

## Research Article

# Comparative Lateral Loads Performance Evaluation of a Highly Slender High-rise Building Under Different Structural Configurations

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## Abstract

This study evaluates the lateral-load performance of a highly slender, 90-storey reinforced-concrete high-rise building under three structural configurations, focusing on serviceability, global stiffness and force demand. The building has a nominal height of 310.50 m, a plan width of 24 m and a height-to-width ratio of 12.9375. Case 1 is a reference multiple-tube structure without outriggers. Case 2 incorporates outriggers at the 30th and 60th floors. Case 3 retains the outriggers and revises the beam dimensions from 400 × 600 mm to 600 × 600 mm and column dimensions from 600 × 600 mm to 900 × 1200 mm. The configurations were analysed using ETABS, with wind and seismic actions considered in accordance with IS 875 (Part 3) and IS 1893 (Part 1), respectively. Performance was compared using maximum storey displacement, drift ratio, upper-level acceleration, modal periods, earthquake and wind base shear, overturning moment and modal response. Maximum displacement decreased from 1039.431 mm in Case 1 to 900.385 mm in Case 2 (13.38% reduction) and 615.833 mm in Case 3 (40.75% reduction). Maximum drift ratios were 0.001057, 0.000957 and 0.000784, respectively; the Case 3 reduction relative to Case 1 was 25.83%. Story 90 accelerations decreased from 0.210 m/s<sup>2</sup> to 0.193 m/s<sup>2</sup> and 0.151 m/s<sup>2</sup>, respectively. The fundamental period declined from 12.093 s to 11.315 s and 10.301 s, indicating increased global stiffness. Under EQX loading, base shear increased from 23,232.83 kN to 23,467.89 kN and 27,797.57 kN, while wind base shear decreased from 35,064 kN to 34,552 kN and 33,865 kN. Base overturning moment increased from 5.431 × 10<sup>6</sup> kN m in Case 1 to 5.477 × 10<sup>6</sup> kN m and 6.487 × 10<sup>6</sup> kN m. Case 3 provided the greatest reductions in displacement, drift and acceleration, but its higher seismic base shear and overturning demand require careful verification of member and foundation capacities.

## Keywords

High-rise Building, Outrigger System, ETABS, Storey Displacement, Storey Drift, Acceleration, Base Shear, Overturning Moment

## 1. Introduction

Rapidly increasing activities of high-rise constructions, emphasis of structural design choice has shifted from strength

alone for resisting gravity load to controlling lateral displacement, inter-storey drift, acceleration, torsional response and

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overturning demand. The effects get more and more pronounced as the height and slenderness of the structure increase. The current study investigates three completed versions of the same 90-storey multi-tube structure. The configuration without outriggers is Case 1. Case 2 has outriggers at 30th floor and 60th floor. Case 3 is a revised Case 2 having new member sizes of  $600 \times 600$  mm beams and  $900 \times 1200$  mm columns. The maximum displacement, the drift, the acceleration, the storey shear, the overturning moment, and the modal periods and mass participation are used for comparison.

### 1.1. Problem Statement and Research Need

The short dimension along the plan of the selected building is 24 m and the nominal height is 310.50 m which gives it a height-to-width ratio  $= 310.50/24 = 12.9375$ . Such slenderness may lead to long natural periods and large roof motion. Thus, there is need of a design intervention, which reduces displacement and drift imposed in the structures without introducing excess force concentration. The research need is to quantify the structural advantage of two-tier outriggers through outputs from ETABS, which are directly comparable.

### 1.2. Objectives

- 1) To determine the lateral behavior of 90 storey core structure with no parts added, which will act as a reference comparison point.
- 2) To analyze the effect of the outriggers on the lateral stiffness, lateral displacements, accelerations, drift, and overturning characteristics of the building when fixed at 30th and 60th levels of the structure.
- 3) To control drift and lateral acceleration through the usage of outriggers and changing the cross sections of the structure.
- 4) To make a comparison on three versions of rigid structures which include pure building with added outriggers and the one with modified cross-sections subjected to various analysis for wind and earthquake response.
- 5) To calculate drift, lateral displacements, accelerations, base shear, overturning moments, modal time-periods, and modal participation of the system.

## 2. Related Work

Tall building systems based on outriggers have been the

subject of a lot of research because they offer an effective way of connecting perimeter columns into resisting overturning moments. The original concept of damped outrigger was proposed by Smith and Willford [1], while Chen et al. [2] presented dynamic response changes due to the influence of outriggers. Later studies focused on optimizing outrigger positioning, negative stiffness approach, and using integrated damping methods [5-7]. Further studies confirmed the importance of the concept of passive vibration control in wind-sensitive buildings [3, 4, 8, 9]. Although the cases studied above do not involve active methods, this allows to demonstrate the differences between stiffness-based and damping methods. Recent research tends to concentrate on combined control systems and taking uncertainties into account in engineering designs [10-13].

Recent investigations further demonstrate the range of vibration-control strategies available for slender high-rise structures. Wang et al. [7] examined adaptive negative stiffness and damping, whereas Sarkar and Fitzgerald [8] investigated tuned mass damper–fluid inerter design. Shirgir et al. [9] emphasized reliability-based tuning under soil–structure interaction uncertainty, and Abrass and Lavan [10] considered outrigger-based inertial damping for seismic performance. Further research on negative-stiffness damped outriggers, tuned mass damper–inerters systems, and viscous dampers integrated with outriggers is represented by Liu et al. [11], Calayir et al. [12], and Hope et al. [13], respectively. Unlike these supplemental damping approaches, the present research uses ETABS to compare the lateral-load response of three 90-storey reinforced-concrete structural configurations. The Bureau of Indian Standards (BIS) specifies reinforced-concrete design requirements in IS 456:2000 [14]. The Bureau of Indian Standards provides dead-load provisions through IS 875 (Part 1):1987 [15]. The Bureau of Indian Standards defines imposed-load requirements in IS 875 (Part 2):1987 [16]. The Bureau of Indian Standards establishes wind-loading criteria under IS 875 (Part 3):2015 [17]. The Bureau of Indian Standards describes earthquake-resistant design requirements in IS 1893 (Part 1):2016 [18]. The Bureau of Indian Standards specifies high-strength reinforcement requirements under IS 1786:2008 [19]. Computers and Structures, Inc. (CSI) [20] provides ETABS, the structural modelling and analysis software used to evaluate the present building configurations. This provides a consistent basis for assessing storey displacement, drift, acceleration, modal periods, base shear, and overturning moment under wind and seismic actions.

**Table 1.** Selected studies relevant to outrigger and passive-control systems.

| Ref. | Author/Year              | Topic                    | Method                               | Principal relevance                                                           |
|------|--------------------------|--------------------------|--------------------------------------|-------------------------------------------------------------------------------|
| 1    | Smith and Willford, 2007 | Damped outrigger concept | Analytical tall-building formulation | Established core-perimeter interaction and damping-enhanced outrigger action. |

| Ref. | Author/Year                 | Topic                                   | Method                                     | Principal relevance                                                         |
|------|-----------------------------|-----------------------------------------|--------------------------------------------|-----------------------------------------------------------------------------|
| 2    | Chen et al., 2010           | Tall buildings with damped outriggers   | Dynamic analytical modelling               | Quantified response reduction and sensitivity to outrigger parameters.      |
| 3    | Kim et al., 2008            | Wind response with                      | Wind-induced response analysis             | Demonstrated effective upper-level passive vibration control.               |
| 4    | Giaralis and Petrini, 2017  | TMD-inerter system                      | Optimization under wind excitation         | Improved vibration mitigation through inerter-assisted damping.             |
| 5    | Habrah et al., 2022         | Core-outrigger optimization             | Static lateral-load optimization           | Identified efficient outrigger locations and stiffness distributions.       |
| 6    | Wang et al., 2020           | Negative-stiffness damped outrigger     | Dynamic response analysis                  | Showed increased control effectiveness through negative stiffness.          |
| 7    | Wang et al., 2022           | TMD with negative stiffness and damping | Adaptive passive-control study             | Improved robustness over varying dynamic conditions.                        |
| 8    | Sarkar and Fitzgerald, 2024 | TMD fluid inerter                       | Design optimization                        | Developed optimized passive control for wind-sensitive buildings.           |
| 9    | Shirgir et al., 2025        | Reliability-based design                | Uncertainty and soil-structure interaction | Demonstrated the importance of uncertainty-aware tuning.                    |
| 10   | Abrass and Lavan, 2026      | Outrigger tuned inertial mass damper    | Optimal seismic design                     | Integrated outrigger stiffness and inertial damping in a unified framework. |

## 2.1. Research Gap

Numerical comparisons are numerical consequences of simultaneous changes in geometry mass and stiffness modifiers supports or load definitions for the same dynamic response or dynamic property performance pointer for which the optimization is to be done. The present study uses Case 1 as a common reference, isolates the influence of two-level outriggers in Case 2, and through Case 3 with re-proportioning of beam and column dimensions, further comparison is made. The influence of the response of outrigger action and increased primary-frame stiffness can be differentiated.

## 2.2. Research Significance and Contribution

The present study provides a systematic comparison of three structural configurations of the same highly slender 90-storey reinforced-concrete building. Case 1 represents the reference model without outriggers, Case 2 evaluates outriggers at the 30th and 60th floors, and Case 3 examines outriggers with revised beam and column sizes. The study compares displacement, drift, acceleration, modal periods, base shear,

overturning moment, and modal mass participation. This approach clearly distinguishes the effects of outrigger action and increased frame stiffness. The findings highlight the balance between improved serviceability performance and increased force demand, supporting effective structural configuration selection for slender high-rise buildings.

## 3. Research Methodology

The study utilizes a finite-element based comparative approach. The first step of developing the model in ETABS is defining the common building geometry. This includes the assignment of storey-wise concrete grades, diaphragm, mass source, support, gravity load, wind load, response-spectrum parameters, etc. Reference system without outriggers is analysed as case 1. The common analytical framework in Case 2 has been retained, but the outriggers are activated at the 30th floor and 60th floor. In Case 3, a new member size was assumed for the beams of 600 × 600 mm and for the columns 900 × 1200 mm. The values of displacement, drift, acceleration, storey shear, overturning moment, modal time periods, participating mass ratios obtained are extracted and compared.

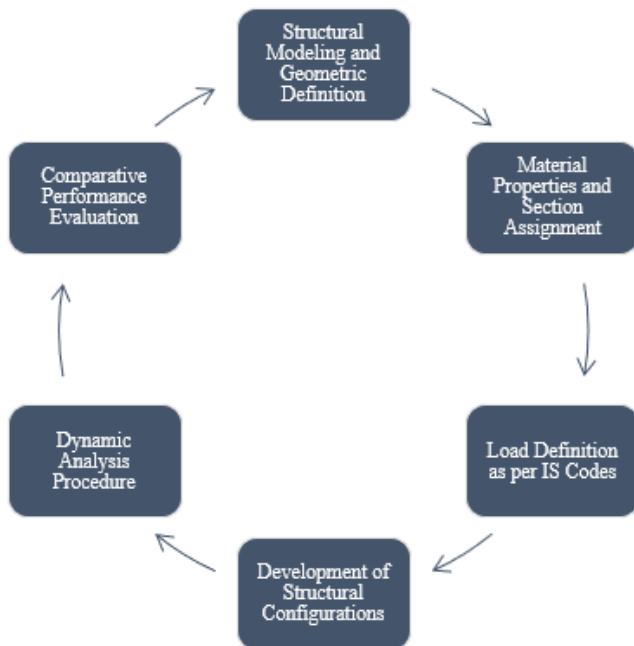


**Figure 1.** Comparative modelling and analysis workflow.

**Table 2.** Analytical configurations.

| Case   | Structural configuration                                               | Purpose                                                                                           |
|--------|------------------------------------------------------------------------|---------------------------------------------------------------------------------------------------|
| Case 1 | Multiple-tube system without outrigger                                 | Reference configuration used to establish the uncontrolled stiffness and response demand.         |
| Case 2 | Multiple-tube system with outriggers at the 30th and 60th floors       | Stiffness-enhanced configuration used to quantify the influence of two-level outrigger action.    |
| Case 3 | Multiple-tube system with 600 × 600 mm beams and 900 × 1200 mm columns | Revised-member configuration used to evaluate the influence of increased primary-frame stiffness. |

### 3.1. Geometry and Structural System

**Table 3.** Building geometry and structural system.

| Parameter               | Adopted value                                        |
|-------------------------|------------------------------------------------------|
| Building system         | Reinforced-concrete multiple-tube high-rise building |
| Structural storeys      | 90                                                   |
| Nominal building height | 310.50 m                                             |
| Uniform storey height   | 3.45 m                                               |
| Plan dimensions         | 24 m × 56 m                                          |
| Plan area               | 1,344 m <sup>2</sup>                                 |

| Parameter              | Adopted value                                       |
|------------------------|-----------------------------------------------------|
| Height-to-width ratio  | 12.9375 using B = 24 m                              |
| Primary lateral system | Multiple-tube framing with reinforced-concrete core |
| Support condition      | Fixed base                                          |
| Outrigger elevations   | 30th floor = 103.50 m; 60th floor = 207 m           |

The ETABS response output has an upper analytical level referred to as Story90 at a height of 310.50 m. As a result, the nominal building height is kept at 310.50 m, while Story91 remains as the closing point of the analytical response. This is done to ensure that the storey labels generated by the software are not altered at all during the process.

### 3.2. Materials and Member Sections

**Table 4.** Storey-wise concrete grade assignment.

| Floor range           | Concrete grade |
|-----------------------|----------------|
| Ground to 30th floor  | M70            |
| 31st to 45th floor    | M60            |
| 46th to 60th floor    | M50            |
| 61st to 75th floor    | M40            |
| 76th floor to terrace | M30            |

**Table 5.** Principal member and material properties.

| Item                 | Adopted value                     |
|----------------------|-----------------------------------|
| Slab thickness       | 150 mm                            |
| Beam size            | 400 × 600 mm (600x600 for Case3)  |
| Column size          | 600 × 600 mm (900x1200 for Case3) |
| Shear wall thickness | 600/800 mm                        |
| Reinforcement        | Fe500                             |
| RC unit weight       | 25 kN/m <sup>3</sup>              |

### 3.3. Loading and Dynamic Analysis

The terms dead loads and live loads are determined based on IS 875 standard whereas wind loads are taken from IS 875 (Part 3) and seismic force is computed in accordance with the response spectrum method set out in IS 1893 (Part 1). The values used in the calculation are the zone factor value  $Z = 0.16$ , importance factor  $I = 1.2$ , response reduction factor  $R = 5$ , and

site type = 1. For the wind load calculations, the static wind velocity was observed as  $V_b = 44$ , and the constant factors were noted as  $K_1 = 1.0$ ,  $K_3 = 1.0$ ,  $K_4 = 1.0$ ,  $K_d = 0.9$ , and  $K_a = 1.0$  along the wind and 0.8 in the direction perpendicular to the wind velocity for the calculations at Site 1.  $K_2$  values are defined in accordance with the height.

$$V_z = V_b k_1 k_2 k_3 k_4 \quad (1)$$

$$p_z = 0.6 V_z^2 \quad (2)$$

$$A_h = \left(\frac{Z}{2}\right) \left(\frac{I}{R}\right) \left(\frac{S_a}{g}\right) \quad (3)$$

$$V_B = A_h W \quad (4)$$

**Table 6.** Loading and verification framework.

| Action/Check   | Basis          |
|----------------|----------------|
| Dead load      | IS 875 Part 1  |
| Live load      | IS 875 Part 2  |
| Wind load      | IS 875 Part 3  |
| Seismic load   | IS 1893 Part 1 |
| Modal coverage | IS 1893        |
| Drift check    | IS 1893        |

## 4. Modelling and Structural Configurations

### 4.1. Case 1: Multiple-Tube System Without Outrigger

Case 1 is the numerical model for reference. The absence of additional outrigger members is used for the multiple-tube framing, core, storey-wise material assignments, diaphragm constraints, member dimensions, load cases and mass definition. The requirements in this configuration outline the reference deformational and modal characteristics to compare the outrigger effect. The maximum displacement, maximum drift, storey shear, overturning moment, the first three modal periods, translational modal participation, acceleration at upper level are principal outputs extracted for Case 1.

### 4.2. Case 2: Multiple-Tube System with Outriggers

In case-2 out-rigger action has been introduced at the 30th floor (103.50m (0.333H) level) and 60th floor (207m (0.667H) level). The internal core region is linked to the outer tube/perimeter columns by the outrigger framing. If the structure sways sideways, the core will twist creating pulling and pushing energy in the pavement columns on the outside, making them more effective and preventing bending. Through the process of altering the outrigger stiffness, it is ensured that load redistribution takes place.

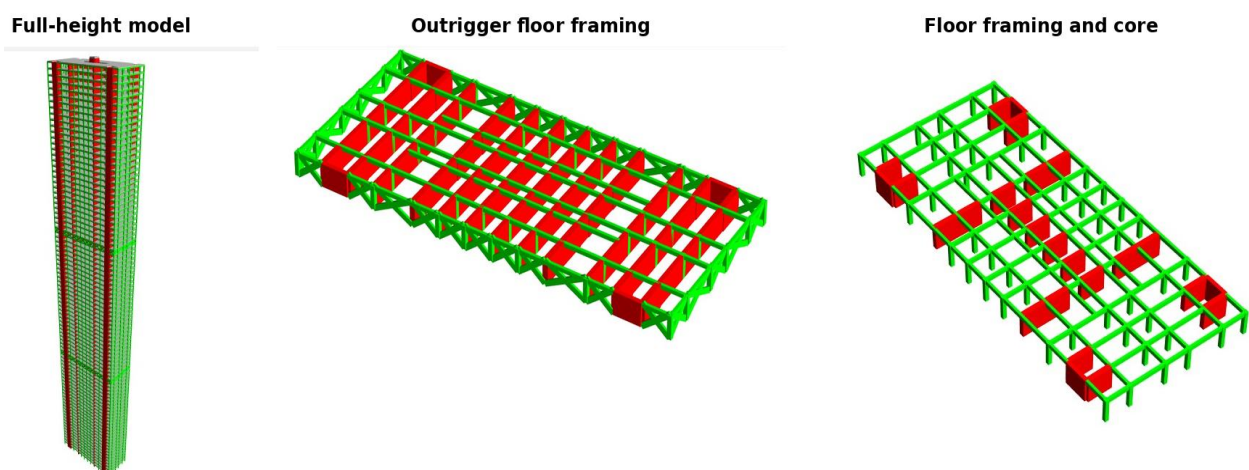


**Figure 2.** ETABS modelling views of the multiple-tube system and outrigger floor framing.

The complete height model exhibits two prominent stiffening bands at the chosen outrigger levels. The views from the floor level show the horizontal truss/frame system that transfers the core rotation to the perimeter structure. It is important for the continuity of the outrigger members and diaphragm compatibility and connection to the external vertical elements

so that incomplete connectivity does not prevent formation of the intended axial-force couple.

### 4.3. Case 3: Revised-Member Configuration

Paper comparison takes place in case number 3 which has

the same analytical framework of 90 storey but the revisions of the principal frame section have been done. The dimensions of the beam are 600 × 600 mm along with that 900 × 1200 mm column. The increase in the size of the individual sections leads to increased structural stiffness of the system and modified displacement, drift, acceleration, modal periods, base/storey shear, overturning demand, etc.

**Table 7.** Case 3 revised modelling parameters.

| Member / parameter | Case 3 value  |
|--------------------|---------------|
| Beam size          | 600 × 600 mm  |
| Column size        | 900 × 1200 mm |

**Table 8.** Primary Case 1-Case 2 response comparison.

| Response parameter                | Case 1                           | Case 2                            | Change           |
|-----------------------------------|----------------------------------|-----------------------------------|------------------|
| Maximum storey displacement       | 1039.431 mm                      | 900.385 mm                        | 13.38% reduction |
| Maximum storey drift              | 0.001057                         | 0.000957                          | 9.46% reduction  |
| Base/storey shear magnitude       | 23,232.83 KN                     | 23,467.89 KN                      | 1.01% increase   |
| Base overturning moment magnitude | 5,430,898 KN m                   | 5,476,750 KN m                    | 0.84% increase   |
| Fundamental period, Mode 1        | 12.093 s                         | 11.315 s                          | 6.43% reduction  |
| Upper-level acceleration          | 0.21 m/s <sup>2</sup> at Story90 | 0.193 m/s <sup>2</sup> at Story90 | 8.10% reduction  |

**Table 9.** Case 3 response comparison.

| Response parameter          | Case 3                            | Change vs Case 1 | Change vs Case 2 |
|-----------------------------|-----------------------------------|------------------|------------------|
| Maximum storey displacement | 615.833 mm                        | 40.75% reduction | 31.60% reduction |
| Maximum storey drift        | 0.000784                          | 25.83% reduction | 18.08% reduction |
| Base/storey shear magnitude | 27,797.57 KN                      | 19.65% increase  | 18.45% increase  |
| Base overturning moment     | 6,487,203.0 KN m                  | 19.45% increase  | 18.45% increase  |
| Fundamental period, Mode 1  | 10.301 s                          | 14.82% reduction | 8.96% reduction  |
| Upper-level acceleration    | 0.151 m/s <sup>2</sup> at Story90 | 28.10% reduction | 21.76% reduction |

## 5.1. Maximum Storey Displacement

Case 1 has a top analytical level maximum displacement, which reached 1039.431 mm. Also, Case 2 had a maximum displacement of 900.385 mm. Hence, the maximum displacement is decreased by 13.38% due to two-level outrigger system. The Case 2 displacement profile is smooth and varies from 0 mm at base to 900.385 mm at Story90. The drop backs

| Member / parameter            | Case 3 value |
|-------------------------------|--------------|
| Acceleration evaluation point | Story 90     |

## 5. Results and Discussion

The same result definitions were used for the three configurations. Deformation measures are interpreted as serviceability indicators, while base shear and overturning moment represent global force demand transmitted through the lateral system. Modal periods and relative frequency ratios are used to check for alterations in global stiffness and the adequacy of the dynamic solution. Percentage change for Case 2 and 3 is relative to Case 1.

up the idea that the outriggers prevent core rotation and involve the perimeter system in global flexural behavior. For tall slender structures, this reduction in displacement is significant because their total lateral sway affects façade movement, partition compatibility, elevator functionality, and occupant perception. The maximum storey displacement of case 3 is Story90 615.833 mm. Its 40.75% less than case 1 and 31.60% less than case 2. The value is also less than IS 1893 limit (621 mm, H/500).

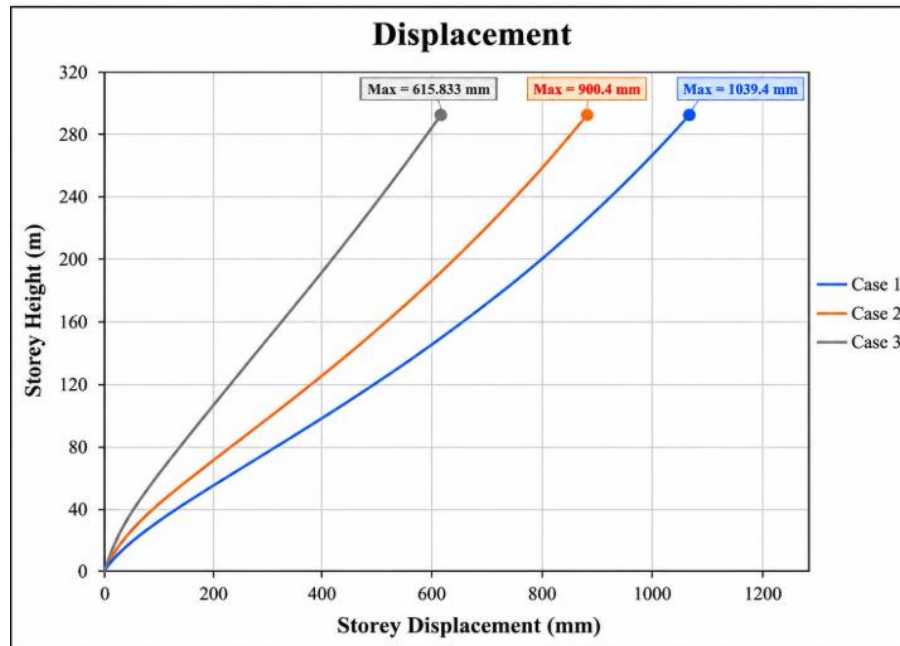


Figure 3. Comparative maximum storey displacement.

## 5.2. Maximum Story Drift

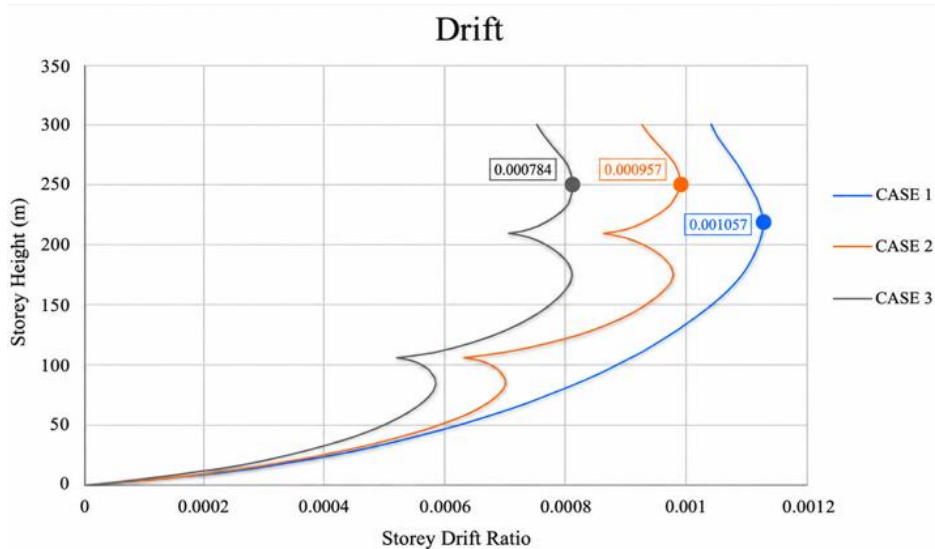


Figure 4. Comparative maximum storey drift ratio.

The reduction in maximum drift ratio, which decreases from 0.001057 in Case 1 to 0.000957 in Case 2, is measured to be 9.46%. In case 2, the maximum tabulated value occurs at Story62, while the plot shows shearing at the outriggers bands which is evident from slope local change. The local disturbances are expected because the outrigger imposes a sudden increase in storey-level rotational restraint and deformities redistribute across adjacent floors. The maximum drift

values are significantly below the limit of IS 1893 i.e. 0.004. Case 1 only about 26.4% and case 2 only about 23.9% gives a reasonable drift and serviceability in both the models and clear improvement after activation of outrigger. In Case 3, Story51 has the largest storey drift ratio of 0.000784. It is 25.83% less than Case 1 and 18.08% less than Case 2. Above value is below IS 1893 limit 0.004.

### 5.3. Storey Shear and Base Shear

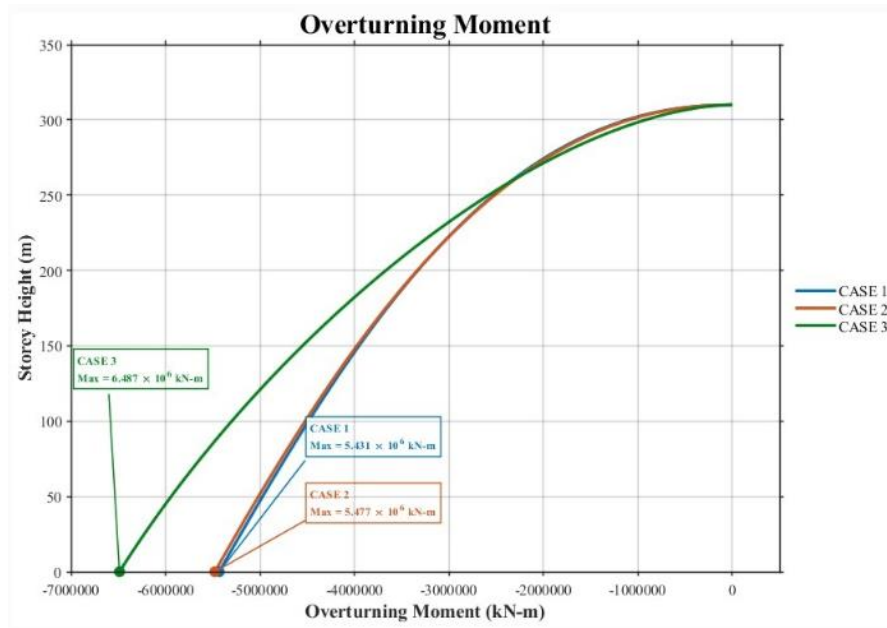


Figure 5. Comparative base/storey shear magnitude.

The storey-shear magnitude rises slightly at the base in Case 1, reaching a base shear of 23,232.83 kN, while in Case 2, the base shear is slightly higher at 23,467.89 kN, which represents an increase of just 1.01%. This is consistent with the use of a relatively stiffer outrigger system, which results in slight increases in the lateral forces while lowering defor-

mations. Therefore, the increase cannot be considered a deterioration in performance, but a trade-off between the reduction in displacement demand and a very high force transfer through the foundations, perimeter columns, core, outrigger members. Finally, in Case 3, the maximum storey shear is approximately 27,797.57 kN at the base, which is up by 19.65% compared to Case 1 and by 18.45% compared to Case 2.

### 5.4. Overturning Moment

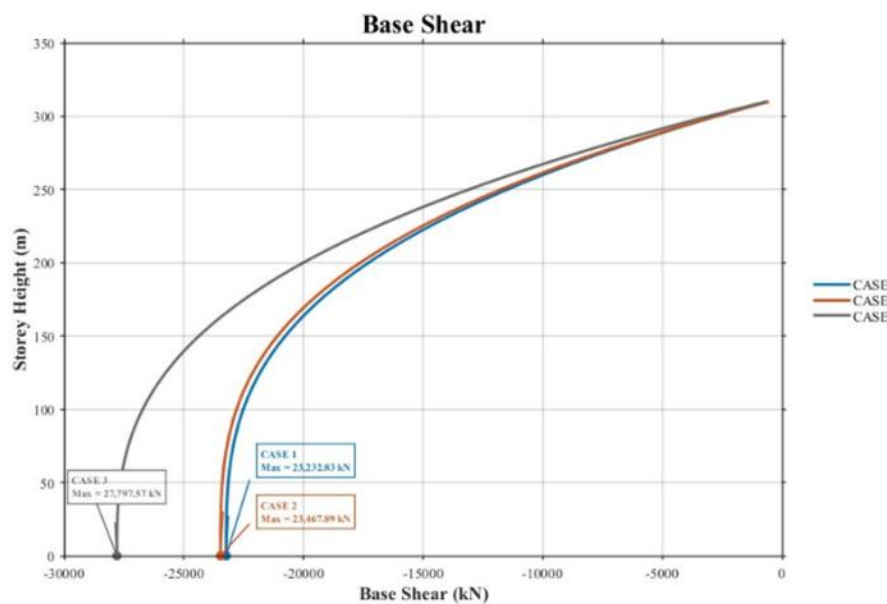


Figure 6. Comparative base overturning moment magnitude.

The overturning moment tends to rise more and more as we approach the base together with the increased lever arm of the lateral forces. The overturning moment values are 5,430,898 KN m in Case 1 and 5,476,750 KN m in Case 2, demonstrating a 0.84% rise. The insignificant rise of the overturning moment value along with considerably decreased displacement of the building shows that the outrigger system takes active

part in redistribution of bending moments. The overturning moment should not be neglected while designing foundations and in terms of axial loads acting on the perimeter columns, which are linked with the outrigger system. The Case 3 provides maximum value of overturning moment of 6,487,203.0 KN m, which is 19.45% higher than Case 1 value and 18.45% higher than Case 2 value.

## 5.5. Acceleration Data

**Table 10.** Acceleration.

| Case   | Configuration        | Point / Location | Acceleration (m/s <sup>2</sup> ) |
|--------|----------------------|------------------|----------------------------------|
| Case 1 | Without Outrigger    | Story 90         | 0.210                            |
| Case 2 | With Outrigger       | Story 90         | 0.193                            |
| Case 3 | Revised member sizes | Story 90         | 0.151                            |

Percentage Reduction: Case 2 vs Case 1 = 8.10%; Case 3 vs Case 1 = 28.10%; Case 3 vs Case 2 = 21.76%

### Acceleration Graph

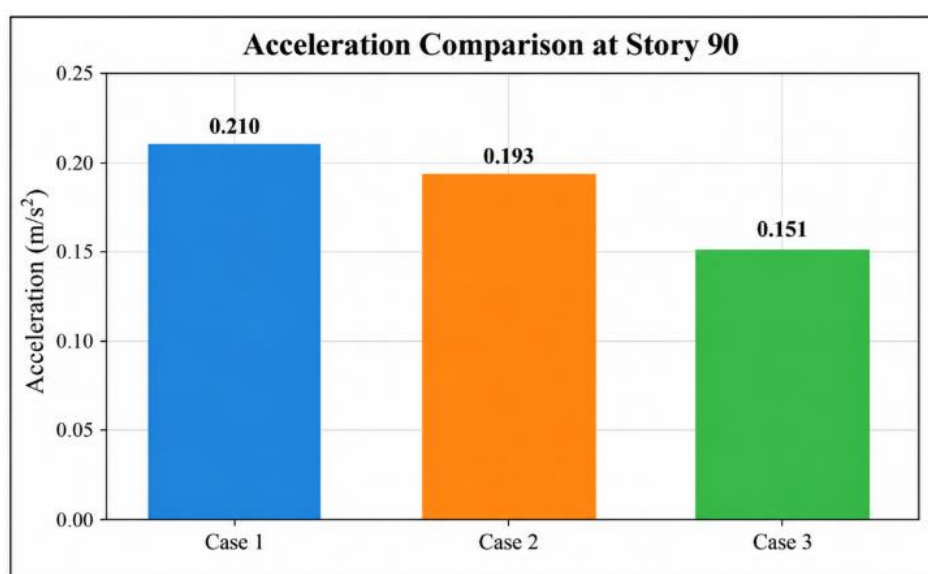


**Figure 7.** Acceleration comparison at Story 90 for the three cases.

At Story 90, acceleration is measured as 0.210 m/s<sup>2</sup> in Case 1, 0.193 m/s<sup>2</sup> in Case 2, and 0.151 m/s<sup>2</sup> in Case 3. The reduction in Case 2, compared to Case 1, is 8.10%, and the reduction in Case 3 is 28.10% compared to Case 1 and 21.76% compared to Case 2. The value is still less than the IS 1893 threshold of 0.25 m/s<sup>2</sup> for commercial buildings.

## 5.6. Modal Periods and Global Stiffness

**Table 11.** Comparison of first three modal periods.

| Mode   | Case 1 Period (s) | Case 2 Period (s) | Case 3 Period (s) |
|--------|-------------------|-------------------|-------------------|
| Mode 1 | 12.093            | 11.315            | 10.301            |
| Mode 2 | 11.722            | 11.118            | 9.486             |
| Mode 3 | 9.693             | 8.679             | 7.681             |

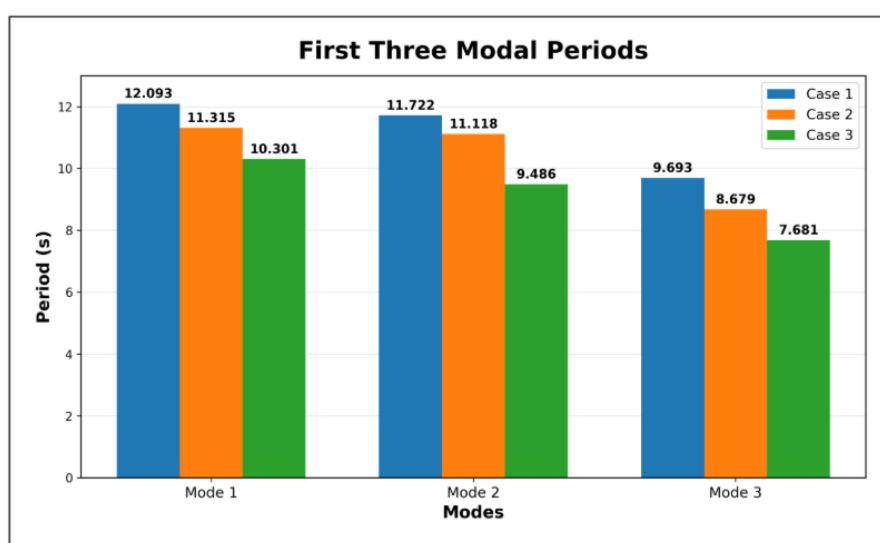


**Figure 8.** First three modal periods for the two structural configurations.

The base period has dropped to 11.315 seconds from 12.093 seconds (6.43%), the second period has dropped to 11.118 seconds from 11.722 seconds (5.15%), and the third period has dropped to 8.679 seconds from 9.693 seconds (10.46%). Continuous reduction of the base periods confirms that the outrigger system has increased the global stiffness in expression for both principal translation and rotation. Considering that the generalized mass of the two case models has not changed significantly, the change in the period in the first

mode corresponds to an effective stiffness coefficient in terms of the period with an increase of approximately 14.2% using the formula of stiffness  $k \propto 1/T^2$ . It should be noted that this calculation serves merely as a gauge for stiffness than an exact calculation of member stiffness. The modal periods for Case 3 are 10.301 seconds, 9.486 seconds, and 7.681 seconds for Modes 1-3. In relation to Case 1, the reductions observed are equal to 14.82%, 19.08%, and 20.76%.

**Table 12.** Wind Base Shear Comparison Cases.

| Sr. No. | Case   | Description          | Wind Base Shear (KN) |
|---------|--------|----------------------|----------------------|
| 1       | Case 1 | Without Outrigger    | 35064                |
| 2       | Case 2 | With Outrigger       | 34552                |
| 3       | Case 3 | Revised Member Sizes | 33865                |

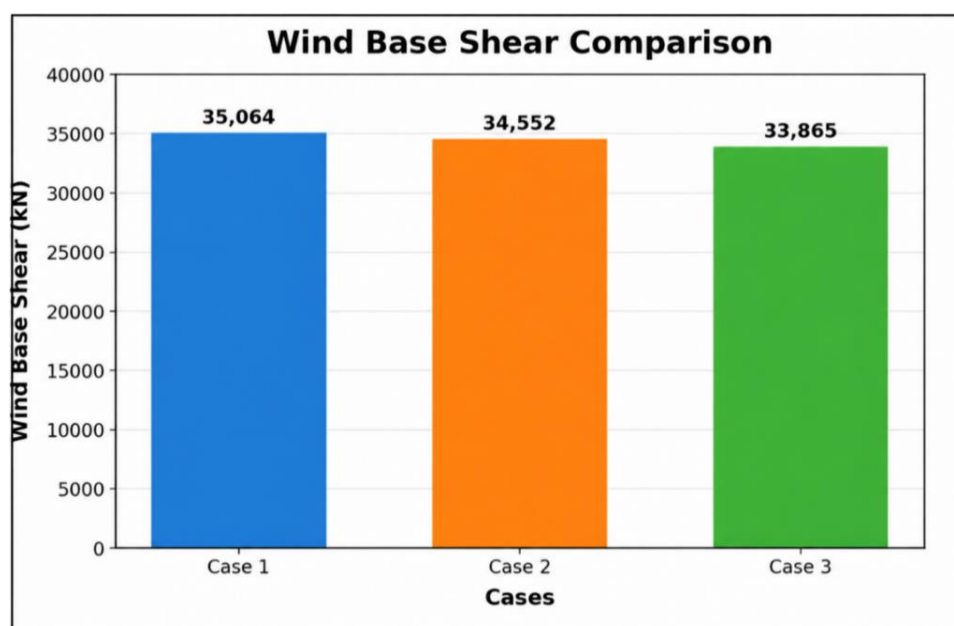


**Figure 9.** Wind Base Shear Comparison Cases.

## 5.7. Modal Mass Participation

**Table 13.** Modal participation comparison.

| Modal quantity        | Case 1             | Case 2             |
|-----------------------|--------------------|--------------------|
| Mode 1 - UX           | 66.30%             | 66.73%             |
| Mode 2 - UY           | 72.07%             | 73.04%             |
| Mode 3 - RZ           | 71.93%             | 73.68%             |
| 90% cumulative UX     | Mode 11:<br>91.44% | Mode 12:<br>91.78% |
| 90% cumulative UY     | Mode 10:<br>91.81% | Mode 10:<br>92.00% |
| Mode 30 cumulative UX | 99.63%             | 99.66%             |
| Mode 30 cumulative UY | 99.82%             | 99.83%             |
| Mode 30 cumulative RZ | 88.87%             | 89.36%             |

The modal participation patterns across the three modes have not changed. Mode 1 shows that participation in Y translation, mode 2 shows that participation in X translation and mode 3 shows that there is a rotation about the Z-axis. In each of the cases considered the condition of 90% cumulative translational mass is satisfied in both main directions.

**Table 14.** Case 3 modal participation.

| Modal quantity        | Case 3          |
|-----------------------|-----------------|
| Mode 1 - UX           | 67.07%          |
| Mode 2 - UY           | 73.38%          |
| Mode 3 - RZ           | 74.40%          |
| 90% cumulative UX     | Mode 12: 92.10% |
| 90% cumulative UY     | Mode 7: 90.09%  |
| Mode 30 cumulative UX | 99.62%          |
| Mode 30 cumulative UY | 99.80%          |
| Mode 30 cumulative RZ | 89.76%          |

## 5.8. Serviceability and Overall Performance Assessment

**Table 15.** Serviceability and structural trade-off assessment.

| Check                | Reference        | Case 1       | Case 2       |
|----------------------|------------------|--------------|--------------|
| Drift limit          | 0.004            | 0.001057     | 0.000957     |
| Maximum displacement | (H/500)          | 1039.431 mm  | 900.385 mm   |
| Base force demand    | Design-dependent | 23,232.83 KN | 23,467.89 KN |

| Check                   | Reference        | Case 1                   | Case 2                   |
|-------------------------|------------------|--------------------------|--------------------------|
| Base overturning demand | Design-dependent | $5.431 \times 10^6$ KN m | $5.477 \times 10^6$ KN m |

Taken together, the results show a progressive stiffness-serviceability improvement from Case 1 to Case 2 and Case 3. Case 2 reduces displacement, drift, acceleration, and modal periods with only small increases in base shear and overturning moment. Case 3 gives the lowest displacement, drift, acceleration, and modal periods, but also the highest force demand because of its larger member stiffness.

**Table 16.** Case 3 serviceability and structural response.

| Check                   | Reference                                     | Case 3                   |
|-------------------------|-----------------------------------------------|--------------------------|
| Drift limit             | 0.004                                         | 0.000784                 |
| Maximum displacement    | (H/500)                                       | 615.833 mm               |
| Base force demand       | Design-dependent                              | 27,797.57 KN             |
| Base overturning demand | Design-dependent                              | $6.487 \times 10^6$ KN m |
| Story 90 acceleration   | 0.25 m/s <sup>2</sup> for commercial building | 0.151 m/s <sup>2</sup>   |

## 6. Conclusions

The three structural configurations are compared and it is seen that a proportional increase in global stiffness improves the serviceability response of the 90-storey highly slender high-rise building significantly.

1. Maximum storey displacement was decreased from 1039.431 mm in Case 1 to 900.385 mm for Case 2 and 615.833 mm for Case 3, meaning there is an improvement in lateral stiffness.

2. The maximum storey drift significantly reduces from 0.001057 in case 1 to 0.000957 in case 2 and 0.000784 in case 3.

3. The acceleration of Story-90 reduced from 0.210 m/s<sup>2</sup> in Case 1 to 0.193 m/s<sup>2</sup> in Case 2 and from 0.193 m/s<sup>2</sup> in Case 3 improving dynamic serviceability.

4. The fundamental time period dropped from 12.093 seconds in Case 1 to 11.315 seconds in Case 2. The further reduction of this global stiffness was demonstrated.

5. When subjected to EQX loading conditions, there was a rise in base shear from 23,232.83 KN in Case 1 to 23,467.89 KN for Case 2 and 27,797.57 KN for Case 3. Under wind loading, base shear dropped from 35,064 in Case 1 to 34,552 KN for Case 2 and 33,865 KN for Case 3.

6. Under wind loading, base shear decreased from 35,064

KN in Case 1 to 34,552 KN in Case 2 and 33,865 KN in Case 3, showing improved wind response.

7. Base overturning moment increased from  $5.431 \times 10^6$  KN m in Case 1 to  $5.477 \times 10^6$  KN m in Case 2 and  $6.487 \times 10^6$  KN m in Case 3.

8. When looking at all three configurations, it is apparent that Case 3 has the lowest displacement, drift, and acceleration time period and that it sees the lowest wind base shear during operations, showing an excellent performance in terms of serviceability.

9. The increased EQX base shear and overturning moment in Case 3 indicate that enhanced structural stiffness attracts greater seismic force demand despite improved deformation control.

10. Overall results demonstrate that the revised-member Case 3 provides the strongest stiffness and serviceability improvement, while requiring adequate member and foundation capacity for increased seismic forces.

## 6.1. Limitations and Future Work

In the present comparison, three configurations that have been completed are compared with the quantities of responses obtained from those configurations. The acceleration has been compared for the case of Story 90 where Case 1 was identified as 0.210 m/s<sup>2</sup>, Case 2 was found to be 0.193 m/s<sup>2</sup> and lastly, Case 3 was rated as 0.151 m/s<sup>2</sup>. Case 3 data is used as a revision member to do a comparison with the paper; it uses the 600 × 600 mm beams and the 900 × 1200 mm columns. The forthcoming work must apply the previously adopted modeling to only the outriggers or the combined outriggers -retro-fitted TMD set-ups and take into account the calculations of forces in members, loads at the foundation and safety of people inside the structure.

## 6.2. Practical Implications and Recommendations

The findings provide practical guidance for improving the lateral-load performance of highly slender high-rise buildings. The two-level outrigger system effectively reduces displacement, storey drift, acceleration, and modal periods, while the revised member dimensions in Case 3 further improve global stiffness and deformation control. However, the higher seismic base shear and overturning moment observed in Case 3 indicate increased force demand on structural members and foundations. Therefore, designers should evaluate serviceability improvement together with member strength, load transfer, and foundation capacity. Appropriate selection of outrigger locations and member dimensions can provide an effective balance between stiffness enhancement, deformation control, structural safety, and overall design efficiency in slender high-rise building systems.

## Abbreviations

|       |                                                         |
|-------|---------------------------------------------------------|
| RC    | Reinforced Concrete                                     |
| ETABS | Extended Three-Dimensional Analysis of Building Systems |
| TMD   | Tuned Mass Damper                                       |
| EQX   | Earthquake Load in Global X-Direction                   |
| IS    | Indian Standard                                         |
| UX    | Translational Degree of Freedom in Global X-Direction   |
| UY    | Translational Degree of Freedom in Global Y-Direction   |
| RZ    | Rotational Degree of Freedom About Global Z-Axis        |

## Author Contributions

**Adeel Ahmad:** Conceptualization, Data curation, Formal Analysis, Investigation, Methodology, Software, Validation, Visualization, Writing – original draft

**Karthik Nagarajan:** Methodology, Resources, Supervision, Validation, Writing – review & editing

**Raju Narwade:** Project administration, Resources, Supervision, Validation, Writing – review & editing

**Gayatri Pathak:** Methodology, Resources, Supervision, Validation, Writing – review & editing

## Conflicts of Interest

The authors declare no conflicts of interest.

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