

# Performance-Based Seismic Assessment of a G+30 Tall RC Commercial Complex Using Non-Linear Static Pushover Analysis

## A Performance Based Seismic Evaluation of a High Rise Special Moment Resisting Frame

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**Abstract** - This study evaluates the seismic performance and safety of a G+30 storey reinforced concrete commercial complex in Ahilyanagar, Maharashtra. Utilizing ETABS, a non-linear static pushover analysis was conducted by subjecting the structural model to incremental lateral loads to trace plastic hinge formation and identify potential failure mechanisms. Key response parameters—including maximum base shear, roof displacement, storey drift, and lateral stiffness—were evaluated against IS 1893:2016 requirements. By tracking structural behavior beyond the elastic limit, the building's discrete performance states were mapped to Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) thresholds, ensuring an optimized, ductile, and seismically secure high-rise design.

**Keywords** - Pushover analysis; non-linear static analysis; plastic hinges; high-rise buildings; performance-based design; storey drift; base shear; structural ductility.

### I. INTRODUCTION

#### A. Introduction:

Seismic design is currently undergoing a significant transformation. We are moving from a stage where elastic analysis was considered sufficient for both elastic and ductile design to a more dedicated non-linear procedure that influences the seismic design as a whole. As buildings increase in height and complexity, such as the G+30 commercial complex analysed in this study, the need for a more effective method to predict non-linear behaviour becomes paramount. This method is popularly known as Pushover Analysis (PA).

Pushover Analysis is a non-linear static procedure used to estimate the strength capacity of a structure beyond its elastic limit, reaching into its ultimate strength in the post-elastic range. Unlike conventional seismic analysis, which assumes a constant initial time period, PA continuously recalculates the structural properties as the analysis progresses and damage occurs. This is achieved by inserting non-linear "hinges" into the structural model to simulate the sequence of damages in each member.

Current static equivalent lateral force procedures can often be overly conservative, potentially leading to uneconomical designs. By utilizing advanced software packages like STAAD.Pro and ETABS, engineers can more accurately demonstrate how progressive failure occurs and identify the final mode of failure. This project aims to utilize these tools to satisfy serviceability conditions and optimize the seismic performance of a high-rise structure within the Indian seismic zones.

#### B. Problem Statement :

The seismic design of high-rise structures, specifically a G+30 commercial complex, necessitates a transition from traditional prescriptive methods to advanced performance-based evaluation. While standard codes primarily utilize Linear Elastic Analysis for design, this approach presents several critical gaps when applied to tall, complex structures:

**Invisibility of Damage Progression:** Linear analysis assumes that all structural members remain within their elastic limits. Consequently, it cannot predict the sequence of failure or identify the specific locations where plastic hinges will form during a major seismic event. This leaves a significant uncertainty regarding the "Strong Column-Weak Beam" mechanism.

**Limitation of Static Assumptions:** Conventional seismic analysis methods often rely on a simplified distribution of lateral forces. For a 30-story building, these methods may fail to capture the complex dynamic behaviour and the redistribution of internal forces that occur once the structure enters the non-linear range.

**Quantifying Reserve Strength:** There is a lack of clear data regarding the "over-strength" or reserve capacity of the structure beyond its initial yield point. Without a non-linear assessment, it is difficult to verify if the building truly possesses the ductility required to survive intense ground shaking without collapse.

**Performance Level Verification:** Modern engineering demands that buildings meet specific performance targets (such as Life

Safety or Collapse Prevention). Linear methods do not provide a direct measure of the structural deformation (drift) and damage state associated with these critical safety thresholds.

### C. Objective:

The primary goal of this research is to perform a comprehensive seismic evaluation of a high-rise structure using non-linear static procedures. The specific objectives are as follows:

1. To Model and Design a High-Rise Structure: To develop a detailed 3D mathematical model of a G+30 commercial complex and perform initial design according to IS 456:2000 and IS 1893:2016.
2. To Perform Modal Analysis: To determine the dynamic characteristics of the building, including fundamental natural periods and mode shapes, ensuring that the modal mass participation meets the 90% regulatory requirement.
3. To Conduct Non-linear Static (Pushover) Analysis: To apply incremental lateral loads to the structure in order to generate the Capacity Curve (Base Shear vs. Roof Displacement).
4. To Identify Plastic Hinge Formation: To monitor the sequence of yielding in structural members and categorize the building's damage state according to performance levels: Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP).
5. To Evaluate Local and Global Ductility: To assess the structure's ability to dissipate energy and identify potential weak stories or "soft story" mechanisms that may lead to failure.

### D. Scope of the project

The scope of this project is centered on the non-linear seismic assessment of high-rise reinforced concrete structures. The study is bounded by the following parameters:

**Structural System:** The study is limited to a G+30 storey Reinforced Concrete (RC) commercial building. The analysis focuses on a Moment Resisting Frame (MRF) system with specific attention to beam-column junctions.

**Analysis Type:** The primary focus is on Non-linear Static Analysis (Pushover Analysis). While initial designs are performed using Linear Static and Dynamic (Modal) methods, the core research involves displacement-controlled "push" procedures.

**Software Platforms:** The modelling and analysis performed in ETABS. The scope includes the evaluation of result generated by specific computational engines.

**Geographic & Seismic Parameters:** The building is designed considering the seismic requirements of Zone III (or as per your specific site location) in India, following the criteria defined in IS 1893 (Part 1): 2016.

**Material Non-linearity:** The scope includes the definition of Discrete Plastic Hinges. Non-linearity is modelled at the ends of frame elements (Beams and Columns) using Moment-Rotation (M- $\theta$ ) and P-M-M interaction properties.

**Performance Evaluation:** The assessment is limited to identifying structural performance levels—Immediate

Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP)—based on global drift and hinge status.

**Soil Conditions:** The analysis considers Type III (Soft Soil) conditions, focusing on how soil characteristics influence the fundamental time period and the resulting capacity curve.

### E. Building Specifications

Name of the City Where this building is being constructed :Ahilyanagar

Zone Factor  $Z = 0.16$

Importance Factor: 1.2

Type of Soil: Soft

Horizontal Dimension of the building along Horizontal axis = 50.25

Lateral Dimension of the building along Horizontal axis = 40.75

Height of the Building, as per IS 1893:2016 = 105.6m

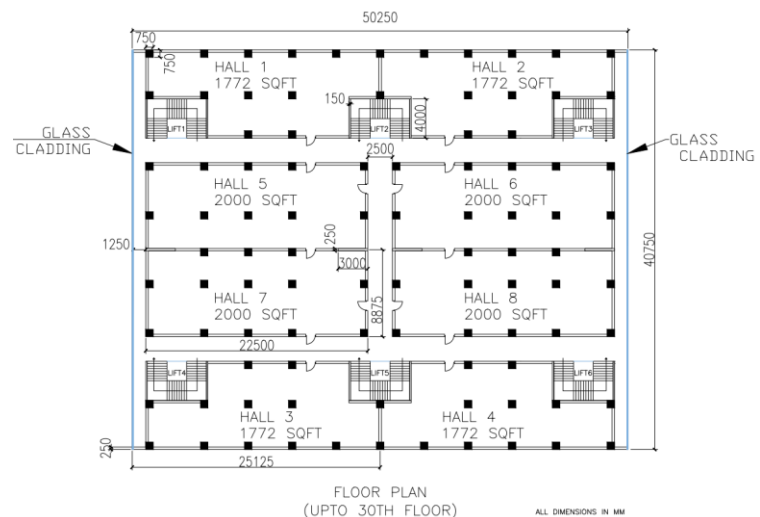
Type of Frame : No Brick Walls or Negligible

Time period = 2.85 sec

Response reduction factor (R) = 5

Building Type : RC Building with special moment resisting frame (SMRF)

### F. Floor Plan



## II. METHODOLOGY

### A. Methodology :

The following steps are executed systematically in ETABS to ensure a valid comparative analysis.

#### Step 1: Geometric Modelling and Material Mapping

The physical characteristics of the G+30 structure are defined.

**Structural Framing:** Beams and columns are modelled as 1D frame elements, while shear walls are modelled as shell or pier elements.

**Material Non-linearity:** The software prepares a "Non-linear Material Table" to handle the stress-strain behaviour of concrete (M30/M35) and steel (FE500) beyond the yield point.

**Diaphragm Assignment:** Each of the 30 floors is assigned a Rigid Diaphragm to ensure accurate lateral mass distribution to the vertical frames.

#### Step 2: Gravity Load Stabilization (P-Δ Pre-check)

Before the lateral "push" is initiated, the structure must be stabilized under vertical loads.

Initial State: A non-linear gravity load case (1.0DL + 0.5LL) is applied.

P-Δ Effects: Given the building's height, second-order geometric non-linearity (P-Δ) is activated. The vertical loads act on the displaced coordinates, which is essential for capturing the true stability of a high-rise.

#### Step 3: Characterization and Assignment of Non-linear Plastic Hinges

This phase involves the transition from a continuous elastic model to a discrete "Lumped Plasticity" model.

Lumped Plasticity Formulation: Plastic hinges are assigned to the proximal and distal ends of frame elements where bending moments are maximized.

Beam Elements (Flexural Dominant): Assigned Uniaxial Moment Hinges (M3) governed by a pre-defined Moment-Rotation (M-θ) relationship.

Column Elements (Axial-Flexural Interaction): Assigned Multi-component (P-M2-M3) Interaction Hinges, as column yielding is sensitive to fluctuating axial loads.

Constitutive Backbone Definition: Each hinge follows a multi-linear "Force-Deformation" curve (e.g., FEMA 356/ATC-40) defining states from Yield (B) to Ultimate (C) and Residual Strength (D/E).

#### Step 4: Modal Extraction and Load Pattern Selection

To push the building realistically, the analysis must account for its dynamic properties.

Eigenvalue Analysis: The software determines the fundamental mode shape (the first eigenvector).

Load Pattern: A lateral load profile is generated proportional to the product of the floor mass and the fundamental mode shape.

#### Step 5: Setting the Displacement Control Parameters

The analysis utilizes Displacement Control rather than force control.

Control Node: The centre of mass at the roof level is selected as the monitoring node.

The "Push": The software applies the lateral load pattern in small increments. After each step, the stiffness matrix [K] is recalculated to account for hinges that have entered the inelastic range.

#### Step 6: Determination of the Capacity Curve

At each increment, the software records the relationship between:

1. Total Base Shear (V<sub>b</sub>): The sum of lateral reactions.
2. Roof Displacement (Δ): The movement of the control node.

These points are plotted to create the Capacity (Pushover) Curve.

#### Step 7: Performance Point Assessment (Demand vs. Capacity)

The Capacity Curve is intersected with the Seismic Demand Spectrum (based on site-specific parameters).

Performance Point: This intersection represents the maximum displacement the building is expected to undergo during a design-level earthquake.

Safety Verification: The status of all hinges (IO, LS, CP) is reviewed at this point to confirm the structure meets the target safety criteria.

### III. LOADS

#### A. Dead Load:

Wall thickness = Internal 150mm External 250mm

Steel = 78.5 kN/m<sup>3</sup>

Concrete = 25 kN/m<sup>3</sup>

Glass = 25 kN/m<sup>3</sup>

Density of Flooring = 24 kN/m<sup>3</sup>

Density of Water proofing = 10 kN/m<sup>3</sup>

Terrace Floor = 150 mm water proofing.

Density of water Flooring = 24kN/m<sup>3</sup>

Density of Filling: 10 kN/m<sup>3</sup>

Density of water Proofing = 10kN/m<sup>3</sup>

#### A. Live Load:

Hall = 3.5 kN/m<sup>2</sup>

Balcony = Same as rooms to which they give access but with a minimum of 4.0

Passage = 4 kN/m<sup>2</sup>

Lift = 5 kN/m<sup>2</sup>

Staircase = 4 kN/m<sup>2</sup>

Terrace = 1.5 kN/m<sup>2</sup>

Toilet = 2 kN/m<sup>2</sup>

Live Load Reduction factors as per IS 1893:2016 Part 1) = 0.5 (Clause 7.3.2 and Table 10)

#### B. Load of Overhead Water Tank:

Dimension = 8.875 X 17.75 X 2.7 = 335.475 m<sup>3</sup>

Density of water = 1000 Kg/m<sup>3</sup>

Capacity of water tank = 3,35,475 litre

Total Water Load = 3354.75 kN

Hydrostatic Pressure on Base Slab = 10 kN/m<sup>3</sup> x 2.7 m = 27.0 kN/m<sup>2</sup>

#### C. Wind Load:

V<sub>b</sub> = 39

Terrain Category 2

Risk Coefficient (K<sub>1</sub>) = 1

Height Multiplier (K<sub>2</sub>) = 1.26

Topography Factor (K<sub>3</sub>) = 1.36

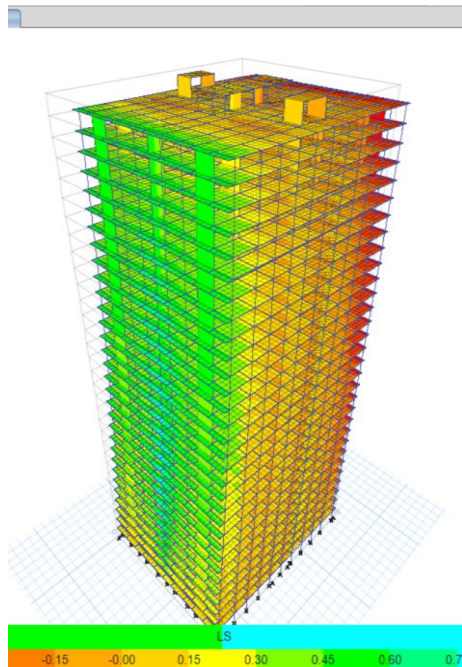
Importance Factor (K<sub>4</sub>) = 1

K<sub>d</sub> = 0.9

K<sub>a</sub> = 0.8

K<sub>c</sub> = 0.9

#### IV. DEFORMED SHAPE OF MODEL



Deformed Body Shape for push X load Condition

#### VI. HINGES APPLICATION

##### A. Application of Plastic Hinges in Nonlinear Analysis

In performance-based seismic design, structural members are permitted to undergo post-elastic deformations to dissipate energy during severe earthquake events. To simulate this behavior in a Static Nonlinear Pushover Analysis, discrete Plastic Hinges are assigned to critical locations along the structural frame elements (typically at the column ends and beam faces where bending moments peak).

##### 1. Hinge Mechanics and Force-Deformation Relationships

The mathematical behaviour of a plastic hinge is governed by a force-deformation (or moment-rotation) relation derived from standard guidelines such as ASCE 41 or FEMA 356. The rigid-plastic or elasto-plastic curve is defined by key operational thresholds:

- Point A to B (Linear Elastic Range): Represents the uncracked, elastic behaviour of the element. Point B defines the actual nominal yield strength ( $M_y$ ).
- Point B to C (Strain Hardening/Plastic Range): Represents the post-yield capacity where the element deforms plastically up to its ultimate capacity (Point C).
- Point C to D (Strength Degradation): Captures the sudden drop in resistance due to concrete crushing or rebar buckling.
- Point D to E (Residual Strength): Reflects the ultimate residual capacity of the member before complete structural failure at Point E.

##### 2. Performance-Based Acceptance Criteria

To evaluate the safety of the 29-story tower under varying lateral drift demands, the hinges are monitored against three distinct Performance Levels marked on the post-yield plateau:

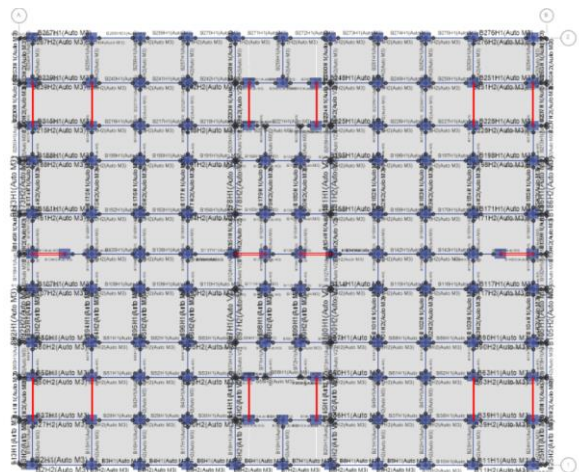
**Immediate Occupancy (IO):** Minimal structural damage; the lateral force-resisting system retains nearly all of its pre-earthquake strength and stiffness. Safe for immediate re-entry.

**Life Safety (LS):** Significant damage to structural components, but a significant margin against local or global collapse remains. Main structural hazards are mitigated.

**Collapse Prevention (CP):** The building continues to support gravity loads but retains little to no residual lateral stiffness. The structure is near total failure and represents a total constructive loss.

By evaluating the step-by-step hinge registry output from the solver, the sequential formation of these hinges can be tracked. This ensures that the tower follows a preferred "strong-column/weak-beam" plastic mechanism, confirming a ductile failure path over its full 105.6m height.

Hinges are applied at 0.15 percent distance from both end

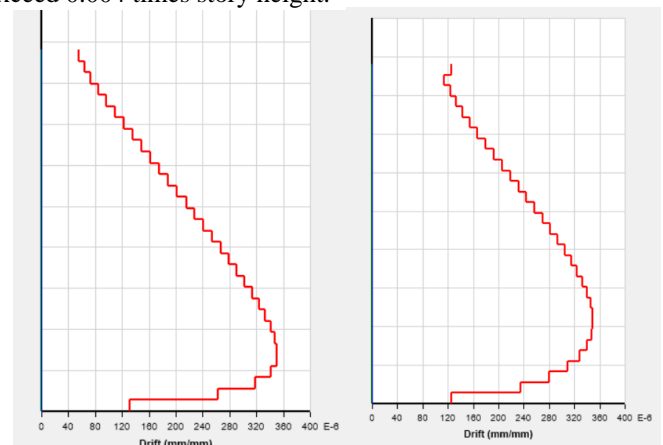


Plan view of hinges at typical story

#### VII. RESULTS AND DISCUSSION

**A. Story drift:** It is the lateral displacement of one level relative to the level above or below. Story drift ratio is the story drift divided by the story height.

According to IS 1893 Part I Cl 7.11.1 story drift shall not exceed 0.004 times story height.



Maximum Story Drift in X direction =  $350 \times 10^{-6}$   
 Maximum Story Drift in Y direction =  $340 \times 10^{-6}$   
 Maximum Story Drift allowed = 0.004

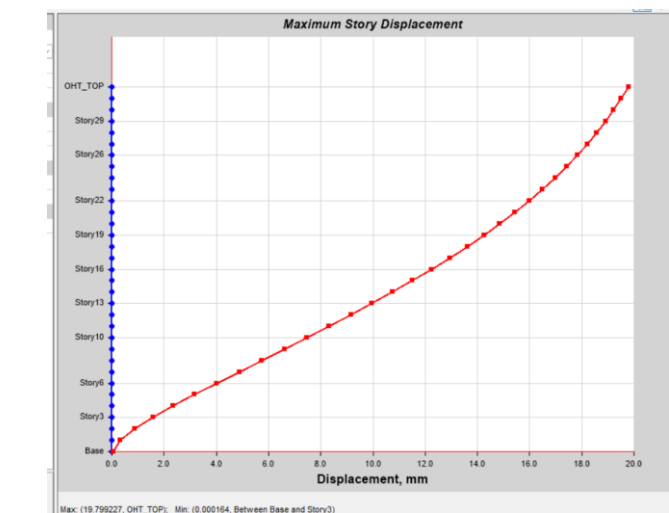
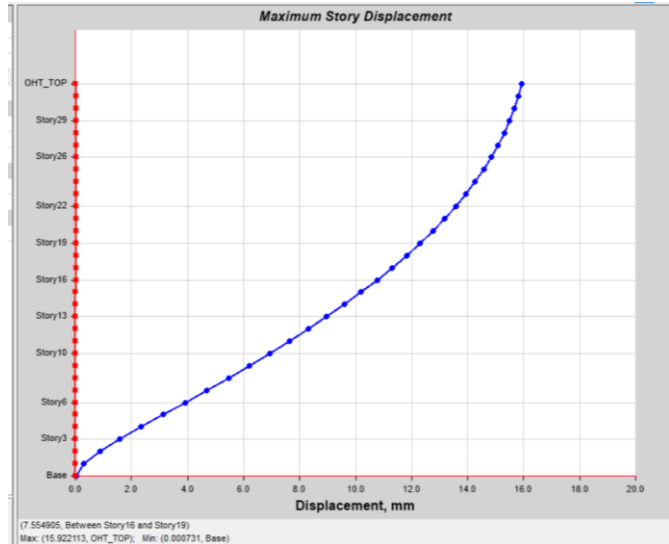
.....Hence OK

B. *Storey Displacement* : Story displacement is the deflection of a single story relative to the base or ground level of the structure.

Allowable story displacement

$$\Delta_{max} = H/500$$

$$105600/500 = 211.2 \text{ mm}$$



Maximum Story Displacement for Push Y Load case

Maximum Story Displacement in X direction = 15.92 mm

Maximum Story Displacement in Y direction = 19.8 mm

C. *Nonlinear hinge performance and acceptance criteria:*

Rigorous evaluation of the structure's nonlinear performance was conducted by extracting the plastic hinge response database across all incremental solution steps of the lateral static pushover analysis. The structural components were evaluated based on their internal force demands—specifically axial force (P) and biaxial bending moments ( $M_2$ ,  $M_3$ )—relative to their operational deformation thresholds as defined by performance-based design paradigms (e.g., ASCE 41-17 and FEMA 356 guidelines).

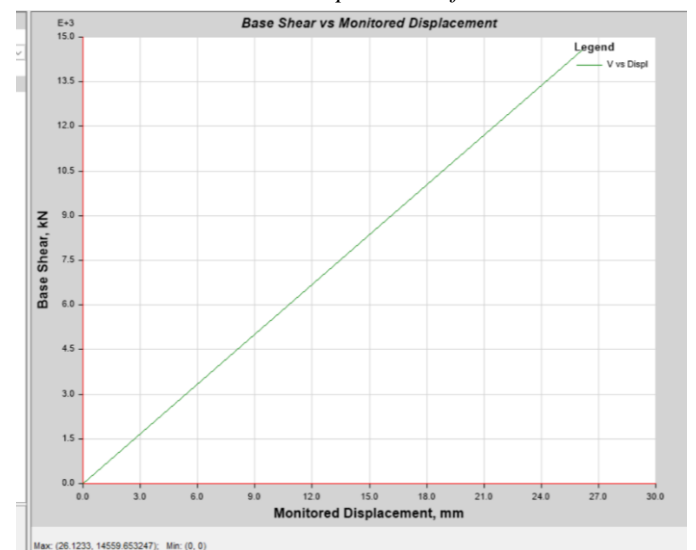
On X-Axis (Monitored Displacement) Measures the lateral displacement of a control node (usually the roof or highest level) in millimeters (mm).

On Y-Axis (Base Shear): Measures the total accumulated lateral reaction force at the base of the structure in kilonewtons (kN).

The structure is currently behaving entirely elastically within this displacement range 0 to 23.4 mm)



*Base Shear V/s Monitored Displacement for Push Y load case.*



As the lateral displacement demands scale incrementally throughout the pushover sequence, the critical frame elements experience a significant escalation in internal forces. At the maximum evaluated step (Step 3), the structure is subjected to a severe base shear force demand of 14,559.65 kN coupled with a monitored lateral displacement of 26.123 mm.

Despite the severity of these simultaneous force interactions, the overall element network exhibits robust structural performance under lateral seismic demands. Throughout the entire tracking sequence, the behavioral

classification for the vast majority of components remains strictly within the A to  $\leq B$  domain, with 24,328 hinges remaining fully elastic. In standard structural plasticity mechanics, Point B marks the boundary where a material transitions from a linear-elastic state to a non-recoverable plastic yield state. While a minor subset of 36 hinges successfully transitions into the B-C yielding phase, the overwhelming majority of the cross-sections maintain their structural integrity and experience no widespread permanent plastic degradation.

**Tabulated Plot Coordinates**

**Capacity Curve Coordinates (Part 1 of 2)**

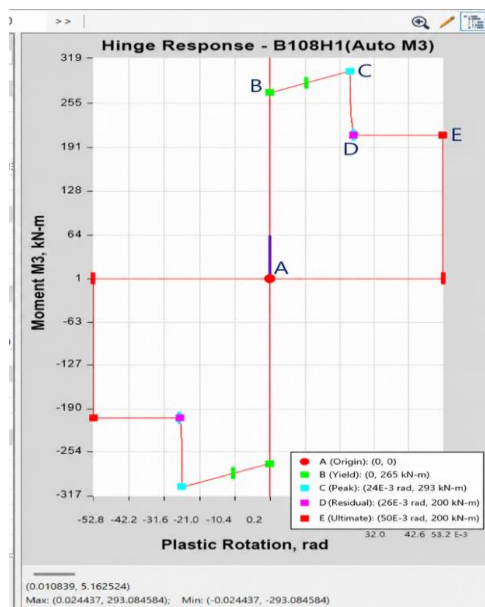
Step	Monitored Displ mm	Base Force kN	A-B	B-C	C-D	D-E
0	0	0	24364	0	0	0
1	19.696	10991.3419	24352	12	0	0
2	26.123	14559.6532	24328	36	0	0
3	26.123	14559.6532	24328	36	0	0

>E	A-IO	IO-LS	LS-CP	>CP	Total Hinges
0	24364	0	0	0	24364
0	24364	0	0	0	24364
0	24364	0	0	0	24364
0	24364	0	0	0	24364

Consequently, the operational performance status of the components is confirmed to be well within the Immediate Occupancy (IO) safety band, with all 24,364 hinges safely retained in this category. This indicates that under full design-level lateral push forces, the localized structural components possess a high degree of reserve capacity, suffering negligible cracking or crushing. The building remains structurally stable, satisfying both serviceability and performance-based safety requirements with a commendable margin against collapse.

**D. Hinge Response:**

**1. PUSH X**



Moment M<sub>3</sub> vs Plastic Rotation for Push X Load Case

Point A to B (Elastic Range): The vertical line passing through (0,0) represents the elastic behaviour before yielding.

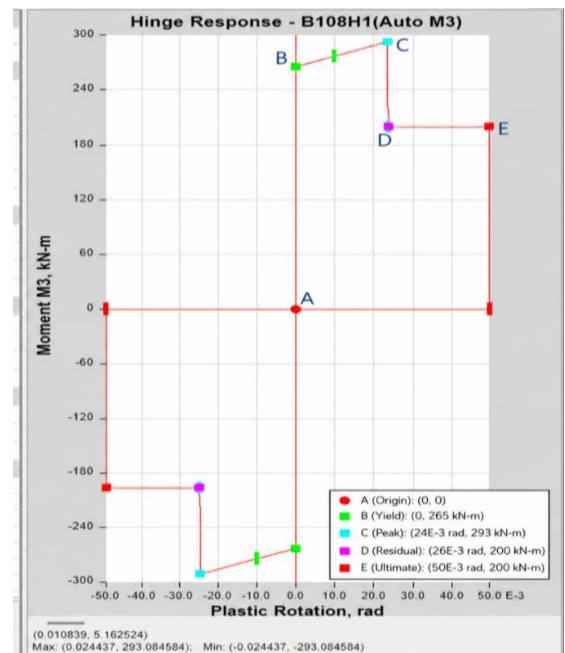
Point B (Yield Point): Yielding occurs at approximately 265kN-m (indicated by the small green tick marks near zero rotation).

Point C (Ultimate Capacity): The peak moment capacity reaches roughly 293kN-m at a plastic rotation of approximately 24.4 x 10<sup>-3</sup>rad (indicated by the light blue/cyan square data points).

Point D (Residual Strength Drop): After passing Point C, the strength drops sharply down to a residual moment capacity of about 200 kN-m (magenta square data points).

Point E (Ultimate Rotation Limit): The curve terminates horizontally at a plastic rotation limit of 50.0 X 10<sup>-3</sup> rad

**2. Push Y**



Point A to B (Elastic Range): The vertical line passing through (0,0) represents the elastic behaviour before yielding.

Point B (Yield Point): Yielding occurs at approximately 265kN-m (indicated by the small green tick marks near zero rotation).

Point C (Ultimate Capacity): The peak moment capacity reaches roughly 293kN-m at a plastic rotation of approximately 24.4 x 10<sup>-3</sup>rad (indicated by the light blue/cyan square data points).

Point D (Residual Strength Drop): After passing Point C, the strength drops sharply down to a residual moment capacity of about 200 kN-m (magenta square data points).

Point E (Ultimate Rotation Limit): The curve terminates horizontally at a plastic rotation limit of 50.0 X 10<sup>-3</sup>rad (red square data points).

## VIII. CONCLUSION

The comprehensive seismic evaluation and performance-based assessment of the G+30 commercial complex in Ahilyanagar, Maharashtra, was successfully executed using Non-linear Static Pushover Analysis in ETABS. Based on the detailed analytical simulations, load interactions, and structural responses, the following conclusions are drawn:

**Structural Grid & Serviceability Compliance:** The high-rise structure, rising to a total height of 105.6 m, exhibits excellent structural stiffness and drift control. The maximum story drift recorded in the X-direction is  $350 \times 10^{-6}$  and in the Y-direction is  $340 \times 10^{-6}$ . Both values are comfortably within the maximum permissible serviceability limit of 0.004 times the story height (i.e. 13.2 mm) specified by IS 1893 (Part 1): 2016.

**Story Displacement Bounds:** The maximum horizontal story displacement computed at the topmost monitoring node (OHT\_TOP) under the incremental lateral load case is 15.92 mm for Push X and 19.80 mm for Push Y. These displacements are well below the strict structural code limitation of  $H / 500 = 211.2$  mm, demonstrating highly adequate lateral stiffness against seismic demands.

**Performance Level Assessment:** The non-linear demand-versus-capacity assessment explicitly verifies that the building meets the performance objectives. Under the ultimate evaluated step (Step 3), which subjected the model to a massive base shear demand of 14,559.65 kN, 24,328 hinges remained completely elastic within the A–B domain. A minor subset of only 36 hinges transitioned into the plastic yielding phase (B–C).

**Ductile Failure Mechanism:** Zero hinges crossed the critical threshold into the strength degradation (C–D), residual (D–E), or collapse (>CP) zones. All 24,364 plastic hinges successfully remained within the Immediate Occupancy (IO) safety band. This satisfies the performance criteria, confirming a highly desirable "Strong Column-Weak Beam" sequence that provides an enormous safety margin against collapse.

**Reserve Capacity Verification:** The capacity curves generated (Base Shear vs. Monitored Displacement) highlight a substantial reserve strength in the post-yield plastic state beyond conventional linear assumptions. The structure effectively utilizes its Special Moment Resisting Frame (SMRF) configuration (R=5) to handle extreme load redistributions efficiently without major material cracking or geometric instability.

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