



# Performance-based seismic analysis of a reinforced concrete framed tall building

Sujan Tripathi<sup>a</sup>, Raju Pal<sup>a,\*</sup>, Raman Aryal<sup>a</sup>, Sagar Prasad Sapkota<sup>a</sup>, Shree Krishna Luitel<sup>a</sup>, Simaran Lama<sup>a</sup> and Subash Lamichhane<sup>a</sup>

<sup>a</sup>Department of Civil Engineering, Thapathali Campus, Tribhuvan University, Nepal

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## Abstract

The seismic hazard in Nepal, highlighted by the 2015 Gorkha earthquake, necessitates the transition from conventional force-based design to more reliable methods for tall buildings. This research presents a performance-based seismic design of a typical 15-story reinforced concrete (RC) framed building with two basements, located in the Kathmandu Valley. The building, designed with a dual system of RC shear walls as per the Nepal National Building Code (NBC 105:2020), was modeled and analyzed using the finite element software ETABS v21.0.0. Nonlinear static (pushover) analysis was used to evaluate the building's post-yield behavior and to check if it meets the Life Safety performance level. Results showed that at a roof displacement of 0.414 m, the structure performs well above the Life Safety limit (0.336 m) and remains far below the Collapse Prevention level (0.676 m), confirming that no collapse will occur under design-level earthquakes. Plastic hinge formation indicated a ductile failure pattern, with most of the 69 hinges forming in beams, supporting the strong-column, weak-beam concept. The study concludes that Performance-Based Design (PBD) offers essential insights regarding a building's resilience and is recommended for new tall structures in earthquake-prone areas like Nepal.

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## 1. Introduction

Earthquakes remain one of the world's most devastating natural disasters, with their impact intensifying due to rising urban densification and vertical expansion. Nepal, situated in the seismically active Himalayan belt, ranks among the world's most earthquake-prone countries. The 2015 Gorkha Earthquake, which resulted in nearly 9,000 fatalities and widespread structural damage in central Nepal [1], exposed the vulnerability of the nation's infrastructure and underlined the urgent need to move beyond conventional design methods. There is a need to adopt performance-oriented design philosophies for tall buildings.

Conventional seismic design methods, such as the Limit State Design approach, rely on linear-elastic analysis with seismic forces adjusted through a response reduction factor (R) to account for ductility. However, these force-based methods fall short in predicting post-yield

behavior and fail to capture the actual failure mechanisms of structures [2]. Recent studies comparing Equivalent Static and Response Spectrum methods have shown that for buildings exceeding 8 stories, static methods can underestimate seismic loads by over 25%, reinforcing the need for more rigorous nonlinear assessments [3]. This divergence increases with building height, making a transition to Performance-Based Design (PBD) crucial for high-rise structures to ensure safety beyond code minimum requirements.

To address these challenges, Performance-Based Design (PBD) has emerged as a more advanced and rational design philosophy. Unlike traditional approaches, PBD targets specific performance objectives under varying earthquake intensities [4]. These performance levels, Operational (O), Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP), correspond to different levels of allowable damage and functional expectations, ranging from uninterrupted use to survival without collapse, illustrated in Figure 1. This performance-driven approach also facilitates

\*Corresponding author:

[raju.077bce100@tcioe.edu.np](mailto:raju.077bce100@tcioe.edu.np) (R. Pal)

better decision-making in design trade-offs related to cost, safety, and usability [5].

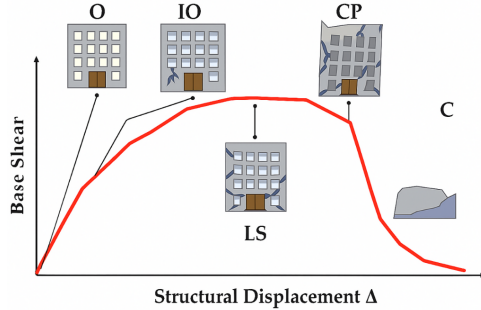


Figure 1: Idealized pushover curve illustrating performance levels [2]

The development of PBD marked a paradigm shift in structural engineering by emphasizing performance objectives over purely strength-based criteria [6]. A major advancement in this domain was the introduction of Direct Displacement-Based Design (DDBD), which promotes displacement- and strain-based strategies. Subsequent studies have demonstrated that PBD allows for improved control over structural behavior, particularly in tall buildings, by enabling a more uniform distribution of inter-story drifts [7]. Numerous case studies have validated the engineering application of PBD using nonlinear static pushover analysis, especially for low- to mid-rise buildings. Researchers have confirmed that specific performance objectives, such as Life Safety, can be reliably achieved while validating ductile failure mechanisms like the strong-column weak-beam concept [2][4][8].

Although pushover analysis is generally effective, its application to high-rise structures becomes more complex due to the influence of higher vibration modes. It has been observed that while displacement-based approaches perform well for first-mode dominant buildings, tall structures require higher-order analysis for accurate multi-modal response representation [9]. Despite these complexities, the benefits of PBD remain significant. For instance, performance targets can still be met in tall shear-wall structures using PBD methods like those outlined in ASCE 41-17, with only moderate increases in material use and construction costs [10]. However, most of the existing research is based on international codes and conditions, leading to a research gap when it comes to applying PBD principles to buildings designed under the Nepal National Building Code (NBC 105:2020). To address this gap, the present study applies the PBD approach using established international guidelines, FEMA 356 [11] and ASCE 41-13 [12], to a

typical reinforced concrete (RC) tall building designed according to NBC 105:2020 [13]. The study evaluates the seismic performance of the structure through nonlinear static (pushover) analysis, aiming to validate the effectiveness of PBD within Nepal's contemporary construction environment. The novelty of this study lies in its focus on a 15-story structure designed specifically under the NBC 105:2020 code, evaluating its post-yield capacity in the context of Kathmandu's soft soil.

## 2. Objectives of the study

1. To conduct a performance-based seismic assessment of a 15-story RC dual-system building, using nonlinear static pushover analysis.
2. To determine the building's IO, LS, and CP performance points and verify that the structure will satisfy the target Life Safety performance objective under a design-level earthquake.
3. To test the failure mechanism by plastic hinge formation and distribution to confirm a ductile, "strong-column, weak-beam" response.

## 3. Methodology

This research follows the core principles of Performance-Based Design (PBD), which involves an iterative process as illustrated in Figure 2. The approach begins by defining specific performance objectives and developing an initial preliminary design, which is then analyzed to evaluate its potential seismic performance. If this design fails to meet the desired performance criteria, it is revised and reanalyzed until performance reaches the required level.

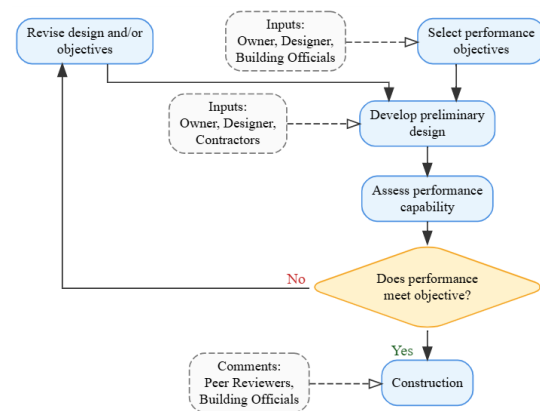


Figure 2: Performance based seismic assessment workflow [2]

For this specific research, the overall procedure is outlined step by step as depicted in Figure 3. Initially,

data collection and preliminary member design were conducted based on the Nepal Building Code (NBC 105:2020) and relevant Indian Standards. The bulk of the work involved creating a nonlinear model in ETABS and performing a pushover analysis to examine the building's capacity. Finally, the structure's seismic performance was compared against the targeted Life Safety performance to evaluate the structure's seismic resilience.

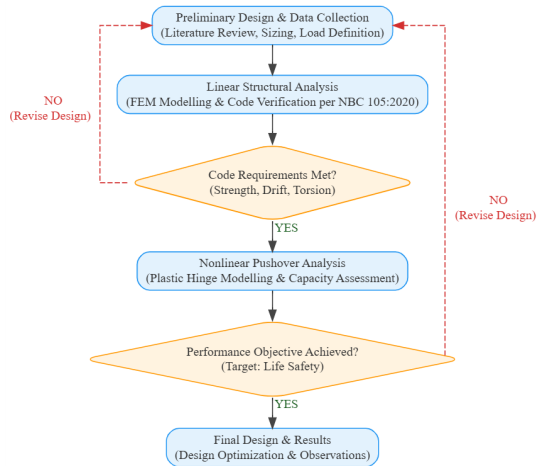


Figure 3: Workflow of the research process

This study focuses on a 15-storey reinforced concrete (RC) framed commercial building (G+12 floors with two basements), typical plan as shown in Figure 4, located in the Kathmandu Valley. Important architectural and structural features of the building are provided in the following description:

- Location: Kathmandu Valley, Nepal
- Soil Type: Type D (Very Soft Soil)
- Structural System: Dual System (RC Moment-Resisting Frame with Shear Walls)
- Importance Factor: 1.25 (Commercial Building)
- Total Height: 53.105 m (47.105 m above Ground Floor Level)
- Material Properties: Concrete Grade M40 ( $f_{ck} = 40 \text{ N/mm}^2$ ); Reinforcing Steel Grade Fe500 ( $f_y = 500 \text{ N/mm}^2$ )
- Design Codes: The structure has been designed as per the Nepal National Building Code (NBC 105:2020)[13], IS 456:2000[14], and IS 13920:2016[15]

A three-dimensional (3D) numerical model was developed as shown in Figure 5 to simulate the building's be-

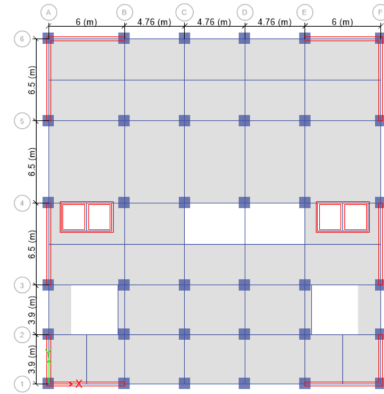


Figure 4: Typical plan view of the building layout

havior using the finite element analysis software ETABS v21.0.0.

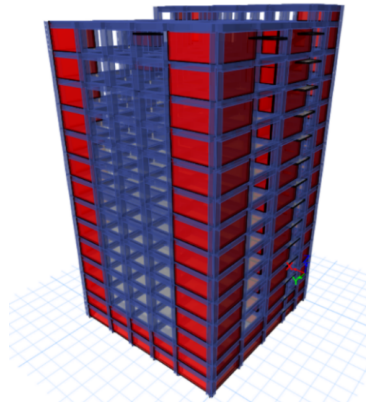


Figure 5: 3D view of the building model

Loadings applied included:

- Dead Loads (DL) and Live Loads (LL), which were assigned according to IS 875 Part I [16] and IS 875 Part II [17], respectively.
- Seismic Loads, which were calculated in accordance with NBC 105:2020. Load cases incorporated all relevant gravity and seismic actions.
- Considered loads include Dead Load, Live Load, Floor Finish, Staircase Load, Water Tank Load, Lift Core Load, Helicopter Load, and Soil Pressure.
- Seismic loads were applied in both X and Y directions for Ultimate Limit State (ULS) and Serviceability Limit State (SLS), following NBC 105:2020.
- Load combinations were generated as per the provisions of NBC 105:2020 to ensure comprehen-

sive evaluation under realistic design scenarios.

### 3.1. Nonlinear modeling and pushover analysis

To address the limitations of linear-elastic analysis, this study employs nonlinear static analysis, commonly referred to as pushover analysis. As a fundamental tool in Performance-Based Design, pushover analysis provides valuable insights into the post-yield behavior of structures. This analysis generates a capacity curve, a plot of base shear against roof displacement, which reveals the building's strength and deformation capacity from elastic response, through yielding, and finally into the inelastic range.

This approach enables identification of potential failure modes and tracks the formation of plastic hinges, which are the critical indicators of seismic performance that traditional force-based design methods fail to capture. To accomplish this, inelastic behavior was modeled by assigning plastic hinges to members based on criteria outlined in ASCE 41-13 [12] and FEMA-356 [11]. Beams were modeled using flexure-controlled M3 hinges at their ends, with moment-rotation backbones and IO/LS/CP acceptance limits taken from ASCE 41-13. Columns and shear walls were modeled using Fiber P-M2-M3 hinges to capture the complex interaction of axial loads with biaxial bending, as a methodology supported by research by [5]. Fiber hinges are unique as they represent the cross-section as a collection of longitudinal fibers (concrete and steel), allowing the model to capture the complex interaction between axial loads and biaxial bending. This explains how a hinge forms in a shear wall, it is the point where the extreme concrete fibers reach crushing strain or steel fibers reach yield strain, simulating the degradation of wall stiffness. After establishing the model, a displacement-controlled pushover analysis was performed. The structure was subjected to incrementally increasing lateral loads according to its fundamental mode shape, following an initial gravity load application. This process generated the full capacity curve needed for performance evaluation.

The nonlinear static analysis produces a capacity curve, defined as the relationship between total base shear and roof displacement of the control node for monotonically increasing lateral load. This curve is then idealized by ETABS into a bilinear force-displacement (FD) relationship with an initial elastic branch and a post-yield branch that preserves energy equivalence to the original pushover curve, as required by the ASCE 41-13 Displacement Coefficient Method.

The building's seismic performance was then evaluated by the software using the Displacement-Coefficient

Method specified in ASCE 41-13 [12], which uses the bilinearized pushover curve. The resulting performance point was then compared against the predefined IO, LS, and CP performance criteria. The IO, LS and CP displacement limits (0.176 m, 0.336 m and 0.676 m respectively) are obtained from the non-linear pushover capacity curve using the performance limit states defined in ASCE 41-13 for reinforced concrete frame-shear wall systems. The ETABS implementation of the Displacement Coefficient Method (ASCE 41) automatically evaluates the roof displacement corresponding to these performance levels based on the plastic hinge properties and member force-deformation relationships assigned in the model.

## 4. Results and discussion

The seismic performance of the structure was evaluated using nonlinear static pushover analysis. Before this advanced analysis, the structural model was verified against the linear dynamic analysis requirements of the Nepal National Building Code (NBC 105:2020 [13]). This initial verification confirmed that the baseline building design is acceptable according to conventional code requirements. The key results from both the code-based checks and the subsequent nonlinear analysis are discussed below.

### 4.1. Modal analysis and code compliance

Modal analysis was conducted to identify the structure's fundamental periods of vibration and their corresponding mode shapes, both being fundamental to understanding the building's response to seismic excitation.

The modal analysis as shown in Table 1 revealed that the fundamental period of the structure (Mode 1) is 0.972 seconds, corresponding to translational motion in the X-direction. The second mode, with a period of 0.932 seconds, also represents translational motion but in the Y-direction, while the third mode at a period of 0.623 seconds, is predominantly rotational. Code compliance checks confirmed that critical parameters such as inter-story drift, eccentricity, soft-story behavior, weak-story, and torsional irregularities all remained within the permissible limits as prescribed by NBC 105:2020 [13]. Furthermore, over 90% of the building's total modal mass was found to participate within the first 12 modes in both principal directions, thereby validating the reliability of the dynamic analysis.

### 4.2. Capacity curve and performance point

The capacity curve, shown in Figure 6, plots the building's total base shear against its roof displacement. This

Table 1: Modal periods and effective mass participation ratios in each direction

Mode	Period (sec)	UX	UY	UZ	SumUX	SumUY	SumUZ	RX	RY	RZ	SumRX	SumRY	SumRZ
1	0.972	0.6372	0.0001	0	0.6372	0.0001	0	4.4E-05	0.3656	0.011	4.4E-05	0.3656	0.011
2	0.932	0.0001	0.6447	0	0.6373	0.6448	0	0.3709	4.4E-05	1.4E-06	0.371	0.3657	0.011
3	0.623	0.0113	6.8E-06	0	0.6487	0.6448	0	2E-06	0.0024	0.6248	0.371	0.3681	0.6358
4	0.242	0.1604	4.2E-06	0	0.809	0.6448	0	5.5E-06	0.2387	0.0029	0.371	0.6068	0.6387
5	0.232	4.6E-06	0.166	0	0.809	0.8108	0	0.2393	6.7E-06	7.4E-07	0.6103	0.6068	0.6387
6	0.153	0.0043	1.3E-06	0	0.8133	0.8108	0	1.9E-06	0.0063	0.1714	0.6103	0.613	0.8101
7	0.114	0.062	0	0	0.8754	0.8108	0	7.3E-07	0.0964	0.0026	0.6103	0.7094	0.8127
8	0.109	5.6E-07	0.0692	0	0.8754	0.88	0	0.1072	9E-07	0	0.7175	0.7094	0.8127
9	0.074	0.0236	5.5E-07	0	0.899	0.88	0	9.1E-07	0.0487	0.0302	0.7175	0.7581	0.8429
10	0.071	0.015	0	0	0.914	0.88	0	0	0.0317	0.038	0.7175	0.7898	0.8809
11	0.069	0	0.0417	0	0.914	0.9217	0	0.0876	0	0	0.8051	0.7898	0.8809

curve illustrates the structure’s behavior as it transitions from its initial elastic stiffness to a state of inelasticity under increasing lateral load.

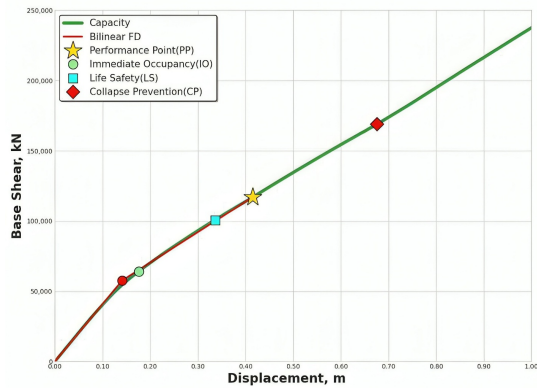


Figure 6: Obtained pushover curve

Key structural performance levels as identified along this curve are:

- Immediate Occupancy (IO): 0.176 m
- Life Safety (LS): 0.336 m
- Performance Point (PP): 0.414 m
- Collapse Prevention (CP): 0.676 m

In Figure 6, the green line represents the raw capacity curve obtained directly from the pushover analysis, while the bilinear red curve is the idealized FD representation used in the displacement-based evaluation. ETABS applies the ASCE 41 Displacement Coefficient Method to this bilinear curve, together with the elastic design spectrum, to compute the target displacement (performance point) at which the expected seismic demand equals the nonlinear capacity. Although ASCE 41-13 allows representation in ADRS capacity-demand format, in this study the performance point is obtained graphically through the Displacement Coefficient Method implemented in ETABS rather than by a

direct intersection of separately plotted capacity and demand curve. The structure’s performance point, which signifies the expected maximum displacement under the design earthquake, was obtained at 0.414 m, lies safely between the LS and CP threshold values. This indicates that, under design-level seismic demand, the structure may experience repairable damage by maintaining its overall stability and without approaching collapse, thereby fulfilling the Life Safety objective.

### 4.3. Plastic hinge formation and damage progression

The sequence and location of damage are critical to evaluating the seismic performance. Figure 7 shows that the initial plastic hinge appeared in the beam at the top level of the building, which is an early indicator of the intended ductile behavior.

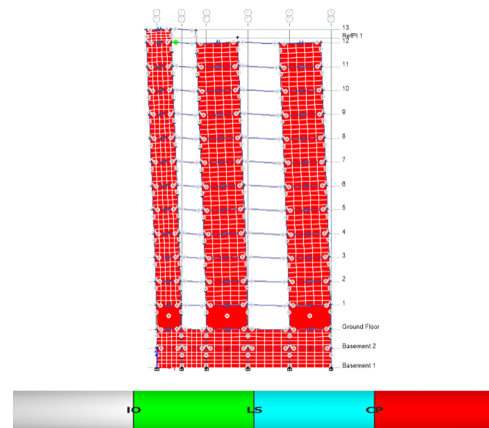


Figure 7: Formation of first plastic hinge

The Performance Point damage state is shown in Figure 8, which illustrates the building’s overall deformed shape and the distribution of plastic hinges. Qualitative assessment of the model immediately indicates that inelastic behavior is largely restricted to the beams, with most of the hinges within the green (Immediate Occu-

pancy) range.

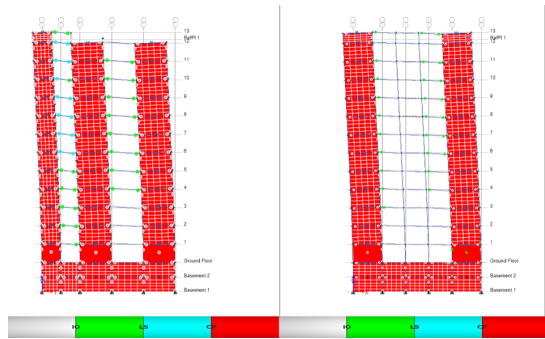


Figure 8: Formation of plastic hinges at the performance point

This observation is quantitatively confirmed by the results presented in Figure 9. The bar chart separates the hinge formation by element type and indicates that the vast majority of the plastic hinges (67) occur in the beams. In stark contrast, only one hinge formed in a column and one in a shear wall, both within the minimal Immediate Occupancy damage state.

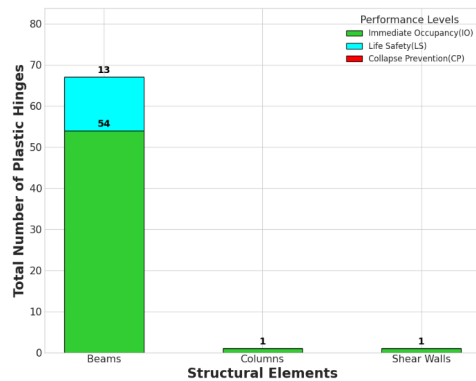


Figure 9: Distribution of plastic hinges by structural element type and performance levels

This cumulative evidence from both the visual model and the quantitative data presents definitive proof of a successful "strong-column, weak-beam" design philosophy. Through ductile yielding being induced into the beams, the integrity of the primary vertical load-carrying system is preserved, which is of the utmost importance in ensuring overall structural safety.

#### 4.4. Quantitative summary of damage severity

The donut chart as shown in Figure 10 reveals that, out of 69 total plastic hinges, 81% (56 hinges) remained within the IO range, 19% (13 hinges) reached the LS range and, 0% reached the CP state. This complete absence of

Collapse Prevention hinges at the performance point further reinforces the safety of the structure.

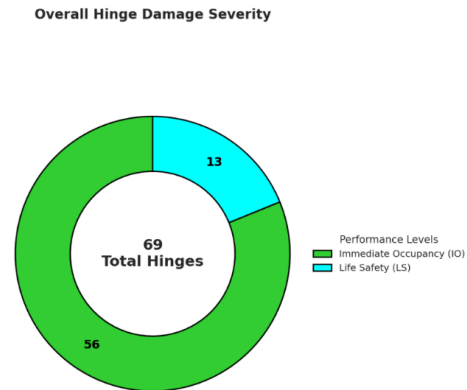


Figure 10: Overall damage severity at the performance point

#### 4.5. Story-wise hinge distribution

The distribution of plastic hinges along the building height, as illustrated in Figure 11, shows that inelastic behavior is not concentrated at a single level but rather spread across multiple floors, particularly between stories 4 and 13.

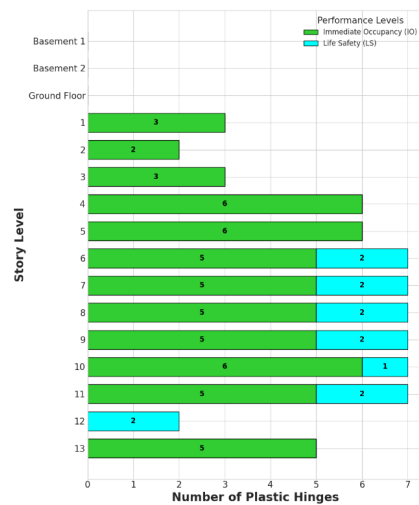


Figure 11: Story-wise distribution of plastic hinges at different performance levels

This pattern is desirable, as it reduces the likelihood of a soft-story failure by avoiding localized overstressing. While several hinges in stories 6 through 12 reached the Life Safety level, their dispersion indicates a balanced and ductile response throughout the height of the

structure, rather than any critical weakness in a specific story.

## 5. Conclusion

This research evaluated the seismic performance of a 15-story RC building with two basements using pushover analysis.

The main conclusions are:

1. The analysis verified compliance with the requirements of NBC 105:2020 in terms of drift, torsional irregularity, eccentricity, stiffness, and strength.
2. The nonlinear static(pushover) analysis identified the building's performance point at 0.414 m, which lies safely between the Life Safety (LS) threshold of 0.336 m and the Collapse Prevention (CP) limit of 0.676 m. This confirms that the structure meets the targeted Life Safety performance under design-level seismic loading.
3. The failure mechanism revealed a ductile response governed by the desired "strong-column, weak-beam" behavior. Plastic hinges formed predominantly in beams, while columns and shear walls remained mostly elastic or minimally damaged. This hinge distribution confirms the effectiveness of the structural design in preventing brittle or soft-story failures.

Recommendations are listed as:

- The use of static analysis does not fully capture the transient effects of ground motions. Future research should employ Nonlinear Time History Analysis.
- Incorporating Soil-Structure Interaction is important for buildings on soft soil (Type D) to more accurately predict displacement demands.

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