

Seismic Performance Evaluation of Reinforced Concrete Buildings: A Study on Base Shear, Displacement, and Column Capacity Ratios

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ABSTRACT

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The findings of this research are a detailed assessment of the seismic response of RC buildings with transfer beams with regard to parameters including base shear force, displacement, inter-storey drifts and column capacity ratios. Based on numerical simulations and Linear Time History Analysis (LTHA), the work explores the response of RC buildings to seismic loads, and considers variability in material properties and structural layouts. Different structures of different heights of three types namely sixty meters, hundred meters and one hundred and fifty meters are considered. It also examines the effect of podium layout patterns with respect to lateral force distribution and its consequences on the (D/C) and PMM interaction of columns. Value of maximum displacements and inter-storey drift ratios are also demonstrated to rise with building height especially in structures with transfer beams, while circular columns are seen to have higher PMM capacity ratios than square ones. The results highlighted by the paper should therefore draw the attention of designers/architects and structural engineers to these parameters in the design /retrofitting of high rise building structures for improved structural performance under seismic actions.

Keywords: Seismic, RC building, Linear time, History Analysis.

1. INTRODUCTION:

This paper focuses on the evaluation of seismic risk for RC building to assess the safety and performance of buildings during earthquake occurrences in different regions. RC buildings particularly high-rise buildings are preferred because of their durability, strength and flexibility in the form of construction. Nevertheless, during the seismic actions these buildings experience dynamic and arbitrary loads that affect its stability and performance [1]. Earthquakes apply horizontal loads which cause both lateral and vertical movements throughout the construction starting from the foundations through the columns, beams and joints. Therefore, it is important to evaluate the seismic risk adequately and reduce potential damages and structural collapse as well as to save lives [2].

Uncertainty is an important aspect of assessment of seismic risks, including structural response, material properties and seismic forces. Consequently, material properties including concrete strength, steel yield, and modulus of elasticity in practice may significantly deviate from their predicted values because of variations in quality control, ageing, and environmental conditions. Such changes have an impact on the building's ability to support seismic loads because changes to the material's properties change the stiffness, ductility, and ability to dissipate energy of the structural members [3].

Likewise, the loads are considered as stochastic because the occurrence of earthquakes is unpredictable in nature. A number of factors including the characteristics of the ground motion (intensity, frequency content, duration etc), distance from the source and nature of local soils also contribute to this variability. The force, deformation, and frequency characteristics must be taken into consideration when designing structures that should be able to respond to seismic forces in the most efficient manner possible [4].

However, uncertainties from the material and loads are accompanied by structural responses that contribute to variability in seismic performance. The behaviors of RC buildings which include transfer beams, podiums, and buildings with irregular shape may experience higher degree of seismic impacts, especially in buildings with high stiffness or mass irregularities [5]. Transfer beams for instance transfer loads within the structure, and can provoke in the regions the concentration of stress and deformation even in the lower floors. This calls for a more refined form of modeling and analysis because of points of failure by way of load flow imbalance and consequent irregular dynamic responses [6]. These uncertainties have compounded the argument for a probabilistic framework for seismic risk assessment, rather than deterministic risk assessment. Incorporation of uncertainties in the material parameters, seismic loads and structural response can be done using probabilistic analysis, which offers more insight into the performance of the building under all the defined probable conditions [7].

In the years past, several approaches have been proposed to evaluate the seismic vulnerability and to evaluate the behaviour of RC buildings subjected to earthquake effects. Conventional seismic design and analysis techniques employ deterministic practices which imply that definite values of material properties, load intensities and structural responses are employed to determine the behaviour of a building[1]. IS 1893 (Indian Standard Criteria for Earthquake Resistant Design of Structures), and IS 456 (Indian Standard Code of Practice for Plain and Reinforced Concrete) codes specify how seismic loads can be predicted and how structural members shall be proportioned to support earthquake loads. However, these deterministic methods used above give only an average idea of the structural performance without considering the variability of response under different circumstances [8].

Due to several aforementioned shortcomings associated with deterministically based risk assessment, probabilistic seismic risk assessment (PSRA) has emerged in the recent past. PSRA includes the probability of different earthquakes and the array of structural behaviors with those earthquake, which makes risk quantification more accurate. For instance, Monte Carlo simulations allow the creation of many different scenarios with different input parameters, for instance ground motion records, material characteristics, and loadings [9]. Each of the scenarios reflects a possible earthquake and building response, which can be statistically compared to consider the chances of various failure mechanisms[10].

However, there are still shortages in the evaluation of uncertainties in seismic risk prediction for RC buildings with a complicated design. Although PSRA, and other probability based approaches offer a global view of risk, they may not easily distinguish between errors in the estimation of uncertainties associated with members such as transfer beams and columns, where complex load conditions such as axial force and moment (P-M) interaction occurs [11]. It has been found that the columns which are stressed by high axial loads and low moments are likely to fail in a brittle manner and hence the detailed study of axial-bending (P-M) interaction. Moreover, there is scarcity of literature about how several sources of irregularities including podium levels, transfer beams and irregular building configurations which can in combination increase the impacts of the seismic forces [12]. In configurations like these, columns are reported to need additional axial strength and it is essential to properly consider their responsiveness to bending forces to achieve desired ductility though specific recommendations are sometimes missing in the present design codes [13]. Moreover, as transfer beams and podium levels have become important components of modern tall buildings, previous research has mainly considered buildings with regular geometric shapes. Transfer beams are sources of localisation of load paths that directly affect base shear, inter-storey drift and stability especially in tall structures [14]. Podium levels, based for parking or the commercial use, can change lateral stiffness distribution further, which makes it difficult for the building to respond to seismic forces. However, there is a lack of systematic investigation on how these features impact the Shear D/C ratio or PMM (axial-biaxial moment) capacity in columns [15].

In addition, current probabilistic approaches pay more attention to the global response quantities, such as base shear and roof displacement, and lack a clear way of addressing local response quantities such as the D/C ratio and the interaction between PMM. These local parameters are important for evaluating safety of transfer columns and other members in buildings having irregular geometries. Current global risk analysis seems to require a more holistic assessment of structural properties applying global and local parameters to better pinpoint potential failure sources and components that need further retrofitting or reinforcement [16].

In conclusion, though, the assessment of seismic risk has moved forward with the emergence of probabilistic methods, as well as improved computational software and hardware, there is still no adequate handling of the uncertainties that relate to complex RC building configurations that incorporate transfer beams, podiums and other irregularities. This study seeks to fill these gaps by using an improved probabilistic approach that considers both global and local performance measures enhancing the understanding of seismic hazards and design factors of RC buildings with intricate lateral force resisting systems.

The main aim of this study will be to evaluate the response of RC buildings with transfer beam in terms of base shear, roof displacements, inter-storey drift and column capacity ratios. The study's objectives are to assess Type B, C, and D building arrangements and heights of 60m, 100m, and 150m to understand structural response during earthquakes. Therefore, through comparing both square and circular columns, the study shall identify the changes in the PMM (axial-biaxial moment) capacity ratios and shear demand-to-capacity ratios especially in the lower floor levels. Also, recommendations are made on how to improve the seismic performance of RC buildings particularly high rise building through conscious improvement in their design and retrofiting.

2. METHODOLOGY

The approach used in this study is a comprehensive one that analyses the seismic vulnerability of RC buildings based on the stochastic nature of material properties, loads and building construction. The process, as outlined in Figure 1, consists of four key stages: Building Selection, Building Design, Seismic Assessment and Findings .

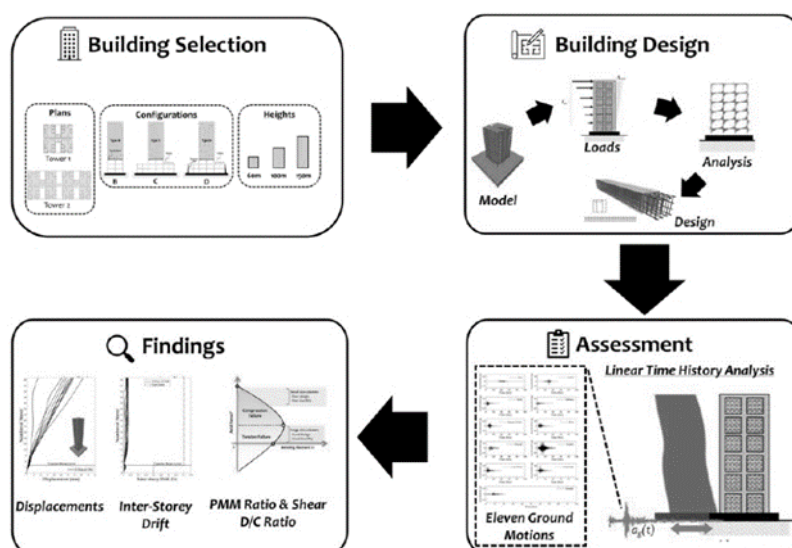


Fig.1.Methodology of the current work.

3.1 Building Selection

The first step entails identifying RC buildings which are diverse in nature for analysis. The selected buildings vary based on several parameters:

- Plan aspect ratios: Two different plan layouts are considered to study effect of geometry on seismic response.
- Structural configurations: This collection features configurations, such as buildings with transfer beams, podiums and retaining walls. These configurations (Type B, C, and D) are characteristic of high-rise residential buildings in the densely populated areas.
- Heights: Three different building heights are therefore considered, 60m, 100m and 150m to study the effect of height on the number of storeys on seismic behavior particularly affected by stiffness and mass irregularities.

This stage ensures that structures selected for modelling exhibit a diverse range of structural behaviour as observed in metropolises such as Hyderabad and Bengaluru.

3.2 Building Design

The second phase revolves round designing the selected configurations after identifying the buildings. Details structural models for each building are developed using advanced numerical simulation tools in ETABS. This involves:

- Applying loads: The buildings are loaded with various load conditions such as the dead load, the live load, the wind load, the earthquake load and other special loads as permitted by Indian code (IS 1893, IS 16700).
- Analysis and design: The response of each building is then calculated to determine such performance parameters as displacement, drift, and base shear. The design process is an iterative one as to meet strength, stiffness, and deformation requirements based on codes for earthquake resistant design.

3.3 Seismic Assessment

The third phase involves the assessment of global and local seismic performance using linear time history analysis. In this method, the building's dynamic response is determined through eleven records of ground motion representing different levels of earthquake. This enables recording of true seismic response of the buildings in different seismic conditions. Of special interest are the distribution of the seismic forces and the resulting displacement, drift, and internal forces in primary and secondary components such as beams, columns, and transfer beams.

3.4 Findings

The last process is an analysis of the information obtained from the seismic assessment. Key performance indicators are examined, including:

- Displacements: The dynamic translations of the structure in the horizontal direction when undergoing the effect of an earthquake.
- Inter-storey drift: The difference in distance between two successive floors, or storey, which can always be a factor contributing to the occurrence of structural damage.
- PMM (Axial Load-Moment Interaction) Ratio: This helps determine the ability of members and structures in bearing both axial and the bending loads.
- Shear demand-to-capacity (D/C) ratio: A measure of the shear forces inherent in structural members and a measure of the potential for failure in the structures.

The findings of this phase are critical in establishing the weaknesses of the structures, and possible alteration to offer better seismic performance.

3. 4. RESULTS AND DISCUSSIONS

These adjusted ground motions are then imposed laterally at the foundation level of the building. To assess performance, three global parameters are considered: base shear, displacements and Inter storey drift ratio. Additionally, two local parameters specific to the transfer columns are evaluated: the PMM capacity ratio and the shear demand-to-capacity ratio, as well as. These indicators are used to evaluate the performance of transfer beam buildings of the total structures.

4.1. Base Shear

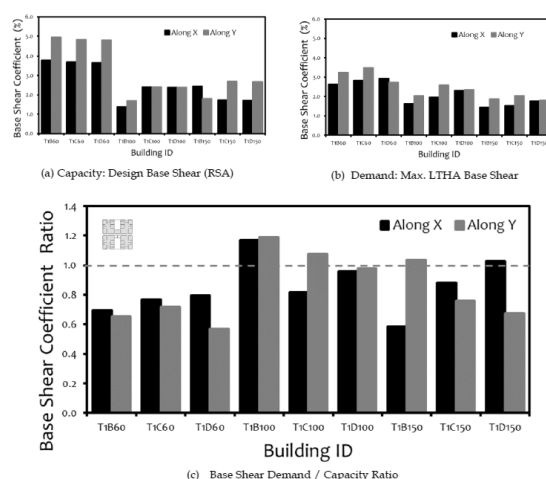


Fig.2. Design (RSA) and LTHA base shear for Tower 1 buildings.

Figure 2 also demonstrates the variation of the design base shear from the RSA and the maximum base shear from the LTHA for different configurations of Tower 1 building along the X and Y directions. The figure consists of three subplots: Figure 2 (a) Capacity: Design Base Shear (RSA); base shear coefficients involve Tower 1 building configuration (T1B60, T1C60, etc.) referring to buildings' design capacity based on RSA. For base shear forces comparison, base shear in both X and Y directions is computed and plotted, some of the buildings, for instance, T1D60 have more base shear in Y direction than that in X direction.

Figure 2 (b) illustrates demand: Max. LTHA Base Shear: The following chart also shows the maximum base shear coefficients from LTHA for the same building configurations. It shows the actual load that comes from the seismic loads and there is a marked difference between the x and y direction. In some buildings (e.g., T1B60), the demand is greater in the Y-direction, while in others (e.g., T1B100), the demand is greater in the X direction than in the Y direction. Figure 3 (c) presents the base Shear Demand/Capacity Ratio: This plot represents the relative demand to capacity ratio of the base shear obtained from LTHA and the design capacity obtained from RSA for each of the building configuration in both direction. The red dashed line shows a one to one relationship since the demand equals the design capacity. Any value above this line, such as T1B100 along the X direction, means that the demand beyond the design capacity while any value below this line, such as T1C100, means that the design capacity exceeds the demand.

In general, the figure presents the differences in the base shear with regard to configurations and directions and that some buildings have adequate design capacity to meet seismic demand, while others may need to reconsider their seismic design due to the demand that may have exceeded their capacity especially in the X direction for some configurations of the building.

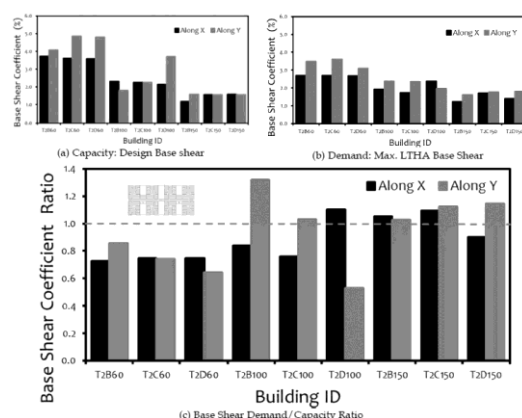


Fig.3. Design (RSA) and LTHA base shear for Tower 2 buildings.

As with the analysis for Tower 1 buildings, Figure 3 shows the design base shear compared with the maximum base shear from LTHA for the various configurations of Tower 2 buildings in the X and Y directions. The figure is divided into three subplots: Figure 3 (a) Capacity: Design Base Shear (RSA): This is the design base shear obtained using RSA for the various Tower 2 configuration such as T2B60, T2C60 . It shows the capability of each building arrangement in the X and Y axis. For instance, configuration such as T2B60, the Y direction is observed to possess a greater base shear capacity than the X direction, or configuration such as T2B100, the first direction has the greater base shear capacity compared with the second direction. Figure 3 (b) Demand: Max. LTHA Base Shear: This plot shows the maximum base shear from LTHA for the same Tower 2 building configurations. It points to the real seismic force that is placed on the buildings. For example, in the case of T2B60, the demand in the Y-direction is higher, and in the case of T2D100 the demand in the X-direction is higher. Figure 3 (c) Base Shear Demand/Capacity Ratio: The following graph shows the variation of the demand from LTHA, in terms of base shear, to the design base shear capacity from RSA. A line parallel to the horizontal axes drawn through the desired demand capacity ratio of 1.0 is shown by the red dashed line. Values above this line defined, for example, for T2B100 in the direction 'Y' mean that seismic load exceeds design load. On the other hand, ratios lower than 1.0 indicate appropriate design capacity to accommodate the seismic demand as evidenced by T2C100 along X and Y.

Altogether, Figure 3 indicates that some of the Tower 2 configurations require the increased demands in certain directions, notably in the Y direction for some of the building models such as T2B100; whereas, others, for example, T2C100, demonstrate adequate design capacity to accommodate the seismic demands. This graph shows the areas that may require redesign to meet or improve the level of seismic performance.

4.2 Displacements

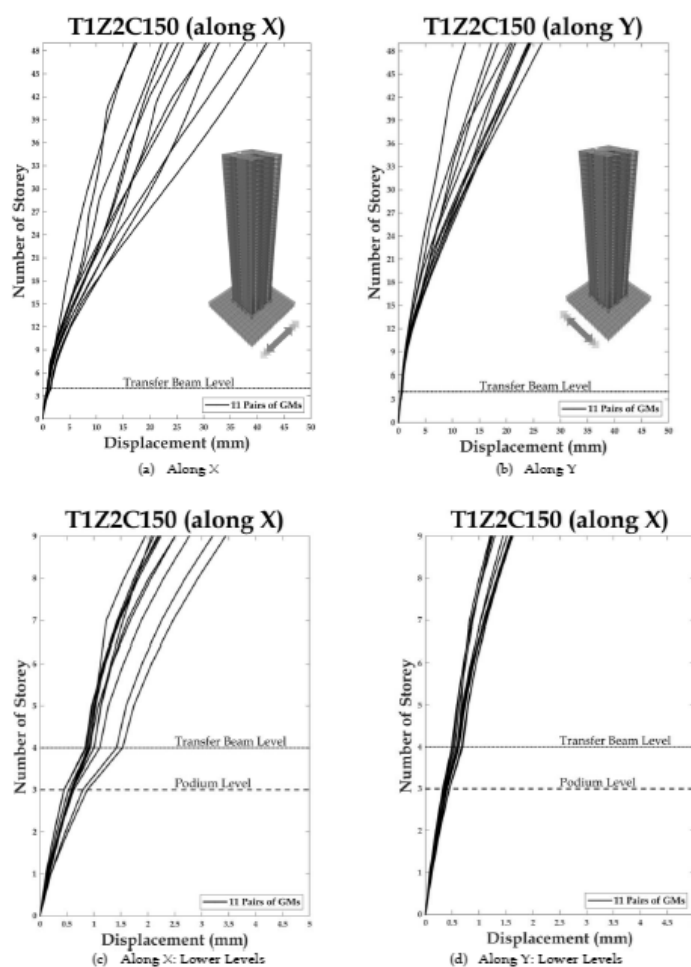


Fig.4. LTHA displacement profile for T1Z2C150.

Figure 4 shows displacement history of building T1Z2C150 subjected to modified ground motions in X and Y direction. These displacement profiles give an understanding of how the structure distorts when subjected to earthquake loading. Figure 4 (a) Displacement along X-direction: The first plot also represents the displacement response of the building on the X direction. The maximum displacement underneath the roof is observed to be approximately 42mm and the minimum displacement observed at the roof level is approximately 17mm. This shows a moderate rocking of the building in the X-direction from the podium to roof level. Figure 4 (b) provides displacement along Y-direction: The second graph is the displacement profile in the Y-direction. As seen here, the roof displacements are smaller than those observed in the X direction and range between 12mm and 27mm at maximum. Slightly lower displacements in the Y direction are evident due to increased wall framing along this axis, thereby increasing the stiffness and the overall lateral movement of the structure is minimized. Figure 4 (c) illustrates Lower-level Displacement along X-direction: This graph focuses on the lower portion of the building especially in the region of the transfer beam/podium. There are no significant changes of these displacements at these levels and it reveals stable displacement profile which indicates that the transfer beam and podium enhance the stiffness and stability of the lower part of the structure. Figure 4 (d) Lower-level Displacement along Y-direction: Also similar to (c), this plot concerns the lower levels but in the Y direction. The displacement profile is maintained flat and continuous all throughout the transfer beam and podium levels, which means that the lower part of the building is stable and seismic movements are well contained.

Therefore, the displacements in the X direction is greater than the displacements in the Y direction, because the stiffness in the Y direction is enhanced by the structural walls. There is no much displacement amplification around the podium and transfer beam levels and as a result of good design in the structure, the seismic displacements are well controlled at these critical points.

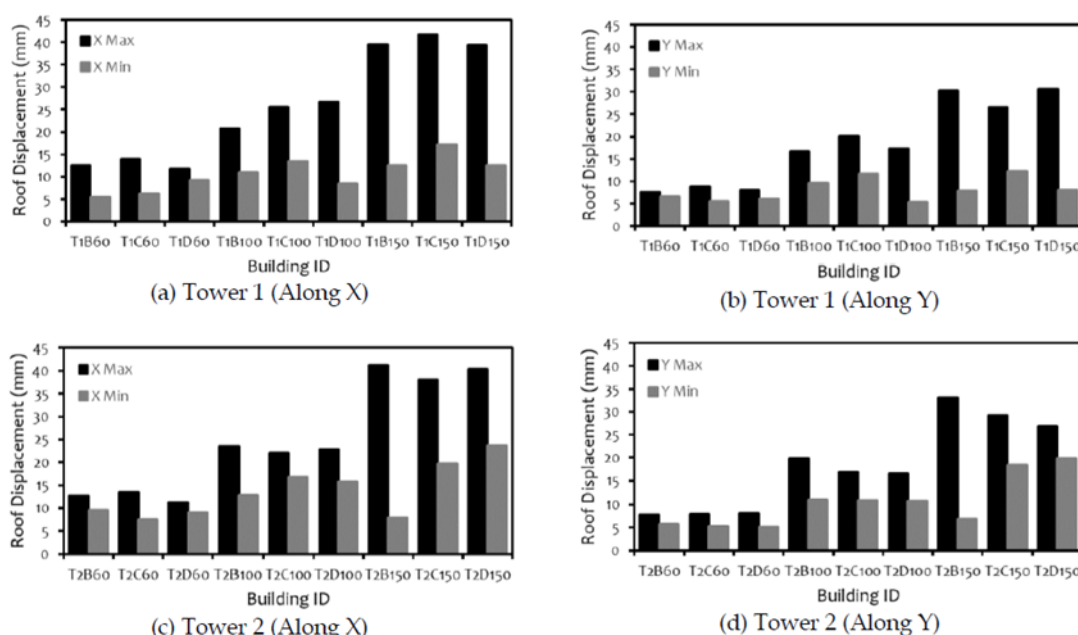


Fig.5. Maximum and Minimum LTHA Roof Displacements.

Figure 5: Maximum and minimum roof displacements due to LTHA for Tower 1 and Tower 2 building along X and Y direction. Figure 5 (a) Tower 1 Along X: The displacement at the roof level is demonstrated in the graph in terms of the X-direction for Tower 1 buildings. The maximum roof displacement also rises with the building height; T1D150 has the largest displacement of 42mm at most. As expected, the minimum displacements have similar profile with the maximum displacements, but are always less than it.

Figure 5 (b) Tower 1 Along Y: In this graph, the Y-direction roof displacement of Tower 1 buildings has been shown. Again, the behavior observed is analogous to the X direction; that is taller buildings have maximum and minimum displacement values such as T1D150; however, the peak and valley displacement values are smaller in Y direction

compared to X direction, which indicates that the structures are stiffer in the Y direction. Figure 5 (c) Tower 2 Along X: For Tower 2 buildings, the magnitude and direction of roof displacement in the X-direction are also a function of building height. T2D150 has the greatest max displacement as observed earlier in Tower 1 case and the displacement magnitude is somewhat lesser for the Tower 2 buildings compared with Tower 1 counterparts. Figure 5 (d) Tower 2 Along Y: Similarly, the Y-direction displacement for Tower 2 buildings is observed and T2D150 possesses the maximum displacement. Similar to Tower 1, the displacement results in the Y direction are relatively smaller than in the X direction, which also shows an increase in the stiffness along this direction for both Tower 1 and Tower 2 buildings.

In summarizing the figure, the figure demonstrates that with height of the structure, the roof displacement rises and more displacement is observed in x-direction than y-direction due to difference in stiffness of the structure in the two directions. Buildings of higher structural heights particularly for those with transfer beams and podiums (T1D150/T2D150), exhibit relatively higher peak roof displacements under seismic loads.

4.3 IDR (Inter-storey Drift Ratio)

Inter-storey drift (ID) is the relative displacement between two consecutive floors of structure during earthquakes or any dynamic loading. Inter storey drift ratio is calculated by dividing the maximum inter storey drift by the storey height. For example, suppose the maximum ID between two adjacent floors equals 100 mm and the storey height equals 3 meters; in this case, the IDR would be equal to $100/3000 = 0.0333$, or 3.33%. When the imports of the IDR are excessive, they may weaken the structures of buildings and even lead to damages of devastating impacts or even total collapse. It also impacts NSE because increased power of IDR is capable of compromising the partitions walls and cladding. Furthermore, there is an IDC tracking in order to protect building occupants and maintain their comfort: large drift ratios might lead to objects falling down or cause motion-sickness like discomfort. Last but not the least, IDR is used as a vital index for determining the performance of a building in case of an earthquake and to determine which of the floor elements need additional work in the form of retrofitting.

Figure 6 shows the Inter-Storey Drift Ratio (IDR) with respect to storeys 1 to 10 for different building types under Linear Time History Analysis (LTHA), with results split for Tower 1 and Tower 2 heights (60s, 100s, 150s) in both X and Y directions. The graphs are categorized into six sections based on building heights:

- (i) T1_60s: In this part, the IDR profile of Tower 1 building, 60 meters high is presented. All the displacements along the X direction show moderate drifts around the lower storeys with concentration on the important points around the podium and transfer section. The IDR profiles indicate that shorter length buildings exhibit typical sway characteristics.
- (ii) T2_60s: As with the Tower 1, 60 meters tall Tower 2 buildings have moderate drift at the lower floors, while the Y-direction has slightly lower IDR values because of additional structural walls in the buildings that enhance stiffness and reduce drift.



Fig.6. LTHA IDR profile for lower storeys for all the buildings.

(iii) T1_100s: The results for the IDR profiles for the 100m tall Tower 1 buildings show that the inter-storey drift has significantly increased, especially around the locations of transfer beams in both directions. This may be an

indication that there is tendency of drift concentration in tall structures, which may call for some considerations in the design.

(iv) T2_100s: The similar pattern was observed for Tower 2 buildings with 100 meters height; however, the drift levels are generally slightly lower than those in Tower 1. This will be attributed to the difference in the design configuration of the two tower types especially in Mass and stiffness.

(v) T1_150s: The IDR profiles for Tower 1 buildings at 150 meters indicate that the near the lower storeys, there is considerable drift, and drift amplification occurs near the transfer beam levels. This implies that in tall buildings drift concentration is more conspicuous and is likely to affect the performance of structures during earthquakes.

(vi) T2_150s: Similar behavior is found for Tower 2 building having 150 meters height; the IDR value is slightly higher near lower storeys, especially in the X direction. However, the Y-direction drifts remain smaller still, due to the additional contribution of the structural walls to stability.

In general, Figure 6 shows the level of inter-storey drift rises with the building height and the most significant tendency of drift grow is found near the transfer beam. In general, Tower 1 and Tower 2 have similar trends, although Tower 2 configurations are generally lower in the drifts, especially in the Y direction, because of the difference in building structure and presence of extra stiffening. As the results suggest the paper underlines the need to manage drift in tall structures to enhance the structures' capability to withstand the impacts of earthquakes.

4.4. Column PMM Capacity Ratio

Columns are structural elements which are exposed to the theoretic axial force (P) and combined biaxial bending moments (M_2 , M_3) due to gravity and lateral loads. These members are designed using a P-M interaction envelope curve as specified by SP:16 (1980). With improvements in computational technology, it is therefore possible to develop a PMM interaction, a 3D interaction capacity surface. Members subject to high axial loads are prone to brittle failure when combined with modest moments. Previous studies have shown that in order to provide ductile behavior columns should possess a large cross-sectional area and that the axial stress should be kept well below the value of the balanced point on the P-M interaction diagram, even when dead, live and earthquake loads are considered. Hence, during performance evaluation it is desirable that the combined demand of axial compressive force and bending moment to be located in the lower third of the compression P-M interaction diagrams for improved ductility and performance.

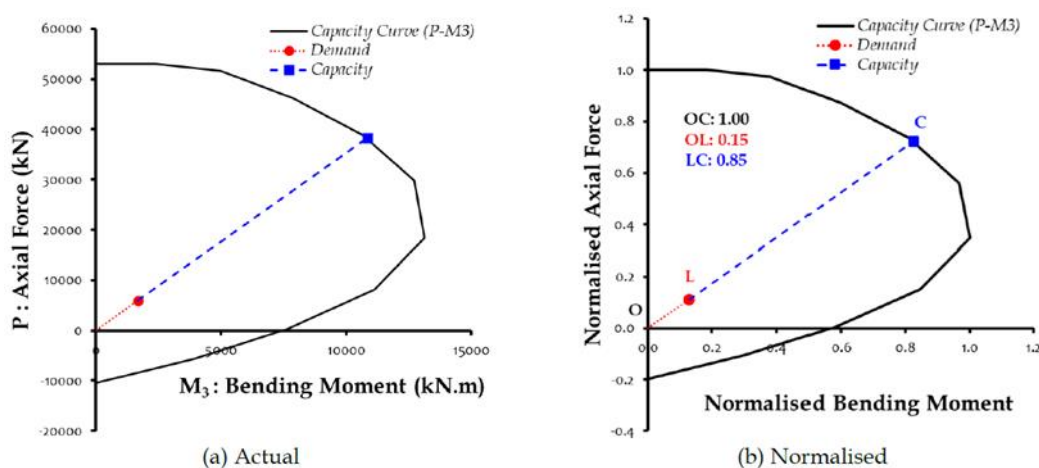


Fig.7. Example of computation of PM capacity ratio.

Figure 7 exemplifies the procedure for calculating the P-M interaction (PM) capacity ratio of structural columns: Actual and normalised P and M_3 curves. Figure 7 (a) Actual: The actual axial force 'P' and ' M_3 ' and their relation graph is shown in the left side of the above figure. The solid black line in the figure indicates the predicted capacity envelope of the column under axial and bending force. The red dot is the demand, it is the axial force and moment applied on the column while the blue square is the capacity of the column to withstand these forces. The blue

broken line links the demand to the capacity point to show the performance level of the column. As it can be observed, the closer the demand is to the capacity curve, it follows that its ability to withstand the combined loads will be very poor. Figure 7 (b) Normalized: The second one shows the same P-M interaction but in normalized scale which is depicted on the right graph. The normalising is done by dividing axial force and bending moment by a factor so that the results are presented in a dimensionless form. The capacity curve is once more represented by the black line, the red spot exemplifying the normalized demand and the blue square as the normalized capacity. The individual points noted as “OC”, “OL” and “LC” depict various stages of loading profile. The demand is represented by L on the same normalized axis as the capacity represented by C to provide a clear comparison of the structural performance.

In sum, Figure 7 essentially shows how to find the PM capacity ratio in the face of demand (applied loads) against the capacity (resistance) of the column. This assists in determining whether the column can handle both axial and bending loads, with higher demand capacitated likelihood of structural failure under those loads.

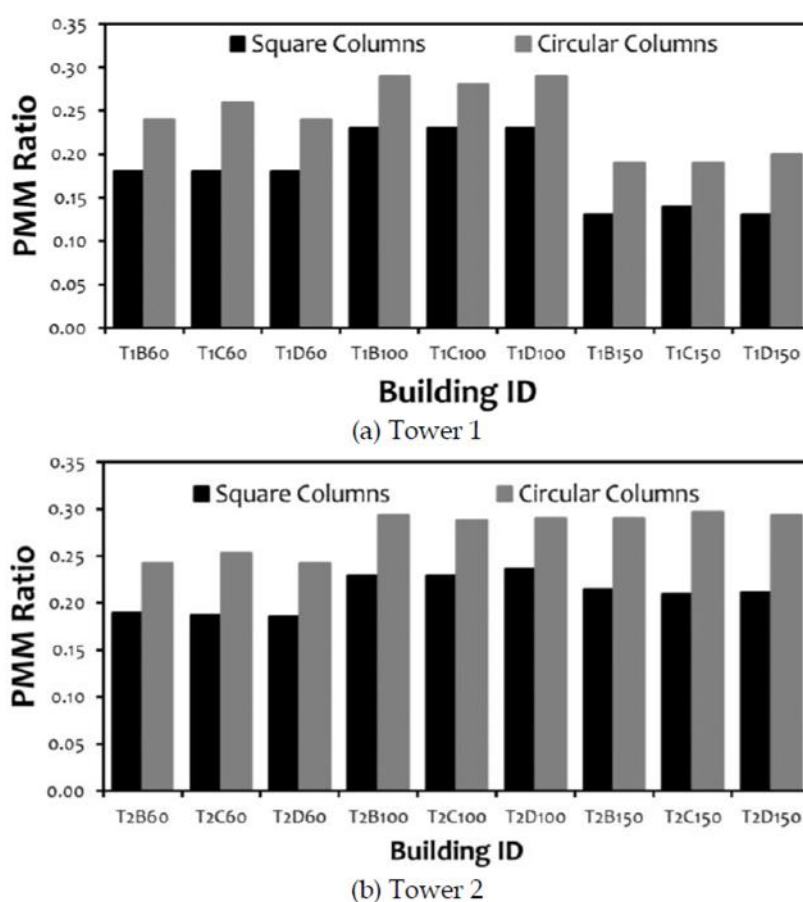


Fig.8. Maximum LTHA PMM capacity ratio (storey 1-4) for all buildings.

To illustrate the maximum PMM capacity ratio for both Square and Circular Columned Building – Tower 1, respective to Tower 2 in upper to lower storey (from Storey 5 to Storey 1), Figure 8 below: The PMM ratio characterizes interaction of axial force and biaxial bending moments (P, M₂, M₃) and gives an idea of how close the columns are to their strength limits. Figure 8 (a) Tower 1: The first graph above illustrates the PMM ratio of square and circular columns of the Tower 1 buildings. This shows that for most of the building configurations, the PMM ratios of circular columns are higher than those of square columns. For example, in T1D100, circular columns had the PMM ratio close to 0.3 which implied that it had higher demand for capacity ratio than the square columns which had the ratio close to 0.2. This implies that generally, circular columns undergo greater interaction between the axial force and bending forces, by putting them closer to their capacities than square columns. Figure 8 (b) Tower 2: The second graph shows the same information relative to Tower 2 structures. Similarly to earlier

observations, circular columns have higher PMM ratios than square ones in almost all the cases. For example, in T2D100, the PMM ratio of circular column is around 0.3 and square columns ratio is slightly more than 0.2. As with Tower 1, the circular columns are much more critically loaded with respect to the axial and bending interaction.

In general, Figure 8 demonstrates that circular columns are subjected to rather higher PMM ratios, and therefore, they experience rather higher combined axial and moment forces get closer to their capacity limits than square columns. This variation is observed in both the Tower 1 and Tower 2 buildings and points to the importance of shape in the manner the columns perform under seismic and load conditions in the lower storeys.

4.5 Column Shear Demand/Capacity Ratio

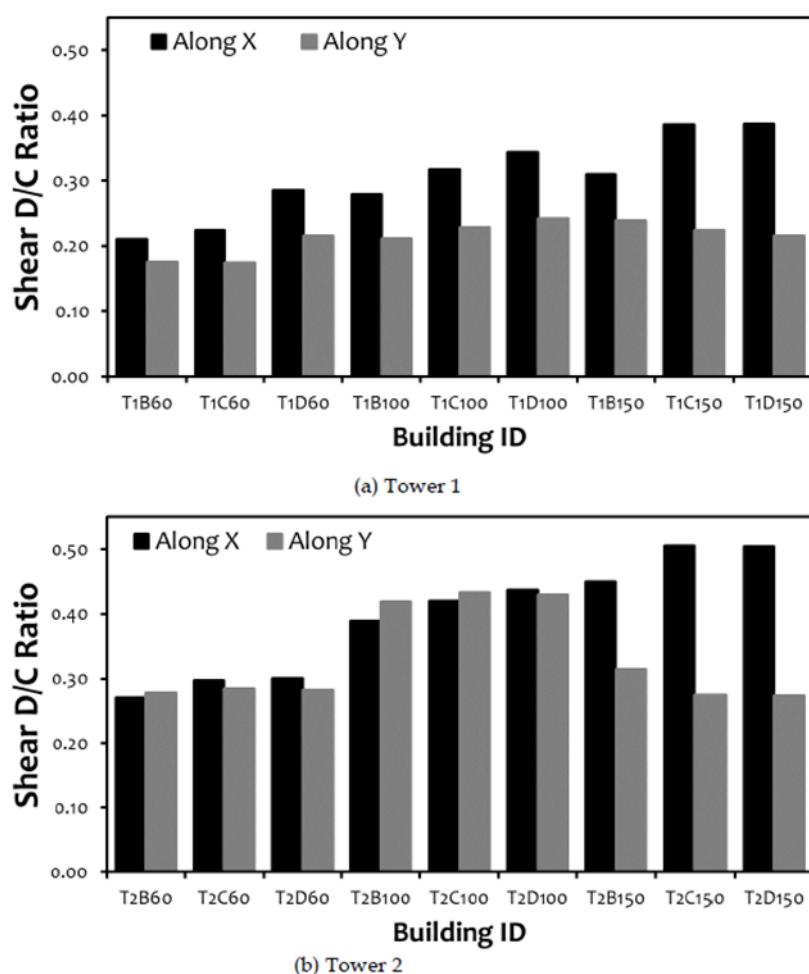


Fig.9. Maximum LTHA shear demand/capacity ratio (storey 1-4) for all buildings.

Filled diamonds, triangles, and squares in Figure 9 denote the D/C ratio for transfer columns in storey 1-4 of Tower 1 and Tower 2 buildings, respectively. By establishing the shear demand through various load combinations and assessing the shear capacity according to IS 456 with the help of reinforcement details, the D/C ratio is determined. The results for the X axis and the Y axis are also shown below. Figure 9 (a) Tower 1: The graph above was created to illustrate that the shear D/C ratio in the X and Y directions as affected by building configuration. The maximum shear D/C ratio in Tower 1 is nearly 0.387 in the X direction for T1D100. In most of the cases, the D/C ratios of podium configuration buildings (Types C and D) are found to be slightly higher than that of Type B buildings. This implies that podium configuration affects the lateral force distribution causing higher shear demands of lower storeys. Figure 9 (b) Tower 2: As with Tower 2 buildings, all the plots show similar trends, with the maximum shear D/C ratio of 0.505 (T2C150 along the X direction). As with Type C and D buildings that have podiums, the D/C

ratio is, in general, higher suggesting that the lateral force distribution changes at lower levels due to the presence of the podiums. In general, shear demands in Tower 2 buildings are slightly higher than in Tower 1 as evidenced by corresponding gross D/C ratio values.

Finally, it can be concluded that the values of the shear D/C ratio for both towers are lower for podiumbed-configured buildings (Types C and D), and that the maximum D/C ratio is reached for Tower 2 buildings. The fact that the increased ratio is observed in these buildings indicates that the podium levels change the load pattern which in turn affects lateral force distribution which in turn produces higher shears on transfer columns. However, the maximum D/C ratios still stay below the critical points; therefore, it can be concluded that the columns are not overloaded, but are close to it; specifically, Tower 2.

CONCLUSIONS

This work offers important information to investigate the response of RC buildings with TB especially on base shear, displacement patterns, and the column capacity ratios. Key findings are as follows:

Base Shear: The LTHA shows that there are higher shear demands in certain building layouts, particularly those with podiums (Types C and D). As suggested by the results in the present research, the increase in the base shear demand/capacity ratio is more pronounced in tall buildings (150m) implying need for redesign of taller buildings.

Displacement and Inter-Storey Drift: From this, it can be deduced that maximum roof displacements and inter-storey drifts are proportional to the building height with more displacement in the X direction than in the Y direction. Transfer beams have an important responsibility of increasing drift near lower floors especially in tall structures and thus require special focus on drift reduction measures.

Column Capacity Ratios: It is observed that circular columns possess higher values of PMM capacity ratios than the square ones, which imply that these members are more critically utilized in the combined axial and bending loads. Further, podium configurations change the lateral force distribution slightly and in turn increases the shear D/C ratios in transfer columns especially in Tower 2.

Design Implications: The results stressed again the need to adopt seismic performance measures including PMM and shear D/C ratios in the design to achieve ductility and eliminate brittleness. Podiums and transfer beams must be carefully located with respect to the lateral force distribution so as to enhance the building capacity.

Therefore, this work points to the importance of developing specific guidelines for the seismic design and retrofitting of RC buildings with transfer beams especially for high rise structures. Through the study of base shear, displacement, and column capacity ratios this study presents a course for enhancing the performance of such buildings during an earthquake and thereby enhance occupant safety and structural stability.

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