise structures.

Nonetheless, there is a clear sensitivity of the C_0 factor to the building's setback irregularity (% Setback). In this research, we propose a revision to the C_0 factor as recommended by FEMA 440. This modified factor aims to deliver more accurate estimates of target displacements, especially concerning setback buildings. Unlike the original FEMA 440 recommendation, our revised C_0 factor specifically incorporates the element of setback irregularity.

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