TORSIONAL AMPLIFICATION FACTORS FOR NON-STRUCTURAL COMPONENTS IN TORSIONALLY IRREGULAR BUILDINGS

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Structural Engineering

Submitted by

R Karunvir Bhardwaj

802324017

Under the supervision of

Dr. Himanshu Chawla
Assistant Professor
Civil Engineering Department

Er. Deepak Rana
Structural Engineer
Civtech Consultants

DEPARTMENT OF CIVIL ENGINEERING

THAPAR INSTITUTE OF ENGINEERING AND TECHNOLOGY

(DEEMED TO BE UNIVERSITY)

PATIALA-147004(PUNJAB)

AUGUST 2025

CIVTECH

Civtech

A-45, SECTOR-4, NOIDA - 201301 (U.P.)

Telefax : 0120-4511853, 4511854, 2544084
Email: admin@civtech.in

STRUCTURAL ENGINEERING, ARCHITECTURE, PROJECT MANAGEMENT

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15 July, 2025

This is to certify that **Mr. R. Karunvir Bhardwaj** has completed his internship at **CIVTECH** Noida from **15 July, 2024 to 15 July, 2025**

He has successfully completed his internship under the guidance of **Mr. Deepak Rana**, Senior Design Engineer.

He is a sincere and hardworking student.

We wish him success in his future endeavors.

Sincerely,
For **CIVTECH**

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DECLARATION

I hereby declare that this is a bonafide work which is presented in this project seminar report entitled "**TORSIONAL AMPLIFICATION FACTORS FOR NON-STRUCTURAL COMPONENTS IN TORSIONALLY IRREGULAR BUILDINGS**" as per the requirements for the award of Master of Engineering in Structural Engineering, submitted in the Department of Civil Engineering, Thapar Institute of Engineering and Technology (TIET), Patiala. This work is carried out under the guidance and supervision of Dr. Himanshu Chawla and Deepak Rana. It is declared that this work is original and has not been submitted anywhere else for the award of any other degree or certificate.

R Karunvir Bhardwaj

Date-29/08/2025

This is to certify that the above statement made by the candidate is correct to the best of our knowledge.

29/08/25

Dr. Himanshu Chawla

Assistant Professor

Civil Engineering Department

TIET

Er Deepak Rana

Structural Engineer

Civtech Consultants

Noida

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R Karunvir Bhardwaj
(802324017)

ABSTRACT

This study examines how acceleration-sensitive non-structural components' seismic response is affected by inherent building torsion. A set of elastic torsionally irregular step-back reinforced concrete moment-resisting frame buildings are examined under bi-directional seismic excitations in order to accomplish this goal. For different damping ratios of non-structural components, the floor torsional amplification factors are determined as a function of the tuning ratio. These factors are defined as the ratio between the floor spectral ordinate at the flexible/stiff edge and the floor spectral ordinate at the center of rigidity. In accordance with the floor displacement-based torsional irregularity indices suggested by the national building codes of the United States and India, the correlations between the peak torsional amplification factors at various floors for the rigid and flexible non-structural components are examined. The torsional amplification factors are shown to be depending on tuning ratio and building parameters. For both the rigid and flexible non-structural components, it is also found that these torsional amplification factors have a strong correlation with the respective floor displacement-based torsional irregularity indices. The very flexible non-structural components, on the other hand, have torsional amplification factors that trend to unity and are therefore seen to be independent of the building's and the non-structural components' features. To design acceleration-sensitive non-structural elements in torsionally irregular structures, floor displacement-based models that are simplified and numerically validated are suggested as a way to calculate the torsional amplification factors. These models can be combined with the current codes.

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CHAPTER 1

INTRODUCTION

1.1 General

The seismic performance of buildings is influenced not only by the structural system but also by the behavior of non-structural components, particularly such as masonry (partition) walls, furnitures, cladding systems, ceiling fixtures, and mechanical equipment. These non-structural components installed in buildings are sensitive to inertia forces, inter storey displacements, or sometimes to both. These components often sustain significant damage during earthquakes, even when the primary structure remains intact. Their vulnerability becomes more pronounced in buildings with irregular configurations—especially setback buildings, which exhibit abrupt changes in plan geometry along their height. This irregularity introduces complex torsional responses under seismic excitation, which are quantified using torsional amplification factors (TAFs). Setback buildings inherently possess uneven mass and stiffness distributions, causing a shift between the center of mass and the center of rigidity. This misalignment leads to torsional vibrations, which amplify lateral accelerations at various floor levels. Acceleration-sensitive non-structural components located near the edges or corners of such buildings are particularly susceptible to these amplified responses. Traditional design approaches often overlook these effects, leading to underestimation of seismic demands on non-structural systems. In addition to their essential role in maintaining the post-earthquake functionality of lifeline structures, non-structural components (NSCs) and building contents constitute a substantial proportion of the total construction cost. Consequently, seismic damage to these elements can result in significant economic losses and prolonged functional disruptions, particularly in commercial and institutional buildings. This underscores the critical need to evaluate the effectiveness of existing seismic design provisions for NSCs.

1.2 Setback and stepback buildings

A setback building is a structure where upper floors are progressively recessed (set back) compared to the lower floors. This results in a stepped profile rather than a uniform vertical façade. There are various types of Setback Configurations:

1. Step-Back Type: Upper stories are recessed at regular intervals.

2. Tower-Podium Type: A wide podium at the base and a slender tower above.

3. Sloped/Terraced Setback: Building steps follow the slope of terrain (e.g., hill buildings).

Setback buildings pose several structural challenges, particularly during seismic events. One of the primary concerns is torsional irregularity, which arises due to the asymmetrical distribution of mass and stiffness. This leads to complex torsional responses under lateral loads, often resulting in amplified displacements and stresses. Another critical issue is the soft storey effect, where lower levels—especially those beneath setbacks—exhibit significantly reduced stiffness, making them more vulnerable to excessive deformation and potential collapse. Additionally, setback configurations increase the likelihood of pounding effects, where adjacent building blocks or projections with different heights collide during earthquakes, causing localized damage. The discontinuity in structural geometry also disrupts the uniform flow of forces, creating an uneven load path that concentrates stresses in certain elements, thereby compromising the building's overall seismic performance.

Figure 1.1: Setback Building in New Delhi

Example: as shown in figure 1.1 Urban High-Rise Buildings - Improve daylight access, Hilly Terrain Developments-Utilize land effectively and reduce excavation, while maintaining structural stability, Modern Commercial Complexes-Functional zoning within the building, better visual appeal, and compliance with fire codes etc.

Step-back columns are vertical structural elements that are not continuous along the full height of a building, typically occurring in setback structures where the upper floors are recessed compared to the lower ones. Unlike regular columns that extend uniformly from the foundation to the roof, step-back columns terminate at intermediate levels due to the building's stepped profile. These columns introduce discontinuity in the vertical load path, resulting in abrupt changes in stiffness and strength between adjacent storeys. This can lead to stress concentrations at the termination point, which becomes a critical zone during lateral loading such as seismic forces. The sudden change in geometry also causes irregular distribution of forces and increases the risk of shear failure or excessive deformation in the adjoining beams and slabs.

Example: -as shown in Figure 1.2 on hilly terrain, where floor plans change with height, in terraced construction where upper floors are recessed, In parking levels or podiums where column grid changes on the superstructure.

Figure 1.2: Building with Stepback columns in Himalayas

1.3 Torsional irregularity

Torsion irregularity in buildings occurs when the center of mass and the center of rigidity do not coincide, causing the structure to twist or rotate about its vertical axis during lateral loading, such as from wind or earthquakes. This rotational motion leads to uneven distribution of forces across the structure, with certain parts of the building experiencing higher displacements and stresses than others. In plan-asymmetric buildings, such as those with setbacks, L-shapes, or irregular layouts, torsion irregularity becomes more pronounced. This can result in significant amplification of lateral displacements on the flexible edges or corners of the structure, increasing the likelihood of localized damage or failure during seismic events.

Torsion irregularity significantly affects floor accelerations, especially in seismic conditions. When a building twists due to eccentric lateral forces, different parts of each floor experience varying accelerations depending on their distance from the center of rotation. Points farther from the center of rigidity undergo amplified rotational and translational motion, resulting in higher peak floor accelerations (PFA) compared to centrally located areas. This variation in acceleration is critical for acceleration-sensitive non-structural components (like mechanical equipment, cladding, or ceilings), which are more prone to damage in corners or edges of torsionally irregular buildings. Moreover, the dynamic interaction between the twisting floor and non-structural components can lead to resonance effects, especially if their natural frequency aligns with the amplified torsional frequency of the floor. In seismic design, this calls for careful placement and anchorage of non-structural elements and, if necessary, incorporating torsional amplification factors in both structural and non-structural component design to ensure safety and performance under earthquake loading.

The Torsional Amplification Factor (TAF) is a multiplier used to account for the increase in torsional response of a building due to plan or stiffness irregularities, especially during seismic events. It quantifies how much the actual torsional displacement (usually at the corners of a floor diaphragm) exceeds the average translational displacement of the structure. The floor torsional amplification factors (TAFs), defined as the ratio between the floor spectral ordinate at the FE or the SE to the floor spectral ordinate at the CR are obtained as a function of the tuning ratio.

1.4 Non-structural components:

Non-structural components (NSCs) are elements of a building that do not contribute to its primary load-bearing system but are essential for functionality, safety, and aesthetics. These include architectural elements (e.g., partition walls, ceilings, cladding), mechanical and electrical systems (e.g., HVAC, piping, elevators), and building contents (e.g., furniture, storage systems). Although NSCs are not part of the structural frame, they can be significantly affected by building movements during earthquakes, especially floor accelerations and inter-storey drifts. In seismic events, poorly anchored or improperly detailed NSCs may fail, leading to economic losses, safety hazards, and disruption of essential services. For instance, the failure of suspended ceilings or mechanical equipment can block evacuation routes or damage vital systems like water supply and power.

The existing methods for seismic design of acceleration-sensitive NSCs can be categorized under different categories:

(i) estimating the equivalent static seismic response of NSCs

(ii) estimating the seismic response of NSCs using the principles of structural dynamics (e.g., combining floor acceleration response for individual modes of vibration, using a modal combination procedure, and

(iii) the direct spectra-to-spectra method (e.g., obtaining floor response spectra directly from the ground response spectra, using spectral amplification factors)

The estimation of design floor acceleration demands for acceleration-sensitive NSCs, using the above- discussed methods, is based on the assumption of a pure translational floor response of the building, i.e., the usual case, when a building is regular and symmetric, in plan and elevation. Though the regularity and symmetry in plan and elevation are considered as the fundamental premise for earthquake safety of buildings; however, achieving them may not be practically feasible in all the buildings due to a variety of reasons. As a result, many buildings are constructed in practicality even with the inherent structural irregularities either in their plan or in elevation, or sometimes even with a combination of both. The presence of either plan or elevation irregularities results in a significant change in the dynamic behavior of the building and hence the floor acceleration response.

Yang and Huang [25] showed that the torsional amplification of acceleration demands on NSCs depends on the modal participation factors for the torsional modes of vibration of the building. With an increase in the floor eccentricity in the building, the acceleration response of the NSCs gets amplified when the NSCs are tuned to structural modes of vibration.

1.5 Buildings on slopes:

Buildings constructed on sloping ground or hilly terrains present a range of architectural and structural complexities due to the natural unevenness of the site as shown in figure 1.3. These structures typically feature stepped foundations, columns of varying height, and split-level floors, which result in significant vertical and plan irregularities. These irregularities make such buildings more susceptible to both gravitational and lateral forces, especially during seismic events. Unequal column heights lead to non-uniform stiffness distribution. Shorter columns on the uphill side are stiffer and attract a disproportionately higher share of seismic forces, while the longer downhill columns are more flexible and tend to undergo larger displacements. This behavior creates a torsional imbalance, which can lead to increased inter-storey drifts, localized damage, and even collapse under strong ground motion. Such conditions also give rise to soft-storey mechanisms, particularly in the lower levels where longer columns are unsupported for greater lengths. Additionally, mass irregularity is often introduced due to the placement of rooms, tanks, or equipment predominantly on certain levels, further complicating the dynamic behavior. The lack of symmetry in both elevation and plan leads to irregular natural vibration modes, making seismic analysis and design more demanding.

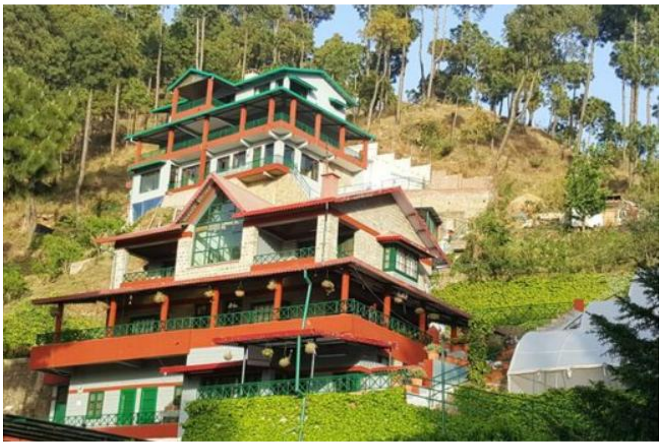


Figure 1.3: Setback building on slopes with stepback columns

CHAPTER 2

LITERATURE REVIEW

2.1 General

This chapter presents a review of the literature related to the key aspects of the study. Previous research on non-structural components is examined to understand their dynamic behaviour and contribution to the seismic performance of buildings. Studies on torsion irregularity index are discussed to outline various methodologies adopted for evaluating torsional effects. The review further includes investigations on buildings located on sloping ground, emphasizing the challenges and irregularities inherent to such configurations. Additionally, literature on torsional amplification factors is explored to establish their importance in assessing seismic response.

2.2 Literature Review

Kingston et al. 2006 explored the seismic response of acceleration-sensitive non-structural components (NSCs) mounted on regular elastic and inelastic moment-resisting frame (MRF) structures. Two-dimensional single-bay frame models of three, six, nine, and eighteen stories were used in the study, taking into account both rigid and flexible configurations, as seen in the image. 40 far-field ground movements recorded on rigid soil are applied to the NSCs, which are represented as elastic single-degree-of-freedom (SDOF) systems with minimal mass in relation to the frame. The DRAIN-2DX program is used to conduct time-history analysis, and both elastic and inelastic structural behaviors are investigated. Numerous influencing factors were examined in the analysis, such as the NSC's location within the building, its period and damping ratio, and the supporting frame's rigidity and strength.

Modal coupling has a significant impact on peak NSC accelerations, especially when the component's natural period and the supporting frame's modal period coincide. Acceleration demands were shown to rise with building height, with the largest amplifications occurring in roof-level NSCs. Because of energy dissipation, structural inelasticity has been demonstrated to decrease component accelerations, especially in the vicinity of the first-mode period. This impact is measured using a modification factor. The work further emphasizes how sensitive acceleration demands are to NSC damping, showing that accelerations two to three times larger than those seen with conventional 5% damping can occur at lower damping ratios (e.g., 0.01%). Damping ratios of 0.01%, 2%, and 5% were taken into account.

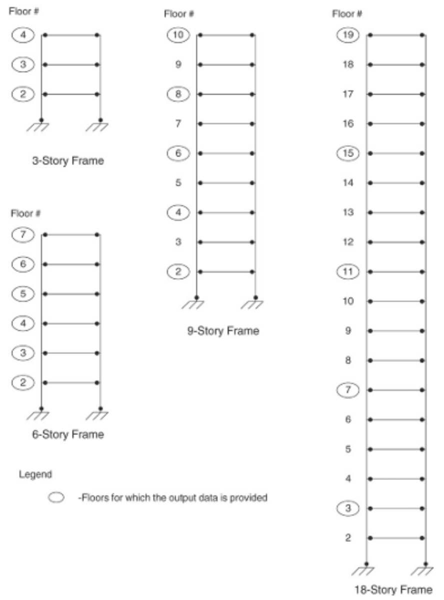


Figure 2.1: Building models and levels for which floor acceleration data recorded [1]

The fact that NSC accelerations are extremely sensitive to resonance with the supporting frame's modal periods is one of the main conclusions. When the component period coincides with one of the building's natural modes, the maximum accelerations take place. Furthermore, the shape of the floor response spectra changes greatly with both height and location, and NSCs at higher floors typically experience stronger accelerations. The paper also emphasizes how energy dissipation caused by structural inelasticity causes peak accelerations to be deamplification, especially in the vicinity of the fundamental period. This behavior is not adequately represented by existing design standards. Current seismic code provisions for NSCs, which employ fixed amplification factors and simplified height scaling, are frequently unconservative, according to a key result of the article. In many cases, especially for lightly damped components or when higher-mode effects are significant, actual demands exceed code predictions.

B. King et al. (2010) conducted a detailed evaluation of acceleration demands on non-structural components (NSCs) in ductile moment-resisting frame buildings. The study examines the behavior of light, linear NSCs found in steel and reinforced concrete buildings using a floor response spectrum (FRS) technique, as illustrated in the picture. Ultimate Limit State (ULS) and Serviceability Limit State (SLS) are the two seismic performance thresholds at which the analysis is carried out. According to the report, NZS 1170.5:2004 provisions frequently don't match real demands. In particular, it is discovered that the floor height coefficient (FHC), which is intended to represent the rise in peak floor acceleration (PFA) with building height, is excessively liberal. On the other hand, it is demonstrated that the component amplification factor (CAF), which takes into consideration dynamic amplification brought on by the component's natural period, is inadequate, particularly under SLS circumstances. These discrepancies suggest that the NZS design formula does not accurately capture resonance effects or the nonlinear behavior of real structures.

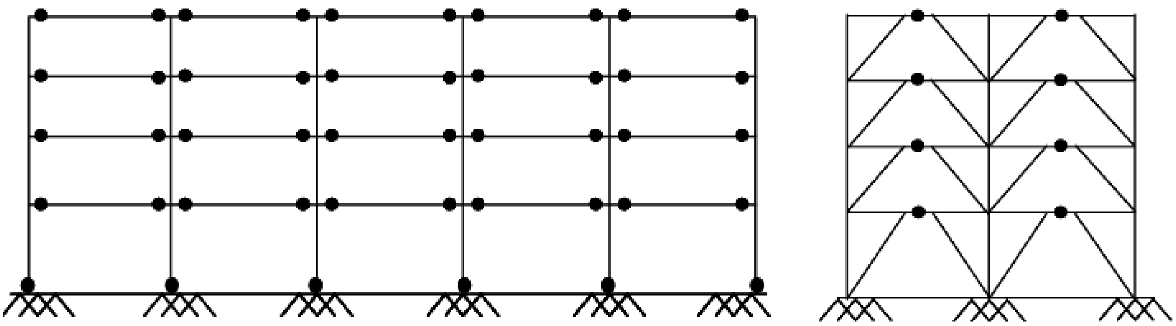


Figure 2.2: Building elevation considered for the evaluation [2]

One significant finding from the study is that acceleration demands on NSCs may be greater under SLS ground movements than under ULS motions when the building reacts primarily elastically. This is due to the fact that resonance effects become noticeable when the natural period of the component coincides with one of the modal periods of the building. On the other hand, under ULS conditions, the structural system's nonlinearity tends to lessen floor accelerations close to the basic mode, however higher modes may still cause amplification, especially in short-period NSCs on lower floors.

The study also demonstrates that, contrary to what NZS anticipated, peak floor accelerations do not rise noticeably with building height. Rather, PFAs are almost constant across height, especially in inelastically reacting buildings. Data collected from instrumented buildings in New Zealand also supports this conclusion. According to the study, normalized period ratios (Tp/TB1) more accurately reflect dynamic amplification effects and are more in line with international standards like ASCE 7 and IBC, which take into account both the component period and the modal features of the building.

The comparison of subduction slab ground motions with crustal ground motions is another important contribution. The authors demonstrate that slab events, especially under SLS, are more harmful to short-period components due to their high frequency content. This demonstrates how ground motion's frequency content significantly affects NSC requirements and ought to be specifically taken into account during design. The study shows that while NZS provisions may be sufficient for the majority of components under ULS, they are frequently unconservative under SLS, particularly for high-rise buildings' long-period NSCs. In order to reduce damage during mild earthquakes when structures are expected to stay operational, it was suggested that design criteria for NSCs be connected to the dynamic response characteristics and shaking intensity of the building.

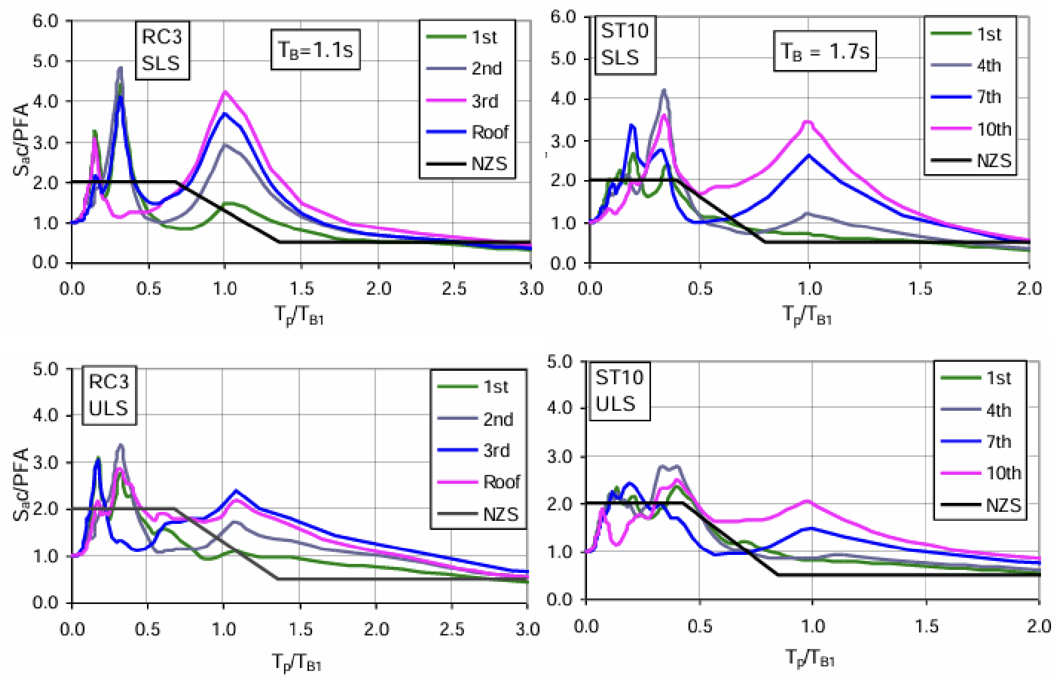


Figure 2.3: Component amplification factor for ULS & SLS model [2]

Mohsenian et al. (2019) presented a novel performance-based methodology for predicting the seismic demand of acceleration-sensitive non-structural components (NSCs), addressing the limitations of current code-based design provisions. The major structural system's performance level and the variation in seismic intensity are frequently overlooked by current standards like ASCE 7-16 and NIST guidelines, which can result in unsafe or unduly conservative designs. In order to predict the seismic actions acting on NSCs based on seismic hazard and the structural system's performance capacity, the authors introduce two crucial parameters: absolute demand acceleration (ADemand) and absolute capacity acceleration (ASupply). The authors assess the seismic reliability of NSCs utilizing fragility curves and incremental dynamic analysis (IDA) in a thorough investigation encompassing five and ten-story steel moment-resisting frame buildings situated in a high seismicity region.

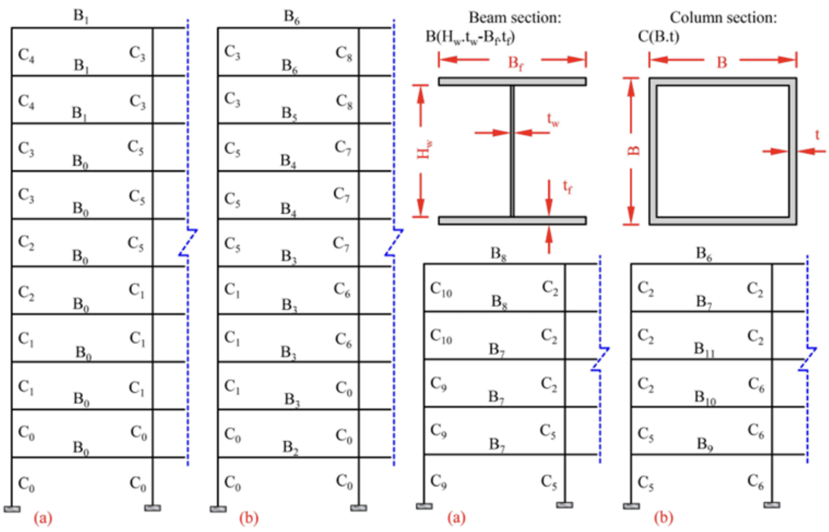


Figure 2.4: Building considered [3]

Figure 2.5: Extensive damages to exterior and facade walls of buildings despite the acceptable performance of the lateral load-bearing systems in the Kermanshah earthquake, 2017 [3]

Table 2.1: Earthquake data selected [3]

Record no.	Earthquake & year	Station	R^a (km)	Component	Mw	PGA (g)
R_1	Imperial Valley, 1979	El Centro Array#11	29.4	230	6.5	0.38
R_2	Chi Chi (Taiwan), 1999	TAP095	109.01	90	7.6	0.15
R_3	Loma Prieta, 1989	CDMG58224	72.2	290	6.9	0.24
R_4	Loma Prieta, 1989	CDMG58472	74.26	270	6.9	0.26
R_5	Kobe (Japan), 1995	HIK	95.72	0	6.9	0.14
R_6	Loma Prieta, 1989	CDMG58223	58.65	0	6.9	0.23
R_7	Manjil (Iran), 1990	Qazvin	49.97	336	7.4	0.13
R_8	Loma Prieta, 1989	Capitola	27.0	0	7.1	0.53
R_9	Landers, 1992	Yermo Fire Station	86.0	270	7.3	0.24
R_{10}	Duzce (Turkey), 1999	Bolu	41.3	90	7.1	0.82
R_{11}	Imperial Valley, 1979	Delta	33.7	352	6.5	0.35
R_{12}	Northridge, 1994	Canyon Country-WLC	26.5	270	6.7	0.48

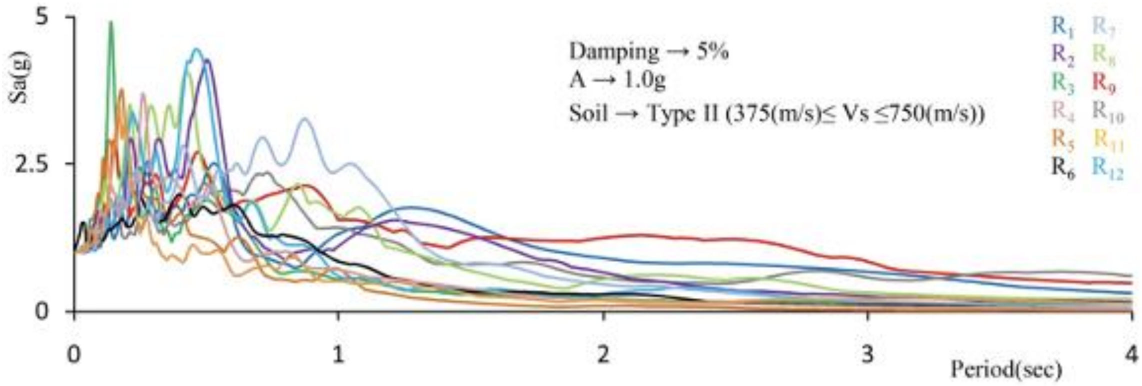


Figure 2.6: Spectral acceleration of selected records [3]

The findings show that the acceleration demands on NSCs under design-level ground vibrations are greatly underestimated by up to 80% by current code-based models, such as those in ASCE 7-16. NSCs may suffer harmful accelerations even if the lateral load-resisting system satisfies the life safety performance goal. The study also showed that a safer and more sensible range for design accelerations was offered by employing percentile-based demand and capacity estimates, such as the 16th percentile of demand and the 84th percentile of supply. The study underlined that the seismic hazard's severity and performance level, which are frequently disregarded in simplified code expressions, have an impact on the dynamic amplification that NSCs face in addition to floor height and structural period. Compared to traditional techniques, this multilevel approach is a major improvement and is more in line with the principles of performance-based seismic design. The suggested methodology is generalizable and can be modified to different structural types with the use of suitable fragility data, even though it has been validated for medium-rise, conventional steel frame buildings. The study also admits that less computationally demanding dependability techniques like nonlinear pushover analysis could make practical implementation easier.

Assi et al. (2025) presented a detailed probabilistic framework for assessing the seismic acceleration demands of acceleration-sensitive, light non-structural components (NSCs) mounted in moderately ductile reinforced concrete (RC) moment-resisting frame buildings. The authors concentrate on how the ductility of NSC attachments and the nonlinear behavior of the supporting structure affect NSC seismic demands and damage probability, acknowledging the shortcomings of existing code-based force calculations. The three-, six-, nine-, and twelve-story archetype RC structures were designed and put through 24 ground motion tests (12 historical and 12 synthetic) that were appropriate for Montreal's seismic risk.

The main conclusions show that, particularly in the presence of elastic structural behavior, enhanced ductility in NSC attachments dramatically lowers acceleration demands. The conservative bias of elastic analytic assumptions is highlighted by the fact that acceleration demands still decrease in nonlinear structures, but to a smaller extent. Acceleration responses are substantially amplified by resonance effects, especially when NSC periods coincide with the structure's fundamental or second modes. NSCs at the roof and intermediate levels are significantly more fragile than those at the bottom floor, according to fragility curves created for various ductility levels and resonance circumstances. Furthermore, the threshold acceleration needed to reach each damage state increases as NSC attachment ductility increases.

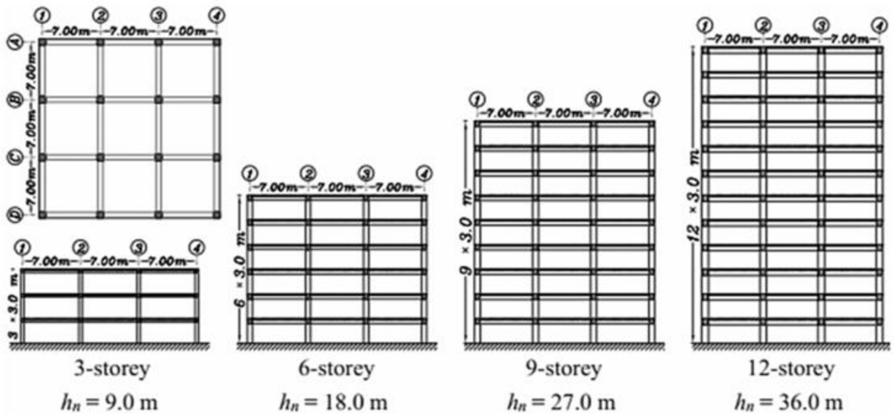


Figure 2.7: Plan and elevation of the building considered [4]

The study also points out that existing code provisions that rely on elastic response assumptions, such those found in the National Building Code of Canada (NBC), have a tendency to overstate seismic demands by as much as 115% in specific situations. This results in overly conservative NSC designs, particularly for those that are tuned to resonate with the initial mode of the building. Nonlinear-elastic demand ratios and comprehensive data for peak floor accelerations across various time periods and ductility levels are introduced.

Roof-level NSCs are much more susceptible to seismic damage than intermediate or ground-level NSCs. The seismic behavior of NSCs varies dramatically as resonance moves from the fundamental resonance period T_1 to higher ones (T_2, T_3). Especially in taller buildings, this change leads to decreased sensitivity to attachment ductility and structural nonlinearity. Elevated ductility levels may raise the possibility of localized damage to NSCs and their attachments, even though they also lessen seismic forces. Thus, in NSC seismic design, striking the ideal balance between ductility and allowable damage levels is crucial. Enhancing the precision of probabilistic seismic evaluations of NSCs requires taking into consideration the nonlinear behavior of the supporting structure. This factor simplifies seismic design requirements for NSCs by affecting acceleration demands uniformly throughout damage states and resonance ranges.

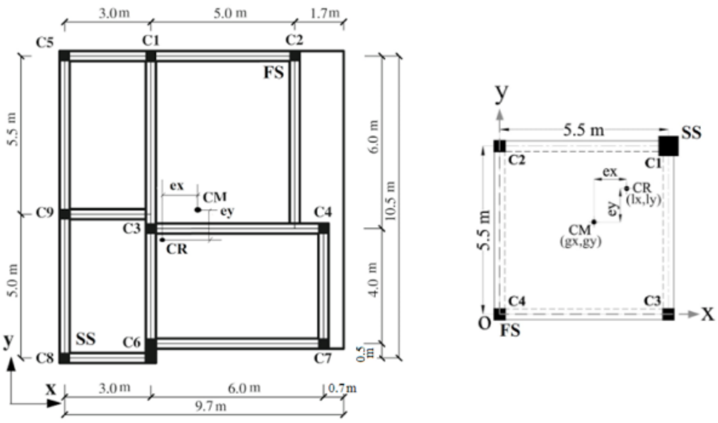
Sarno et al. (2025) presented a detailed and methodologically rigorous evaluation of the seismic reliability of tunnel-form concrete buildings, with a particular focus on the role of non-structural components (NSCs). The authors point out a significant gap in the literature, notably the disregard for NSCs in seismic reliability studies, despite the tunnel-form system's positive seismic performance as a structural system. A multi-level and multi-objective framework is used in the study to differentiate between acceleration-sensitive NSCs (such as façade panels), displacement-sensitive NSCs (such as staircases), and structural components. The study compares structural and non-structural performance across a range of seismic intensities, including design basis earthquakes (DBE) and maximum considered earthquakes (MCE), using five- and ten-story cast-in-place RC tunnel-form buildings that were modeled and examined using nonlinear time-history simulations.

One important finding is that, even in cases when the structural system operates within immediate occupancy or life safety restrictions, acceleration-sensitive NSCs considerably lower overall building reliability. When included in system-level performance evaluations, these components frequently sustain moderate to severe damage during demand-level and capacity-level earthquakes, resulting in dependability losses of up to 100%. On the other hand, displacement-sensitive NSCs exhibit sufficient seismic safety, taking use of the tunnel-form walls' natural rigidity. Additionally, the authors show that present seismic code requirements, including those included in Iranian and U.S. standards, significantly underestimate the acceleration demands for NSCs, especially in upper floors, with inaccuracies often above 50%. In order to solve this, they provide a novel empirical formula specifically designed for tunnel-form systems that forecasts peak floor acceleration (PFA) with noticeably smaller error margins, which has been confirmed by thorough time-history studies.

Figure 2.8: Extensive damage to non-structural components during earthquake [5]

The study illustrates the discrepancy in damage coordination between structural and non-structural parts using a combination of reliability models, probabilistic demand-capacity assessments, and sensitivity analysis. The study's multi-objective reliability curves make it abundantly evident that NSCs dominate a building's reliability profile—a fact that is frequently disregarded in traditional design methodologies. A paradigm shift in seismic design philosophy is ultimately advocated by the study, which calls for the incorporation of NSC performance into the foundation of performance-based earthquake engineering for tunnel-form structures. Additionally, it urges that current code-based force estimates be updated to more accurately represent the dynamic responses that are really seen in these kinds of systems.

Aldeka et al. (2022) a significant advancement in the seismic design of acceleration-sensitive non-structural components (NSCs) by addressing the limitations of Eurocode 8 (EC8) in predicting NSC demands in irregular reinforced concrete (RC) buildings. The authors stress that torsional behavior and irregularities in plan and elevation, which can greatly increase NSC responses, particularly at the flexible sides and top floors of buildings, are not sufficiently taken into account by traditional seismic provisions. A modified design model for the acceleration amplification factor utilized in EC8 is introduced in the study to close this gap. It has been calibrated using more than 5000 nonlinear dynamic finite element analyses on 33 irregular RC structures. The results are reliable and broadly applicable because the assessments take into account a variety of factors, including ground types, seismic capacities, plan layouts, and eccentricity ratios.



a) Groups 1, 2, 3 (G1, G2, G3) b) Group 4 (G4)

Figure 2.9: Building considered [6]

The incorporation of torsional effects through a suggested torsional amplification factor (FT), which is closely related to the top-floor rotational reaction, is a significant innovation of the work. In order to quantify the structure's resistance to seismic forces, the authors additionally present a dimensionless seismic capacity factor. The oversimplified original EC8 expression is replaced with an enhanced acceleration demand equation that incorporates these parameters. The suggested model outperforms EC8 in terms of accuracy, especially for NSCs whose periods resonate with the fundamental or torsional modes of the supporting structure, according to thorough comparisons with finite element (FE) results. The model is further tested across different ground types, eccentricity ratios, and structural heights. It continuously demonstrates superior predictive capability, with errors falling from 36% under EC8 to less than 10% in most cases.

All things considered, the study offers strong proof that adding maximum seismic capacity and torsional impacts to design models results in safer and more accurate forecasts of NSC acceleration needs. This study closes a significant gap in seismic design guidelines and offers a clear way to improve Eurocode-based procedures, particularly for irregular buildings where NSCs are most at risk.

Martinelli et al. (2019) investigated the dynamic behavior of acceleration-sensitive non-structural components (NSCs) in buildings subjected to seismic action. The authors suggested an improved method that takes into account the coupled response of the NSCs and the primary structure after realizing that existing seismic code provisions, such as Eurocode 8, oversimplify the estimation of seismic forces on NSCs. In order to achieve this, the study models the building-component system as a nonlinear two-DOF system in which the NSC maintains its elasticity while the main structure is permitted to yield. 264 natural ground motions are used in a comprehensive parametric analysis, with important parameters including the mass ratio, force reduction factor (R), and the structure and NSC period being varied.

The findings demonstrate that the nonlinear behavior of the primary structure has a substantial impact on the seismic demand on NSCs, particularly in the vicinity of resonance situations where the NSC's period coincides with the structural period. The results show that the NSC acceleration demands are significantly impacted by the force reduction factor R, which takes into consideration the ductility of the primary structure and is typically overlooked in design guidelines. The amplification factor (AF), which compares NSC demands for inelastic versus elastic structural responses, and the resonance factor (RF), which shows the ratio of peak NSC acceleration to structural base acceleration, are two novel indices that are presented. The study reveals that mass ratio has negligible influence within typical practical ranges, while period ratio (Ta/T1) and R are the dominant factors. Significantly, the study demonstrates that Eurocode 8 frequently misrepresents or understates NSC requirements, especially for flexible and highly ductile NSCs. The study makes a stronger case for updating codified acceleration amplification formulations by defining probabilistic distributions for RF and integrating record-to-record variability analysis.

Medina et al. (2007) investigated the nonlinear (inelastic) behavior of moment-resisting frame (MRF) structures affecting the seismic acceleration demands on acceleration-sensitive non-structural components (NSCs). The authors suggest an acceleration response modification factor (R_{acc}) to measure how inelasticity alters peak component acceleration (PCA) demands because they acknowledge that the majority of current design codes rely NSC seismic forces on elastic assumptions for the basic structure. The paper assesses floor response spectra (FRS) for different NSC damping ratios, periods, and locations within structures using comprehensive nonlinear time-history simulations of MRF buildings with three to eighteen stories and forty far-field ground motions. It has been demonstrated that inelastic structural behavior typically lowers acceleration demands on NSCs close to the building's fundamental mode, but that modal interactions can amplify these demands in other frequency ranges, especially in the high-frequency (short-period) region. The R_{acc} factor varies significantly across the height of the building, NSC period, and damping ratio, with values ranging from 0.13 to 23, highlighting the limitations of a one-size-fits-all amplification factor as assumed in many seismic codes.

Three spectral regions—long-period, fundamental-period, and short-period—are identified in the study, and R_{acc} behavior is examined in each. Amplification can happen in the short-period range, particularly at lower floors and for moderately damped NSCs, where R_{acc} may be < 1, indicating greater demand than in the elastic case, although inelasticity tends to diminish demands in the fundamental period area (i.e., $R_{acc} > 1$). The results highlight how crucial it is to take into consideration component damping, structural ductility, height of the supporting structure, and NSC placement while designing for seismic activity. The study also demonstrates that, particularly for medium- to long-period buildings, the fundamental period of the structure (T_B_1) has less of an impact on R_{acc} than other characteristics.

All things considered, the study offers a thorough, statistically backed case for improving seismic design guidelines for NSCs by taking structural nonlinearity effects into account. A logical and useful method for converting elastic floor response spectra into more realistic inelastic demands is the suggested R_{acc} factor. For important or light NSCs in particular, where oversimplified elastic estimations could result in dangerous or unduly conservative designs, these discoveries have significant ramifications for performance-based seismic design.

Miranda et al. (2018) discussed recent advancements in the seismic design of non-structural components have emphasized the limitations of existing code provisions, particularly for acceleration-sensitive elements. Analyze critically the drawbacks of present design approaches, which mostly rely on amplification factors that are not supported by empirical evidence and simplified equivalent static force procedures. Given their vulnerability to damage at lower seismic intensities than structural elements, their study emphasizes the critical role non-structural components play in total building performance and economic loss. The authors point out that, particularly in the vicinity of the building's modal frequencies, floor vibrations within buildings frequently deviate significantly from ground motions in terms of both amplitude and frequency content. Conventional clauses overlook these subtleties, which could cause demands to be overestimated or underestimated. They suggest a novel design philosophy to solve this, in which bracing systems are designed to function as fuses, limiting the seismic stresses that are passed to non-structural elements and their attachments.

By purposefully creating nonlinearity in the bracing parts, this method not only lowers the demands for force and deformation but also minimizes uncertainties by managing failure processes. Their results mark a major advancement in performance-based design approaches for non-structural systems, especially in key facilities where function retention after an earthquake is crucial.

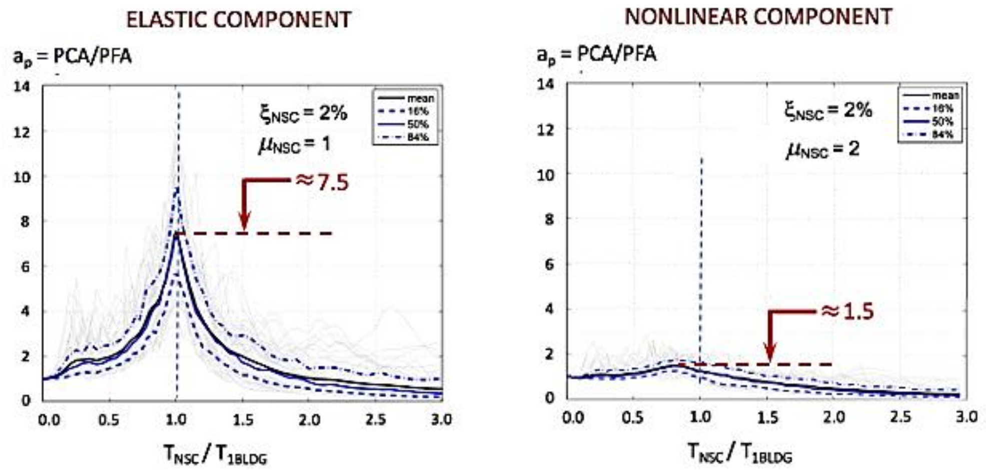


Figure 2.10: Floor spectra graph [9]

Perrone et al. (2019) conducted a detailed investigation into the seismic performance of non-structural elements (NSEs) during the 2016 Central Italy earthquake, emphasizing their significance in overall building resilience and post-earthquake functionality. According to the study, non-structural damage was widespread and frequently had a greater effect than structural damage, especially in important institutions like historical buildings, hospitals, and schools. Masonry infills and internal walls, suspended ceiling systems, storage racks, pipes, and decorative architectural elements were the most commonly damaged parts. Over 700 structures were inspected, and the results showed that poor connection details, lack of seismic design, and insufficient anchorage were mostly to blame for the damage, particularly in buildings constructed before the 1970s. In-plane and out-of-plane collapses of masonry infills were mostly caused by severe inter-story drift and floor accelerations. Poor perimeter fastening and a lack of bracing frequently caused ceiling systems to collapse. Joint failures occurred in piping systems, and storage racks that were not secured were prone to toppling. The investigation also found that there were significant safety issues due to the deterioration or collapse of heritage features such frescoes, vaults, and stuccoes. On the other hand, structures with basic mitigating techniques, such as perimeter clearances, anchored pipes, or shelf bracing, demonstrated noticeably improved NSE performance.

Figure 2.11: Collapse of infill panels [10]

This highlighted how vital it is to include NSE-focused seismic design in building standards, particularly in Europe where there aren't many of these options. According to the study's findings, enhancing NSE design can significantly lower financial losses, avoid operational disruptions, and improve life safety during future earthquakes.

Figure 2.12: Typical damage to masonry infills. internal partition [10]

Figure 2.13: Damage to plasterboard ceiling system [10]

Surana et al. (2017) address a critical gap in seismic design by exploring how structural inelasticity, expressed through the response reduction factor (R), affects peak floor acceleration (PFA) demands—parameters that are vital for safeguarding acceleration-sensitive non-structural components (NSCs). The authors demonstrate that the period and strength ratio (R_l) of the structure have a significant impact on PFA demand through comprehensive numerical research that includes 1120 nonlinear dynamic calculations on four mid-rise RC moment-resisting frame buildings.

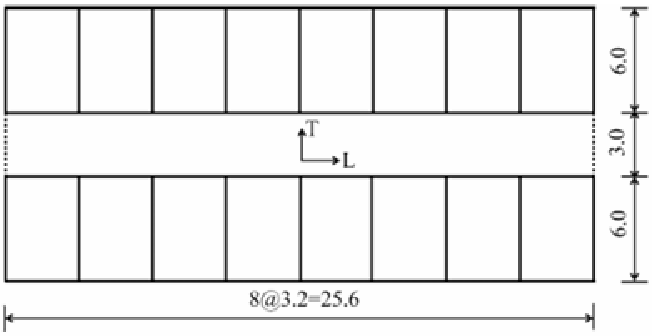


Figure 2.14: Plan of building considered [11]

The paper finds that floor acceleration needs dramatically decrease with increasing building flexibility (higher T_1) and ductility (higher R_l), particularly at the roof level. Existing seismic codes such as IS 1893, ASCE 41, EN 1998, and NZS 1170.5, which often assume a linear increase in acceleration along the height and ignore dynamic phenomena like higher-mode involvement and the whiplash phenomenon, stand in stark contrast to this discovery.

The work suggests a height-wise PFA prediction model that incorporates three crucial parameters—relative floor height (Z/H), fundamental period (T_1), and strength ratio (R_I)—in order to address these disparities. Validation with spectrum-compatible time-history studies demonstrates that this model is still appropriate for both elastic and inelastic structures and more accurately depicts the variation of PFA than the present code provisions. A step toward more performance-based and logical seismic design of RC structures in India and elsewhere, this work provides an improved method for calculating seismic demand on NSCs.

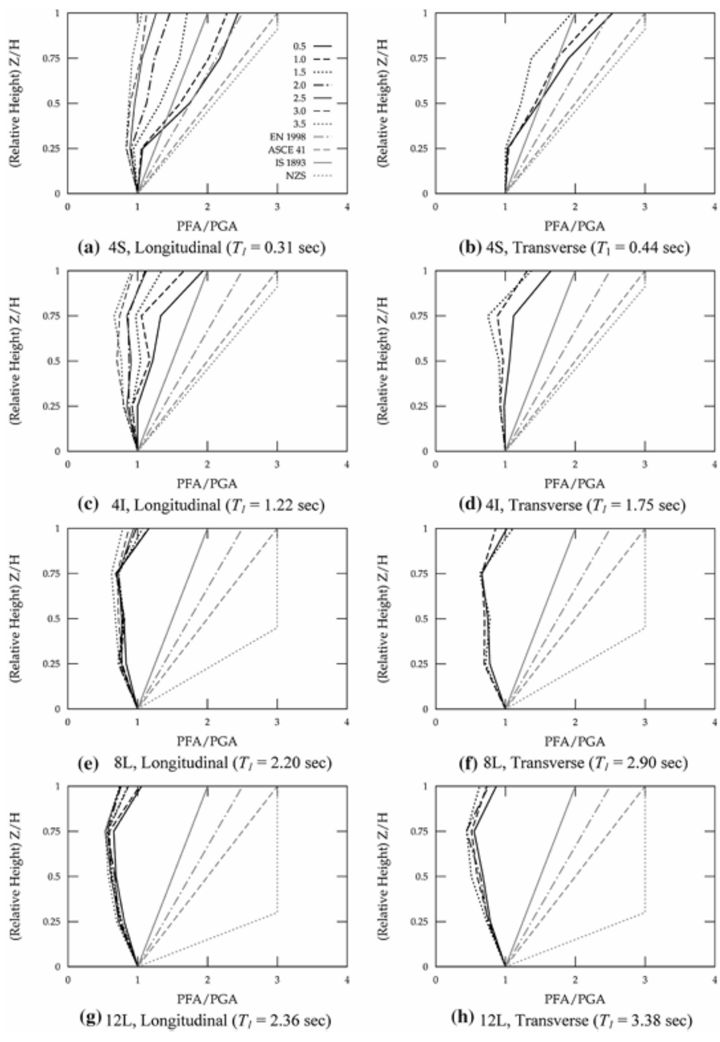


Figure 2.15: Median PFA/PGA demands [11]

Sarkar et al. (2010) present a focused investigation into the dynamic behavior of stepped building frames, a form of vertical geometric irregularity increasingly common in urban architecture. Current seismic design codes, such IS 1893 and ASCE 7, are criticized in the paper for failing to sufficiently account for the intricate mass and stiffness fluctuations brought about by stepped geometries. In contrast to traditional geometric ratios or previous dual-index techniques, the authors suggest a novel regularity index (η) that incorporates both mass and stiffness distributions to more properly characterize irregularity. This index is generated from modal participation components.

They show that greater irregularity causes a change in modal behavior, with less first-mode dominance and higher-mode participation, by conducting in-depth investigations on 78 RC moment-resisting frames with different step configurations and heights. The study also suggests a revised empirical formula based on the regularity index for determining the fundamental period of stepped buildings. Strong agreement between predicted and computed fundamental periods is demonstrated by validation using both 2D analytical models and a 3D case study of the Delhi Secretariat building.

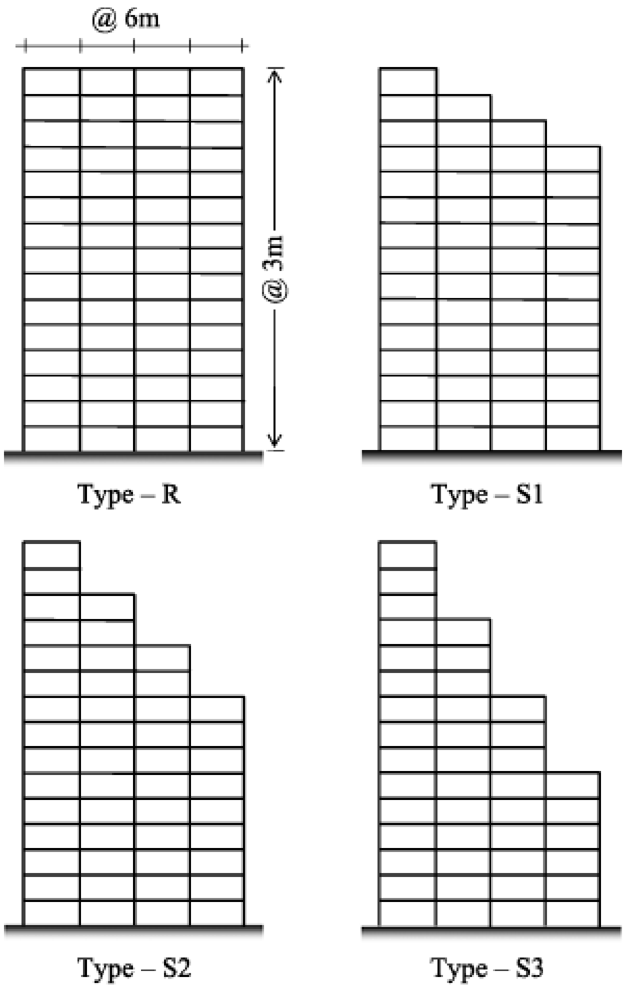


Figure 2.16: Buildings considered [12]

Figure 2.17: Stepped Building Located in New Delhi [12]

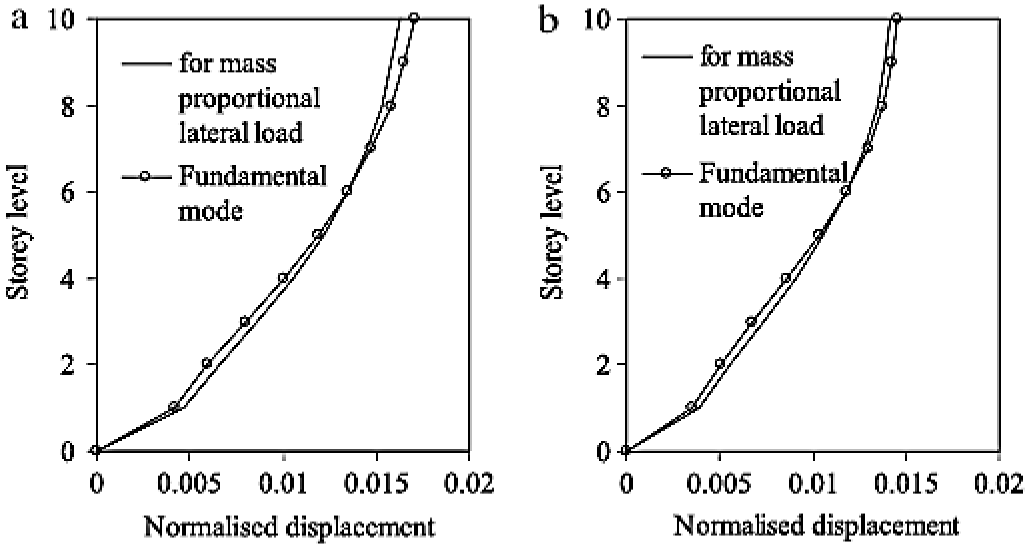


Figure 2.18: Comparison of static displacement shape and fundamental mode shape for (a) selected stepped building and (b) similar regular building [12]

Venkataramana et al. (2016) investigated into the seismic behavior of vertically mass irregular RC framed buildings with in-plan eccentricity, using time history analysis on 375 finite element models subjected to El-Centro ground motion. Important dynamic response characteristics were examined for different mass ratios and eccentricity configurations in 5-, 10-, and 15-story buildings. These parameters included natural period, base shear, roof deflection, and roof rotation. The findings show that in-plan eccentricity significantly increases torsional effects, especially when upper-story mass anomalies are present. When compared to regular frames, there was a maximum increase of 116.5% during the natural period and 138.5% during roof rotation. The authors suggest a new irregularity index (α) to measure this influence, which takes into account the position and magnitude of in-plane eccentricity and vertical mass irregularity. Strong association and practical design applicability are demonstrated by regression-based predictive equations for base shear and natural period based on α. Clear guidelines for structural design in seismic zones are provided by the study's conclusion that putting heavy or eccentric masses at lower levels greatly enhances seismic performance. The natural period of frames with vertical mass irregularity at the top level is 8–44% longer than that of frames with irregularity at the bottom. When masses are positioned at the upper levels of the frames, the maximum amount of roof rotation occurs by 16–67% greater than when masses are positioned at the lower levels.

Tluanga et al. (2024) investigated the geotechnical and structural interplay affecting slope stability in hilly terrains, with a particular focus on the urban context of Aizawl, a seismically active and topographically complex city. Utilizing a methodologically sound framework, the study evaluates the factor of safety under various slope geometries and structural loading circumstances utilizing the Limit Equilibrium Method, more especially the Morgenstern-Price methodology, which is implemented through Slope/W software. With geotechnical metrics including soil cohesiveness, angle of internal friction, and unit weight obtained from established laboratory testing in accordance with ASTM protocols, the study integrates comprehensive site-specific data from 50 chosen locations. Together with slope geometry and structural loadings calculated using Staad Pro v8i, these input parameters provide a thorough and accurate modelling environment.

The thesis makes a significant contribution by comparing two popular architectural configurations that are found throughout Aizawl's urban fabric: step-back buildings and step-back setback structures. Through the use of seismic coefficients that correspond to Zone V according to IS 1893 (Part 1):2016, the study methodically measures the impact of different structural typologies on slope stability under both static and dynamic (seismic) loading conditions. Higher factor of safety values across all slope categories examined show that the step-back setback topology consistently demonstrates greater slope stability. This is explained by improved load distribution properties, decreased concentrations of shear stress, and a more stable relationship with the slope geometry.

This study includes a sensitivity analysis on the shear strength parameters as well, emphasizing how the angle of internal friction (ϕ) and cohesion (c) varied in their effects under different loading conditions. The results show that cohesion is the primary element affecting stability in free slope settings, but the angle of internal friction becomes more crucial when structural and seismic loads are included. This sophisticated comprehension highlights the significance of context-specific parameter evaluation in slope stability modelling and offers insightful information about soil-structure interaction.

The study also looks at how slope geometry affects the failure surface, showing that higher slope height and angle result in deeper and narrower critical slip surfaces, which lowers the overall factor of safety. It's interesting to note that, although the critical slip surface's shape is rotational and essentially constant across building types, the factor of safety's magnitude varies greatly based on the applied structural load, supporting the idea that building topology, not failure surface morphology, determines stability magnitude.

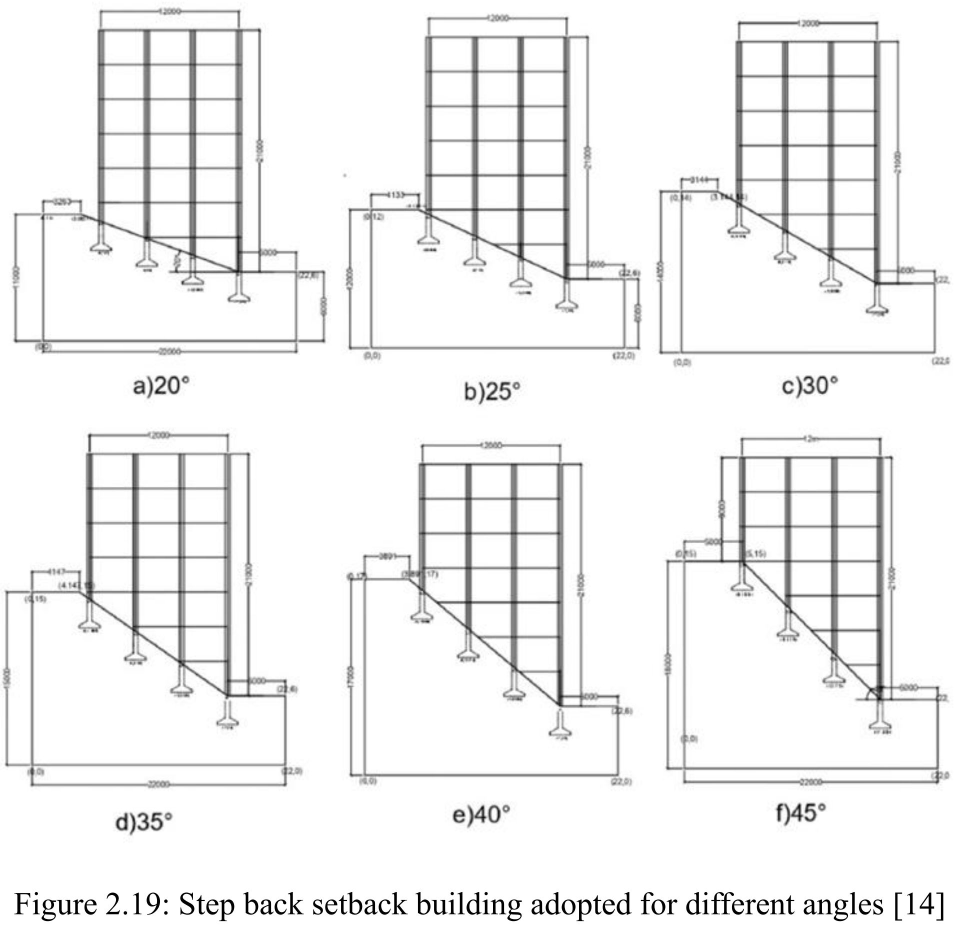
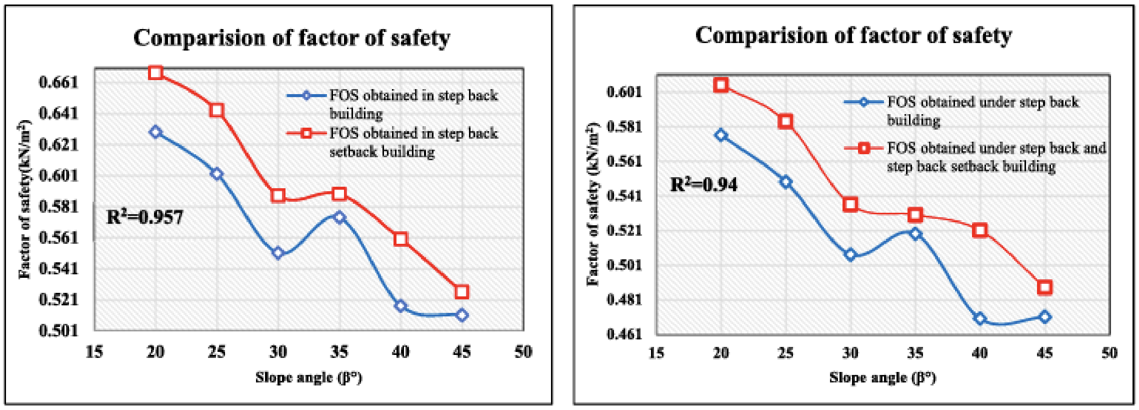


Figure 2.19: Step back setback building adopted for different angles [14]



(a) FOS obtained with building loads under static conditions. (b) FOS obtained with building loads under seismic conditions.

Figure 2.20: Comparison of the factor of safety obtained with step back building and step back setback building [14]

The conclusions are in line with accepted geotechnical engineering concepts and pertinent literature, and they are substantiated by practical data and analytical results. The thesis offers specific suggestions for urban development in hilly areas, supporting the use of step-back setback building designs as a workable way to reduce the risk of landslides. Additionally, the effort advances knowledge of integrated geotechnical and structural design in metropolitan regions that are dominated by slopes and are seismically sensitive.

Magapu et al. (2023) detailed and methodologically advanced study focused on evaluating the seismic performance of hill buildings using Incremental Dynamic Analysis (IDA) and fragility curves. By taking into consideration the structural irregularities present in buildings on slopes, especially step-back and step-back setback configurations, which are typical in the hilly regions of the Indian subcontinent, the study seeks to close a significant gap in the assessment of seismic susceptibility.

Five distinct reinforced concrete (RC) structural configurations have been modelled by the authors using a strong numerical framework that incorporates actual geometrical and material features. The SeismoStruct platform was used to assess these models, and ground motions were chosen from a variety of actual earthquake incidents. In order to verify spectrum compatibility, the selected seismic records were scaled in accordance with ASCE/SEI 7–10 principles. Damage was quantified using engineering demand parameters (EDPs), such as inter-story drift, and intensity was measured using peak ground acceleration (PGA).

Table 2.2: Ground motion selected [15]

Sr.no	Earthquake Name	Station Name	Magnitude	Mechanism	Distance from the source(km)
1	"San Fernando"	"Lake Hughes #12"	6.61	Reverse	13.99
2	"Friuli_ Italy-01"	"Tolmezzo"	6.50	Reverse	14.97
3	"Imperial Valley-06"	"Cerro Prieto"	6.53	Strike Slip	15.19
4	"Irpinia_ Italy-01"	"Calitri"	6.90	Normal	13.34
5	"Corinth_ Greece"	"Corinth"	6.60	Normal Oblique	10.27
6	"New Zealand-02"	"Matahina Dam"	6.60	Normal	16.09
7	"Loma Prieta"	"Gilroy Array #6"	6.93	Reverse Oblique	17.92
8	"Northridge-01"	"Beverly Hills – 12520 Mulhol"	6.69	Reverse	12.39
9	"Hector Mine"	"Hector"	7.13	Strike Slip	10.35
10	"Chuetsu-oki_ Japan"	"Yoshikawaku Joetsu City"	6.80	Reverse	13.68
11	"Iwate_ Japan"	"AKT023"	6.90	Reverse	11.68

The study's thorough modelling of buildings with mass eccentricity and short column defects—defects that are frequently disregarded but have a big impact on seismic behavior—is one of its main advantages. Based on FEMA 273, fragility curves are developed for each model across five performance levels (OP, IO, DC, LS, and CP). This gives the vulnerability evaluation a probabilistic component and makes it possible to see performance thresholds under rising seismic demand more clearly. According to the results, step-back setback frames are more seismically resilient than step-back frames. Furthermore, it is well known that short column effects and mass eccentricity have a detrimental influence on seismic performance, increasing torsional loads and causing damage to begin earlier, particularly in lower performance states.

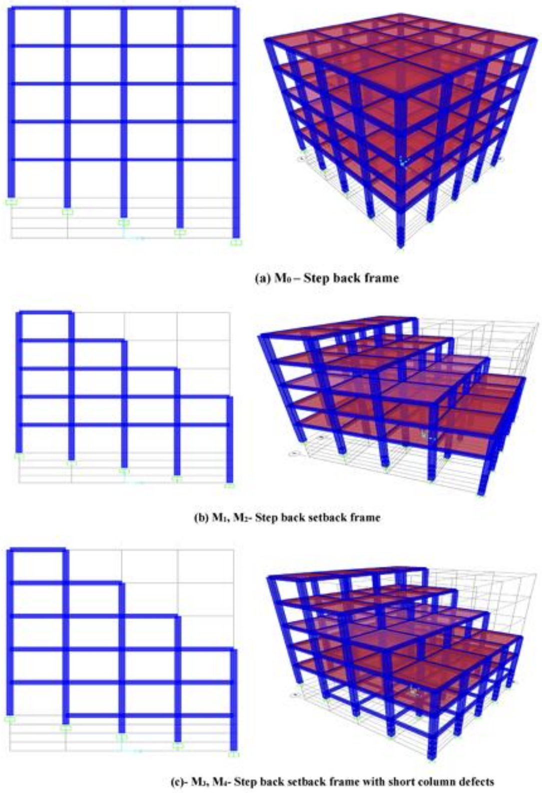


Figure 2.21: Models considered [15]

Regardless of the damage condition, a comparison of the fragility curves shows that step-back setback frames produce a lower probability of damage than other step-back setback frames. Additionally, it is determined that setback frames are more prone to damage at low PGA values than step-back setback frames. When mass eccentricity is introduced, it has detrimental effects, and when torsional and short column effects are combined, the damage to buildings on slope is increased. It is imperative that configurations with short column faults be avoided.

Menon et al. (2025) represented a significant advancement in the understanding of seismic risk associated with RC buildings constructed on sloping terrains. In areas like the Indian Himalayas, where building on steep slopes is common and earthquake risks are significant, this study is especially pertinent. Four common building types—flatland, split-foundation, step-back, and step-back setback configurations—are assessed for seismic vulnerabilities using a probabilistic seismic risk assessment methodology that includes nonlinear time history analyses, fragility curves, collapse margin ratios, and seismic drift hazard curves. The numerical models are rigorously designed in accordance with Indian codes (BIS 1893-1:2016, BIS 13920:2016, and NBC 2016) for Zone V, and use fibre-based modelling with SeismoStruct to capture nonlinear dynamic behavior. Ground motions used for dynamic analysis are carefully selected and scaled from the PEER NGA West2 database to match the Indian design spectrum, and the study follows validated modelling strategies with comparisons to experimental data.

The thorough examination of failure mechanisms and dynamic response characteristics in this work is one of its strongest points. In particular, it identifies torsional coupling and premature shear failure in short columns at the upper street level, which are found to have a significant impact on SB and SF building types. It's interesting to note that the SBSB arrangement effectively mitigates both plan and vertical flaws, showing superior seismic resilience with no storeys at risk from design-level shaking. The authors evaluate how well the current geometric and dynamic seismic irregularity descriptors predict seismic risk. The results indicate that, particularly at lower damage states, a large number of commonly used descriptors exhibit statistical or logical association with seismic risk. This demonstrates the urgent need for creating new descriptors specifically suited to these configurations and supports the claim that the code-based irregularity metrics currently in use are insufficient for paired abnormalities found in sloping ground buildings.

Furthermore, the computed CMR values for every typology show adherence to global safety standards (such as FEMA P-795), suggesting that, as long as the design assumptions are strictly adhered to, the current Indian seismic codes are suitable for mid-rise sloping ground buildings intended as special moment-resisting frames.

Sil et al. (2025) presents a scientifically robust and practically significant study aimed at improving seismic safety in hill slope construction. Step-back and step-back with setback are two popular building layouts. The authors compare their seismic performance under different slope circumstances and story levels in these four- and five-story reinforced concrete buildings. The study investigates how topography and structural design affect seismic response parameters like base shear, spectral displacement, and damage probability using sophisticated numerical modelling via ETABS and the response spectrum method, nonlinear static pushover analysis, and time history analysis.

The study's use of fragility analysis, which measures the likelihood of surpassing damage thresholds for buildings on slopes between 0° and 30°, is one of its main strengths. The research's probabilistic risk paradigm is strengthened by the use of fragility curves created using lognormal distributions and dynamic inputs from previous ground motion data. According to the study, structures with step-back and setback layouts perform better overall, exhibiting reduced displacement and base shear at all slope angles. For instance, a five-story step-back building at 0° had a base shear of 540 kN, whereas the step-back and setback configuration had an equivalent of 380 kN, a reduction of around 30%. Comparable patterns were noted in the results of fragility and displacement, indicating that setback elements effectively distribute lateral loads and reduce torsional irregularities, enhancing seismic resilience.

The data further supports the finding that higher seismic vulnerability is associated with steeper slopes, as fragility curves demonstrate a dramatic increase in damage probability from minor to total damage as slope angles increase. For areas in seismic Zone V, like the Indian Himalayas, where haphazard or uncontrolled slope development is still common, this is crucial. Real seismic records (such as those from the Imperial Valley, Kobe, and Northridge) and suitable code-based assumptions that are in line with IS 1893:2016 for Indian seismic design further support the study. The study has many drawbacks in spite of its advantages. Although the nonlinear static pushover method is helpful for calculating force distribution and target displacement, it ignores complex dynamic phenomena such cyclic degradation or higher-mode interactions, which might be important in tall or irregular buildings. Likewise, the response spectrum approach lacks the realism of nonlinear time history simulations for extreme occurrences and assumes elastic behavior. In order to overcome these issues, the authors propose integrating nonlinear dynamic analysis and performance-based design in the future.

Table 2.3: Building configuration considered [17]

Building type	Number of stories	Slope angle (degrees)	Configuration
Four story step-back	4	0° 7.5° 15° 22.5° 30°	Step back
Five story step-back	5	0° 7.5° 15° 22.5° 30°	
Four story step-back and setback	4	0° 7.5° 15° 22.5° 30°	Step-back and setback
Five story step-back and setback	5	0° 7.5° 15° 22.5° 30°	

To sum up, this work significantly advances the fields of slope construction safety and seismic engineering. In addition to providing engineers and urban planners with a workable design approach for seismically active hill locations, it provides unambiguous, data-supported evidence supporting step-back and setback configurations above traditional step-back designs. It is an invaluable resource for scholars and practitioners because to its methodological clarity, usage of fragility curves, and adherence to seismic regulations. Buildings with a step-back and set-back design outperformed those with just a step-back design in terms of seismic performance, according to the comparison. For instance, at a 0° slope, the base shear for the step-back and setback configuration was 280 kN for 4-story buildings and 380 kN for 5-story buildings, lower than the 460 kN and 540 kN, respectively, for step-back configurations. Similarly, story displacement was reduced in step-back and set back configurations across all slope angles.

Saha et al. (2021) presents a comprehensive and scientifically robust investigation into the seismic behavior of RC buildings constructed on sloping terrains, with a particular focus on the role and placement of open stories, a common architectural feature in hill regions used for parking or access at road levels. Using nonlinear time history analysis in SAP2000, the study examines two commonly seen building configurations in hilly regions: step-back and split-foundation, across three distinct story ratios (0.5, 1.0, and 2.0), taking into account nonlinear behavior, torsional irregularities, and infill wall effects. A suite of 22 recorded ground motions, scaled in accordance with IS 1893 standards, is used to apply bi-directional seismic loading to 24 structure models. To comprehend the seismic demand and structural susceptibility, key engineering demand parameters are assessed, including peak inter-story drift ratio (IDR), peak floor acceleration, peak roof displacement, and story shear.

By creating Complementary Cumulative Distribution Functions for various EDPs, this work's probabilistic fragility evaluation provides a clear knowledge of the risk of damage across a range of seismic intensities, which is one of its main strengths. The results show that there is torsional coupling in both SB and SF buildings, particularly when an open story is positioned at the topmost foundation level (UFL). When compared to structures without an open story, this arrangement increases IDR at the UFL by three to five times, and in low-rise layouts, it amplifies roof displacement by more than fifty percent. The likelihood of breaching damage thresholds is significantly increased by these open stories; in certain configurations, the possibility of surpassing the allowable 0.4% IDR limit under design-level shaking is about 100%. Furthermore, performance is adversely affected by the existence of an open tale at the lowermost foundation level (LFL), although this effect is not as strong as that of UFL placement. Crucially, UFL's open-story structures exhibit lower story shear and floor acceleration above the open story. However, this is because of their greater flexibility, which increases torsional vulnerability and displacement demands. Because of their intrinsic stiffness anomalies, split-foundation buildings typically perform worse than step-back buildings, particularly at lower story ratios (≤1.0), where torsional behavior is most pronounced. Additionally, the study reveals that story shear concentrates right above the UFL. Removing infill at the UFL lowers this shear demand because of less inertia, but it also causes a significant increase in displacement and drift. Potential non-structural failure zones are indicated by the study's findings that peak floor accelerations (PFA), which are crucial for the safety of non-structural components, are maximum when there are no open stories present and that their magnitudes fall above an open story at UFL while increasing below it.

P-delta effects, material nonlinearity, cracked section characteristics, and short-column failure modes are all included in the paper's extensive modelling methodology, which helps to simulate the behavior of hilly buildings in a realistic way. In addition to being well-supported, the findings are also very relevant to structural engineers, urban planners, and legislators who work to reduce seismic risk in hill towns. When such architectural features are inevitable, the study strongly advises against providing open stories at the uppermost foundation level and instead suggests careful seismic design consulting.

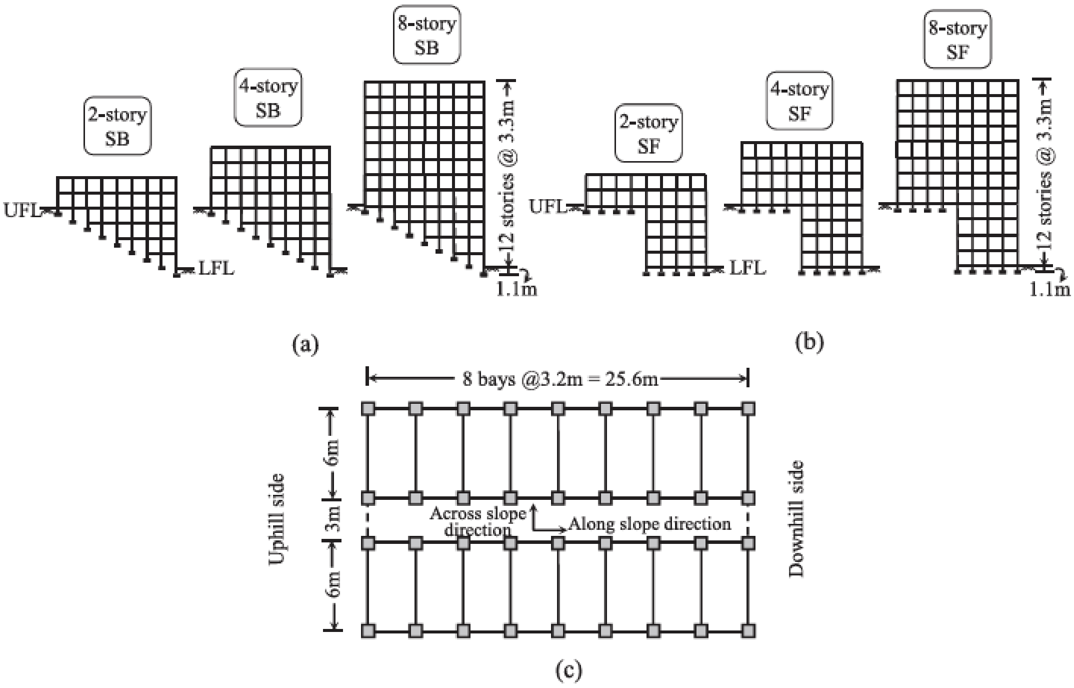


Figure 2.22: (a) Elevation of 2-, 4-, 8-story SB building models, (b) elevation of 2-, 4-, 8-story SF building models, and (c) generic plan of the buildings [18]

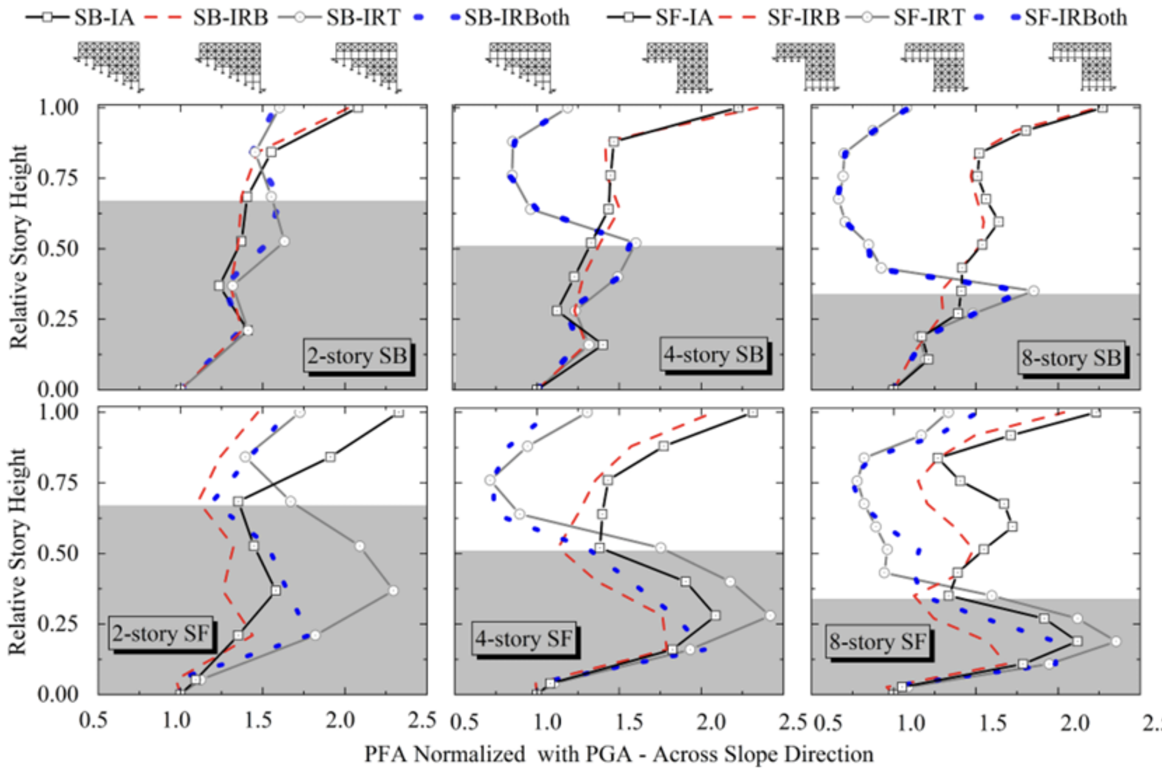


Figure 2.23: Variation of the median of PFA normalised with PGA [18]

Bhattacharya et al. (2025) The study discusses important drawbacks of traditional Rapid Visual Screening (RVS) methods, which usually only take anomalies into account when determining their severity. The authors suggest an improved scoring system that determines fundamental seismic vulnerability scores and score modifiers according to the kind and extent of structural irregularity, utilizing the OSHPD-HAZUS methodology and FEMA P-155 criteria. The study evaluates a variety of regular and irregular reinforced concrete (RC) buildings with characteristics such overhangs, beam eccentricities, floating columns, diaphragm discontinuities, and short-column effects using nonlinear pushover analysis in SAP2000. This greatly improves the objectivity and dependability of RVS evaluations by enabling the accurate, data-driven classification of buildings into vulnerability classes (A to D).

The study's main conclusions are enlightening and useful. First, a typical five-story reinforced concrete building on medium soil in seismic Zone V was found to have a basic score of 2.76. Because masonry infills provide more lateral resistance, buildings with these types of walls performed better, raising basic scores to 3.22 to 3.50. The lowest ratings, as low as 0.94, indicated extreme vulnerability for buildings with floating columns. This was ascribed to the creation of plastic hinges at beam-column joints, concentrated point loads, and redistribution of shear forces—all of which jeopardize structural stability. In certain models, the short column effect—particularly because of partial wall heights—also resulted in collapse circumstances, with scores falling below 1.0, signifying total structural failure.

Additionally, it has been demonstrated that buildings with beam eccentricities develop hinge forms and stress concentrations at misaligned beam connections, gradually decreasing their seismic scores as their eccentricity increases (for example, from 1.50 to 1.45 for 5-story structures with 1.0–1.4 m eccentricity). Stability was also severely affected by the overhangs; structures with overhang area ratios higher than 20% had modifiers of -0.73, which brought total scores down to catastrophic levels. Diaphragm discontinuities indicate decreased stiffness and increased vulnerability to dynamic stresses, even if they produce smaller score drops, increased roof displacements, and longer natural periods.

Additionally, the study created classification tables that connected susceptibility classifications (A–D) to quantitative ratings for every irregularity. For instance, structures with partial infill walls, re-entrant corners, or overhang ratios greater than 20% were consistently classified as Class D, suggesting a significant danger of collapse. The study's new scoring approach, which gives improved resolution and the ability to prioritize retrofitting operations based on severity rather than presence alone, validates well with FEMA and Indian RVS procedures.

Ferraioli et al. (2024) presents a comprehensive and technically robust methodology for improving the seismic performance of plan-asymmetric reinforced concrete (RC) buildings, with a particular focus on mitigating torsional effects a frequently underestimated but critical aspect of seismic behavior in irregular structures. The authors criticize the shortcomings of displacement-based design and classic nonlinear static procedures, which frequently use simplified single-degree-of-freedom approximations, overlook higher mode effects, or assume uniform lateral load distributions. The study proposes a "two-step" seismic retrofit approach to address these issues: buckling restrained braces with hysteretic steel dampers are used in the second step to improve energy dissipation and regulate displacements, while non-dissipative elastic steel braces are inserted in the first step to balance lateral stiffness and lessen torsional irregularities.

The benchmark structure is a real-world school gymnasium building in Vibo Valentia, Italy, which has notable torsional anomalies and was originally only intended for vertical loads. Fiber-based modelling in SAP2000 is used to do detailed nonlinear analyses, such as pushover analysis and nonlinear time-history simulations. The study's conclusions are both practically significant and rigorously quantitative. Significant torsional mitigation was shown by the first mode's torsional modal mass ratio around the Z-axis, which decreased from 0.278 (as-built) to 0.007 following Step 1 of the retrofit. Peak inter-story drift ratios (IDRs) of 1.22% in the X-direction and 1.62% in the Y-direction were attained by the retrofitted structure with HBF dampers in Step 2, remaining well below the 2% collapse prevention criterion. The nonlinear time-history analyses, performed under seven different spectrum-compatible ground motions, confirmed that all HBF dampers yielded, validating their role in dissipating seismic energy.

Additional findings demonstrated that, under Life Safety and Collapse Prevention performance levels, chord rotation demands for beams and columns stayed below permissible capacity limits. Following the refit, there was also a notable improvement in the safety index (ζ) for critical limit states, which is the ratio of capacity to demand in terms of peak ground acceleration and return period. For example, the safety index under Life Safety conditions rose from 0.093 (as-built) to levels showing post-retrofit compliance with seismic code standards. The retrofitted structure shows improved global stiffness and strength, as well as a greatly reduced torsional coupling and more uniform lateral demand distribution, as confirmed by the capacity spectra, bilinear idealizations, and modal characteristics.

Wang et al. (2025) represents a comprehensive and high-resolution study on how orthogonal seismic combination rules, particularly the 100-30% rule impact the collapse performance and force demand of torsionally irregular structures. Inconsistencies between international codes are addressed in the study, specifically between ASCE 7-22, which requires bidirectional combination for TI buildings, and ASCE 7-16, which does not. Three Special Concentric Braced Frame buildings of different heights (4, 9, and 20 stories) are modelled with three design variants: regular (RP), TI without combination rules, and TI with the 100-30% rule (TI-100-30%). The authors use Incremental Dynamic Analysis (IDA) to perform a probabilistic seismic performance assessment in order to settle this dispute. With realistic assumptions, such as material nonlinearities, gusset plate behavior, brace buckling, and out-of-plane defects, the study is based on nonlinear response-history simulations conducted with a robust modelling framework in OpenSees. 22 pairs of ground motions from FEMA P-695 are used in 990 nonlinear analyses per building case. Rp, or the ratio of real bidirectional force demand to predicted combined demand based on unidirectional inputs, is the primary statistic used to assess the suitability of the 100-30% rule. The results show that, even when there is considerable torsional irregularity, the 100-30% rule overestimates the force needs for both X-axis and Z-axis braces since Rp values are constantly less than 1.0. It was discovered that the likelihood of Rp $\leq$ 1.0 was higher than 84%.

For every design scenario, 50-year collapse risks and collapse fragility curves were also created. According to the findings, TI buildings that do not follow the 100-30% guideline are more likely to collapse than both ordinary and TI-100-30% types. For the 4-, 9-, and 20-story structures, the rule's implementation specifically decreased the danger of collapse by 35%, 10%, and 25%, respectively. Interestingly, TI-100-30% buildings outperformed their standard counterparts, indicating that the guideline encourages the use of fundamentally conservative designs. Additionally, the study demonstrated that the 100-30% rule improved performance under strong seismic stress by increasing brace sizes and decreasing peak story drift ratios.

Menon et al. (2020) represents a scientifically rigorous and timely investigation into the effectiveness of various torsional irregularity indices for predicting seismic demands in reinforced concrete moment-resisting frame buildings. The paper criticizes the extensive use of scalar descriptors, many of which are supported by international seismic codes including IS

1893, Eurocode 8, and ASCE 7, such as normalized static eccentricity, torsional radius to mass radius of gyration ratio, and displacement ratios ($\Delta_{max}/\Delta_{min}$). It makes the case that these descriptors, particularly when applied singly, are ineffective for reliably forecasting seismic response at various ground motion intensity levels, especially in the inelastic region.

Thirteen three-story RC frame buildings with controlled variations in stiffness and strength eccentricities are subjected to a thorough nonlinear seismic examination by the authors. The study links inter-storey drift needs with distinct torsional irregularity indices under a suite of 20 bidirectional ground motions, scaled to different intensity levels (S_a_avg ranging from 0.025g to 0.15g), using statistical regression techniques and Multiple Stripe Analysis. The main conclusion is that drift needs in both elastic and inelastic phases cannot be accurately captured by a single scalar indicator. For instance, indices of torsional flexibility (r_k/r_m) and stiffness eccentricity (e_k/B) exhibit a moderate relationship with inter-storey drift needs at low seismic intensities. These become less important, though, as structures start to behave nonlinearly and strength eccentricity (e_v/B) takes over as the primary indicator of seismic demand.

To forecast maximum inter-storey drift, a mathematical demand model that incorporates this vector (based on a power-law relationship) is developed. The suggested model regularly produces high R_2 values, surpassing conventional single-index descriptors, according to regression analysis. Additionally, the model shows that the influence of strength eccentricity rises nonlinearly at high shaking intensities, indicating that e_v/B management is essential for reducing inelastic torsional response. This realization is essential because strength eccentricity is rarely explicitly limited by current seismic codes. According to the authors, their model may be used as the foundation for fragility functions that are sensitive to irregularities, improving the ability to distinguish between systems that are torsionally balanced and those that are not.

Jarapala et al. (2025) offers a substantial and scientifically grounded contribution to the seismic evaluation of reinforced concrete buildings constructed on hill slopes. The insufficiency of current Seismic Vulnerability Descriptors and Seismic Irregularity Descriptors when applied to buildings with coupled abnormalities, such as those found on sloping terrains, is a key gap in seismic design and assessment frameworks that the authors fill. The intricate interplay of torsional, vertical, and plan-asymmetric irregularities that emerge in hill slope buildings is not adequately captured by the descriptors that are now in use. These descriptors are mostly based on geometry, dynamic properties, or a mix of these factors and were primarily established for flatland structures. Utilizing a comprehensive collection of nonlinear time history analyses on four representative hill-slope building typologies—flatland, split-foundation, step-back, and step-back-setback—a novel SID that more accurately correlates with seismic risk across a wide range of damage thresholds was developed and statistically validated. Using 3D fibre-based models in SeismoStruct, the study incorporates bi-directional seismic input, P-delta effects, and nonlinear material behavior. It is methodologically rigorous. Mean annual exceedance probabilities, seismic fragility functions, and probabilistic seismic demand models are used to quantify seismic risk. Statistical correlation analysis is performed on 24 descriptors currently in use.

Key findings of the study show that step-back-setback structures have the best seismic resistance, frequently surpassing even standard flatland designs, while split-foundation buildings are consistently the most vulnerable across damage states. Although the step-back layout is generally safer, short-column effects cause premature shear failure at the upper street level, which needs to be taken into account during design. Crucially, the statistical correlation research shows that a number of academic SIDs and SVDs, as well as existing code-based SIDs (such those from BIS 1893 and FEMA), do not significantly correlate with the observed seismic risk across different damage thresholds. In contrast, the proposed descriptor—grounded in fundamental dynamic properties—demonstrates strong and consistent statistical correlation across all thresholds (including Immediate Occupancy, Life Safety, and Collapse Prevention), making it a robust tool for seismic risk estimation.

The study also sheds important light on the dynamic behavior of hill-slope buildings, emphasizing how vertical irregularities drastically change drift profiles and damage mechanisms and how torsional and plan asymmetries are amplified at particular locations (especially close to the upper street level). The analysis highlights how inadequate 2D modeling is for these structures and promotes thorough 3D evaluation in both practice and research.

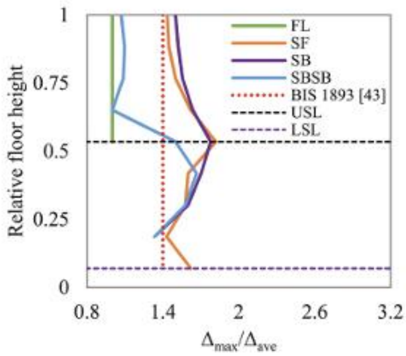


Figure 2.24: Torsion irregularity [22]

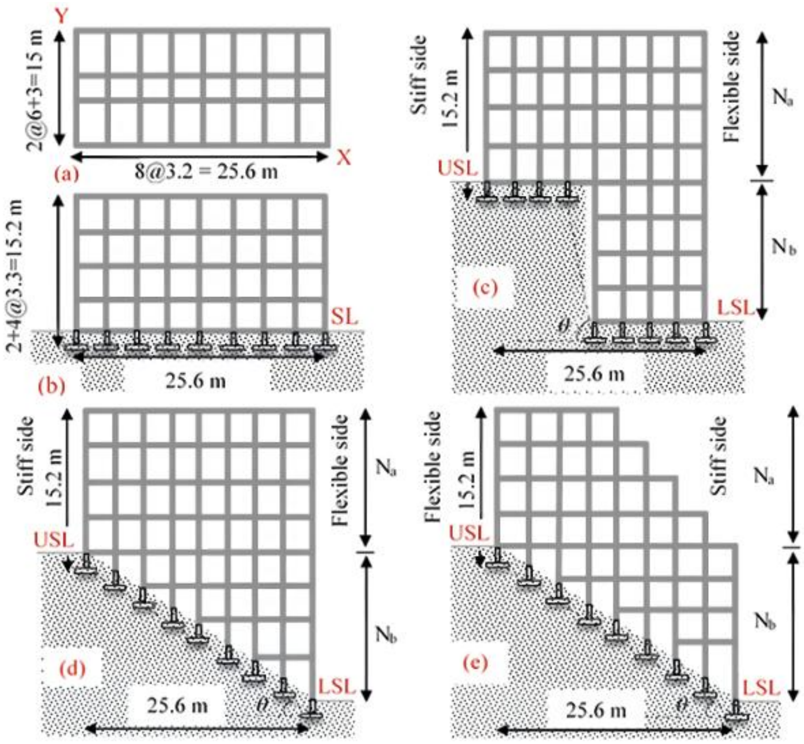


Figure 2.25: Plan and elevation of building considered [22]

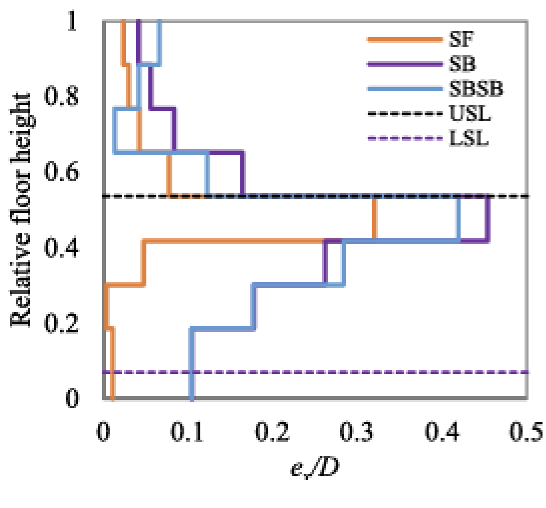


Figure 2.26: Normalised floor eccentricity [22]

Arif et al. (2017) presents a parametric and comparative analysis of the seismic performance of two common hill building configurations: step-back and step-back setback. This paper tackles a critical problem in seismic engineering, particularly in hilly areas where structures are built on steep slopes and are vulnerable to complicated structural behavior because of irregularities in geometry, different column heights, and an uneven distribution of mass and stiffness.

Three-dimensional modelling in ETABS v9.0 is used in the study, and 18 building models are analysed using reaction spectrum analysis (RSA) and the corresponding static approach. The study investigates the effects of changing plan dimensions, particularly length along and across the slope, on dynamic parameters such base shear, storey drift, top storey displacement, fundamental time period, and storey shear. The results show that the empirical time period calculations from the IS code frequently differ from those obtained via modal analysis, particularly for irregular hill buildings—highlighting a significant limitation in the current code-based seismic assessment. All models were created in accordance with IS 1893:2002 for seismic Zone V.

The main conclusions show that, in practically every assessed parameter, step-back setback buildings outperform step-back structures in terms of seismic performance. Step-back setback models consistently showed decreased top story displacement and shear forces in structures with longer bays along the slope. In particular, storey drift reductions ranged from 10 to 50% in both slope directions, while base shear reductions reached up to 45%. Furthermore, it was found that foundation shear was more evenly distributed in step-back setback configurations but concentrated in the lower frames of step-back structures. Step-back buildings once again shown increased seismic demands when models were adjusted over the slope (with fixed height), especially at mid-height levels, indicating susceptibility to torsional behavior and stress concentration.

Crucially, the study finds that because step-back setback layouts have less torsional irregularity and less seismic weight, they are structurally more robust under seismic pressures. For such irregular geometries, the authors highly advise employing reaction spectrum analysis instead of the comparable static method because the latter is unable to fully capture the dynamic behavior of these structures. Because the step-back setback configurations have a lower seismic weight than step-back buildings, they encounter fewer torsional moments and seismic forces. When compared to step-back layouts, step-back setback buildings exhibit a base shear value reduction of about 45%. Additionally, step-back buildings exhibit greater storey shear and drift, increasing their susceptibility to seismic pressures.

Surana et al. (2022) addresses a critical gap in current seismic design practices by quantifying how inherent torsional irregularities in buildings, particularly step-back configurations typical in hilly regions, amplify acceleration demands on NSCs. Under a suite of ground movements from FEMA P695, the study applies bi-directional linear dynamic time-history analysis to three prototypical RC step-back buildings (2-, 4-, and 8-story) that were modelled using ETABS and OpenSees. The research focuses on a practically relevant issue: many real-world buildings, particularly those in mountainous terrain, stray from the regularity prescribed by the code. The study emphasizes the vulnerability of acceleration-sensitive NSCs to torsional effects, which are frequently essential to developing functionality. The 2:1 H: V slope is considered.

A thorough computational method that includes dynamic analysis, 3D modelling, and various NSC damping and tuning ratios. thorough analysis of torsional irregularity indices ($\Delta_{max}/\Delta_{min}$ and $\Delta_{max}/\Delta_{avg}$) based on ASCE 7 and IS 1893, offering insights that are in line with the code. shows that in certain floors, peak torsional amplification factors (TAFs) can be greater than 9.0, which means that if torsion is disregarded, NSC needs may be underestimated by more than 800%. emphasized how NSCs are particularly vulnerable to torsional amplification when they have poor damping and specific tuning ratios. Less torsional amplification is seen in very flexible NSCs ($T_s/T_1 \geq 1.5$), indicating easier treatment in these situations.

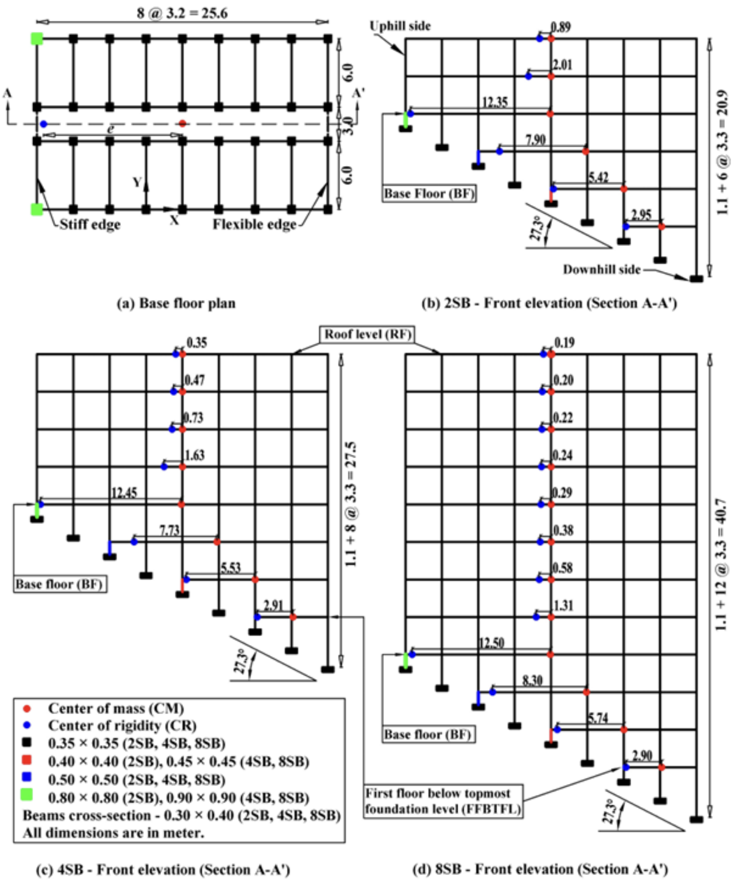


Figure 2.27: Plan and elevation of buildings considered [24]

The analysis ignores any dynamic interaction with the structure since it believes that NSCs have very little mass. This restricts its applicability to heavy equipment or tanks, but it is acceptable for light NSCs. Only linear or elastic behavior is taken into account. Although appropriate for NSCs, results could be changed by nonlinear torsional effects in the primary structure.

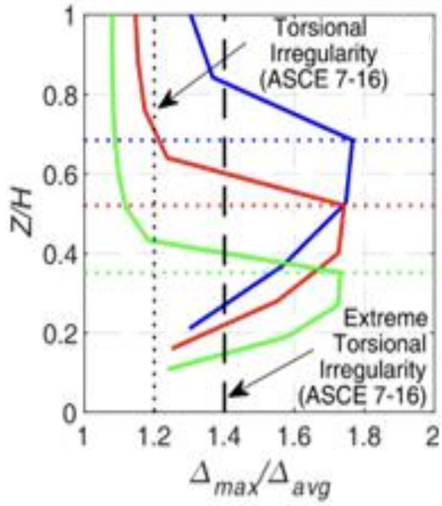


Figure 2.28: Torsion irregularity indices [24]

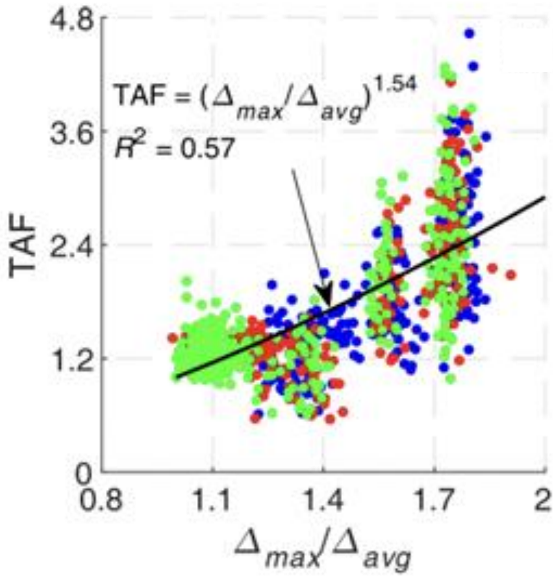


Figure 2.29: Torsional amplification factor for acceleration sensitive NSC's [24]

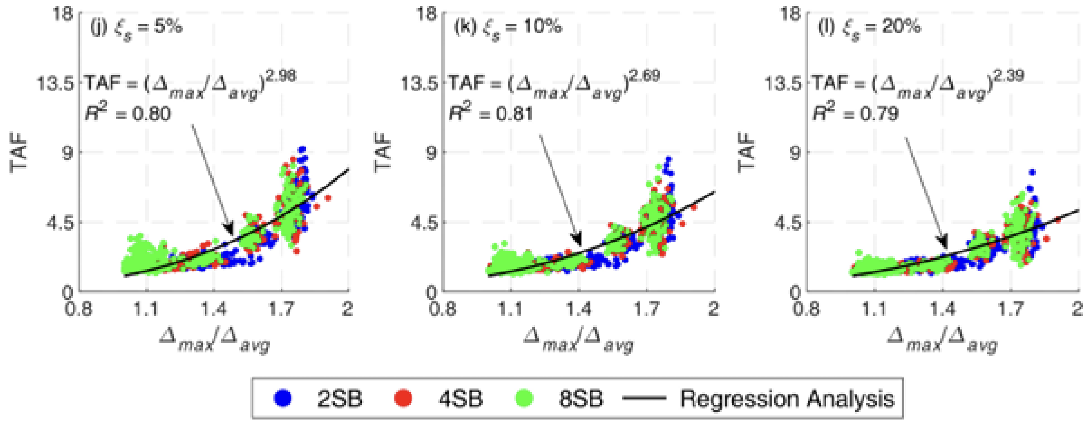


Figure 2.30: Correlation of torsional amplification factor with torsional irregularity indices [24]

Huang et al. (1997) presents a rigorous analytical investigation into how torsional irregularities in buildings affect the seismic behavior of light, eccentrically mounted equipment. The authors create a two-degree-of-freedom per floor model that accounts for both translational and rotational dynamics, a significant breakthrough for researching multi-story torsional structures, after realizing that earlier research had mostly ignored torsional effects. They estimate peak equipment responses using modal synthesis (more precisely, the Complete Quadratic Combination method) and provide closed-form solutions for the coupled system's modal characteristics by using perturbation techniques. Their results demonstrate that equipment is subjected to noticeably greater accelerations when tuned to the building's flexural modes—particularly those with large modal participation factors—as opposed to torsional modes. Additionally, the study discovers that when the mass of the equipment decreases (i.e., "light" equipment), its acceleration first rises before levelling out at a particular point, suggesting that interaction with the structure becomes insignificant. Crucially, the authors use nodal points in the torsional mode forms to pinpoint places on each level where equipment might be installed to reduce response. They also point out that, depending on location and tuning, the eccentricity of the equipment can have a major impact on both modal and peak responses in torsional buildings, although it has little effect on extremely light equipment on shear buildings. Although linear elastic behavior, classical damping, and light equipment mass are assumed in the study, these simplifications are justifiable and provide analytical clarity.

The paper offers useful design insights, such as when entire dynamic interaction needs to be taken into account and when floor response spectra are adequate. By providing both theoretical depth and useful advice, it significantly closes a crucial gap in seismic design for non-structural components. For engineers and academics working with irregular buildings, particularly in seismic zones, this makes the work extremely important.

2.3 Research Gaps

Based on above literature review following gaps have been identified:

- While numerous studies emphasize the seismic performance of structural components, there is limited research on the detailed modelling and dynamic characterization of non-structural components. The contribution of NSCs to overall building response, particularly in terms of stiffness, damping, and mass distribution, is often simplified or neglected.
- Existing studies primarily examine regular or uniform building configurations, with minimal exploration of stepback buildings. This limits the understanding of torsional irregularities and seismic demand variations unique to stepback geometries, which are common in hilly terrains and urban landscapes
- Research addressing slope effects on building performance is often restricted to single slope angles, failing to explore how varying slope gradients influence seismic response. A more comprehensive parametric study on slope variations is required to capture realistic site conditions.
- Few studies integrate NSC-specific properties, such as flexibility and anchorage conditions, with global seismic behavior of buildings. This gap hinders accurate prediction of overall performance and potential damage scenarios under earthquake loading.

2.4 Objectives

The objectives of this study are as follows:

- To model and analyze NSCs with realistic stiffness, damping, and mass properties to accurately assess their contribution to the overall dynamic response of buildings.
- To extend the analysis to stepback geometries, which are common in hilly terrains, to understand their unique torsional irregularities and seismic performance challenges.
- To conduct a parametric study by varying slope angles to capture the effect of different site conditions on torsion amplification and overall seismic demand.

- To study acceleration sensitive non-structural components in a building and study the correlation between torsional amplification and torsional irregularity index.
- To propose a simplified yet robust analytical or numerical model that incorporates both building irregularities and NSC effects, and validate it through simulation and literature comparison.

CHAPTER 3

CHARACTERISTICS OF BUILDING

3.1 General

This chapter presents the fundamental inputs and modelling considerations adopted for the seismic analysis of the building. The geometrical configuration and structural properties of the building are first described to establish the basis of the analytical model. The loading details, including gravity and lateral loads, are then outlined in accordance with relevant design provisions. Subsequently, the selection and characteristics of the time history data used for dynamic analysis are explained, highlighting their suitability for representing seismic demand. Finally, the modelling of non-structural components is detailed, with emphasis on their dynamic properties and interaction with the main structure. Together, these aspects form the essential framework for the subsequent analytical and numerical investigations.

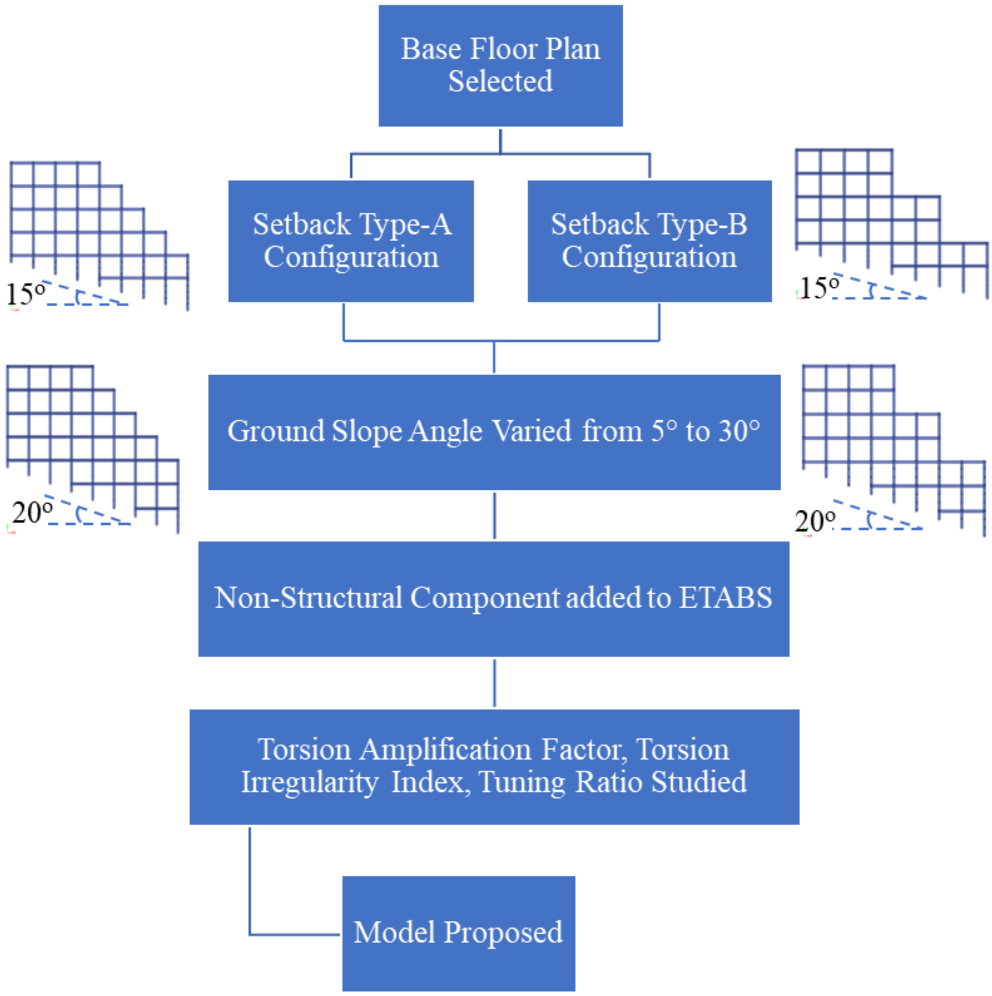


Figure 3.1: Brief methodology

3.2 Geometrical Properties

In this study, torsional amplification in floor acceleration response is examined, with particular focus on the torsional amplification factor at both the flexible and stiff edges, evaluated against floor displacement-based torsional irregularity indices. For this purpose, a set of setback–step-back reinforced concrete frame buildings are analysed. A representative building plan, adapted from earlier research, is adopted (figure 3.2). Two building configurations with different elevations are considered, as illustrated in figure 35 to figure 46. The structures are assumed to rest on sloping ground with inclination angles of 5°, 10°, 15°, 20°, 25°, and 30°.These buildings are named on type and angle of slope they rest on as shown in figures 3.3 and 3.4. These slope variations represent typical site conditions in hilly terrains, where such building forms are commonly constructed to meet functional requirements.

The selection of these models is significant as they exhibit considerable variations in floor eccentricities along their height. Across all cases, the number of stories above the topmost foundation level is kept constant, while the number of stories below it depends on the ground slope. The floor height is maintained at 3.0 m. Column heights from the foundation to the immediately higher floor level vary according to the slope angle. Structural modelling is performed in ETABS, with beams and columns designed using M40-grade concrete, while the floor slabs are modelled as rigid diaphragms. Beam size is kept at 300X400 and column size of 350 X 350 and 500 X 500. Slab of 150mm thickness was modelled.

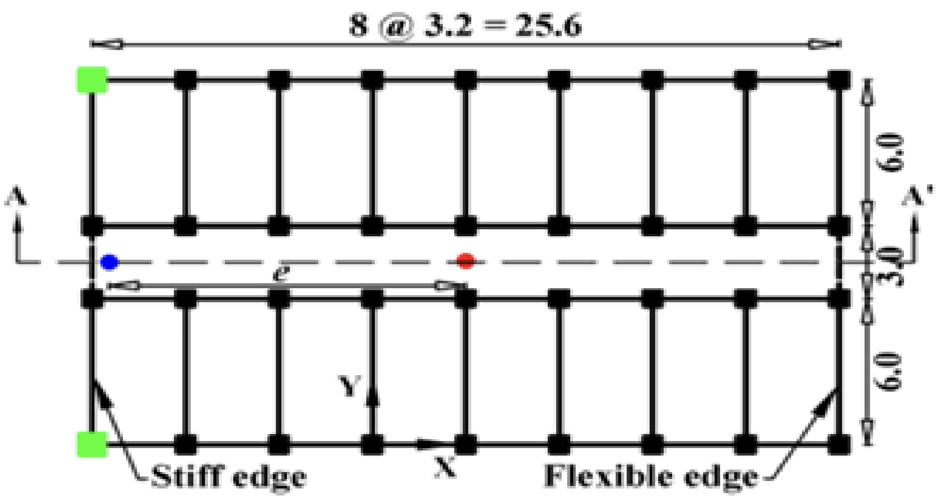


Figure 3.2: Base floor plan of building considered

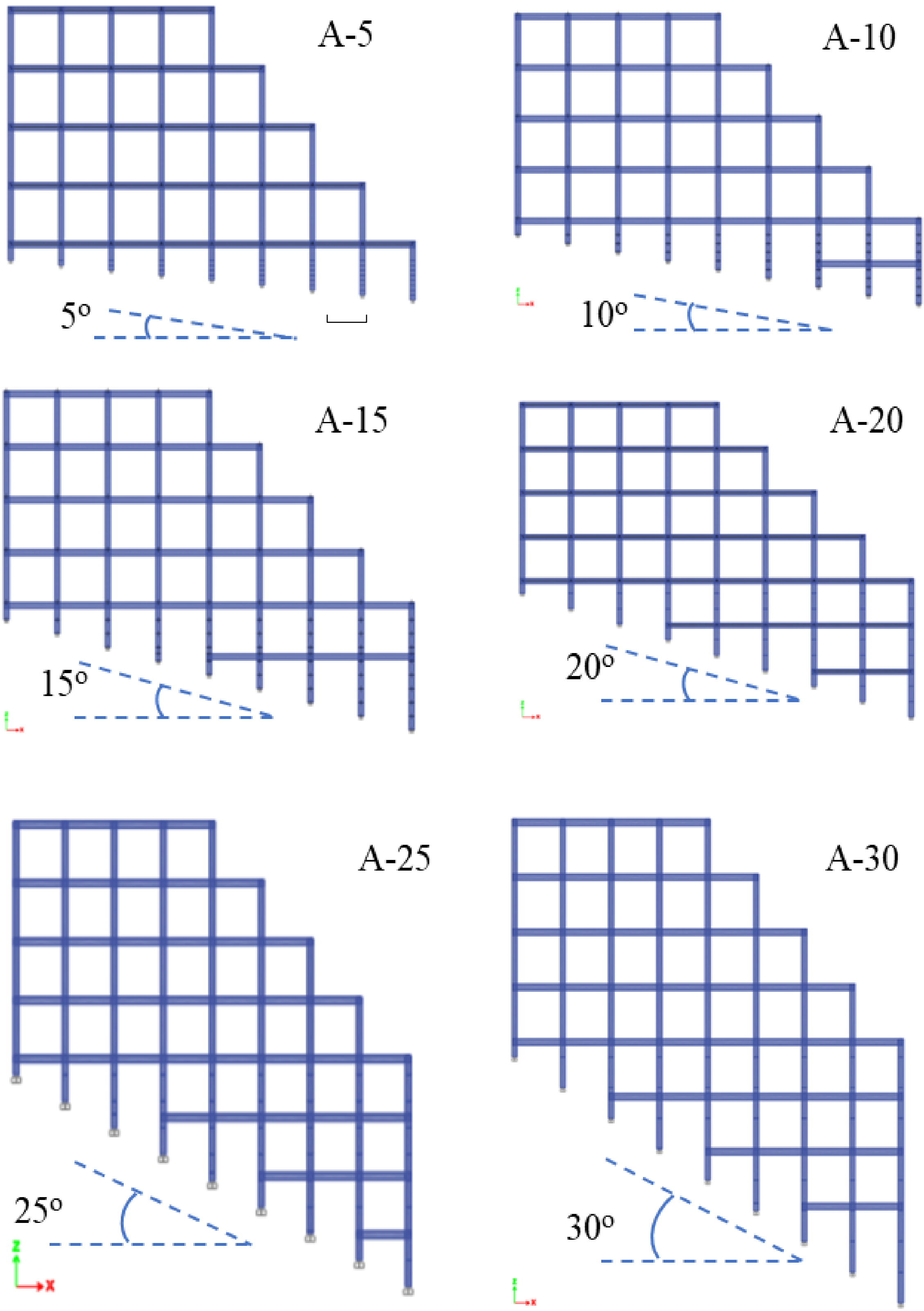


Figure 3.3: Building type-A configuration considered for different ground slopes

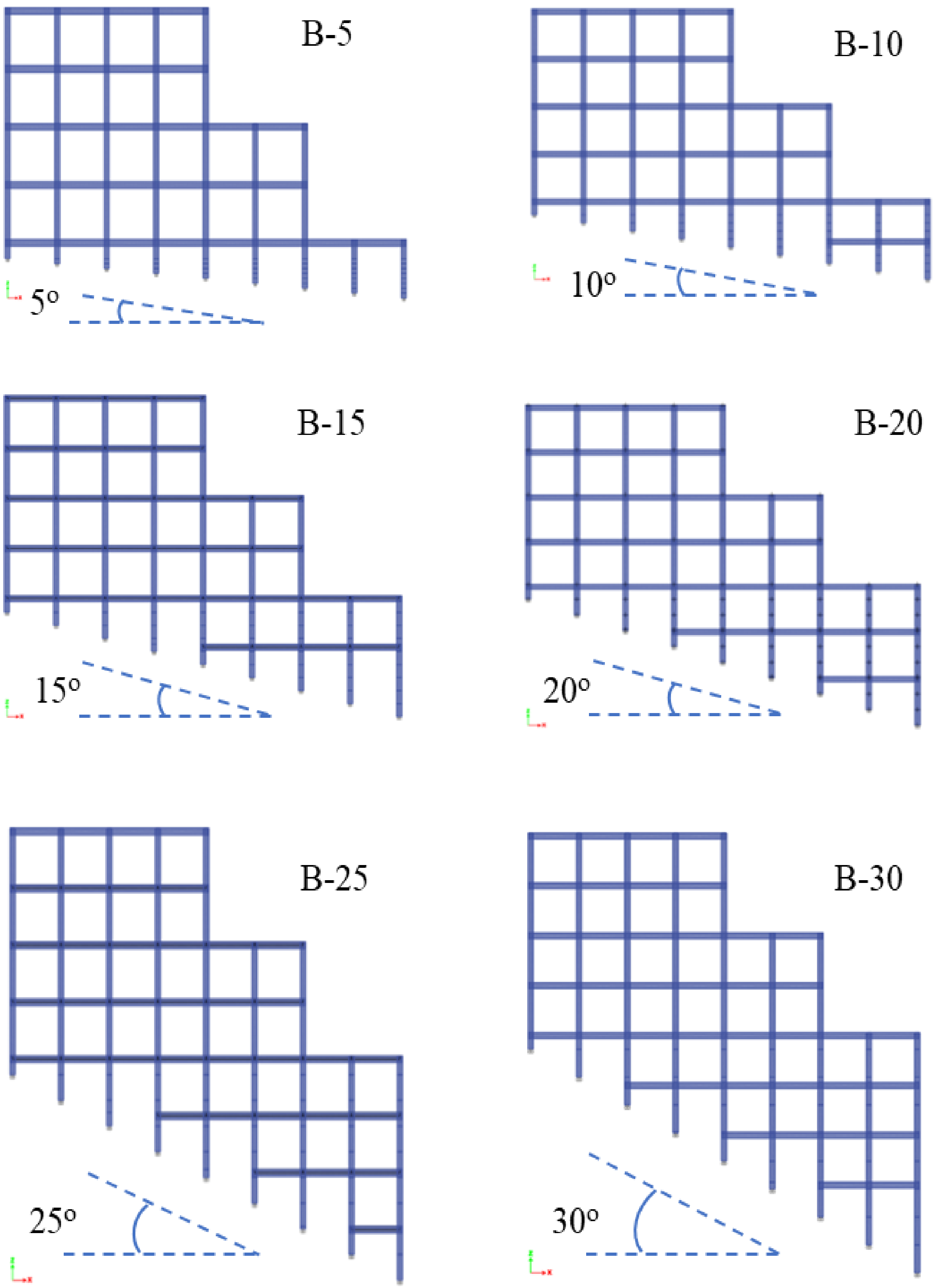


Figure 3.4: Building type-B configuration considered for different ground slopes

3.3 Loading

Dead loads and live loads are applied to the structural models, with superimposed dead loads also taken into account. According to the provisions of Indian Standards, a live load of 3.0 kN/m² is assigned to each floor, while the roof is subjected to a reduced live load of 1.5 kN/m². The buildings under investigation are modelled as Special Moment Resisting Frames (SMRFs) and are assumed to be situated on a rock or stiff soil site, representative of Indian Seismic Zone V conditions. For seismic evaluation, bidirectional linear dynamic time-history analyses are performed.

3.4 Dynamic time history analysis

The ground motion record suite consisted of 15 pairs of recorded seismic ground motions from PEER ground motion database as shown in Table 3.1. The characteristics of the selected suite of ground motion records included in the study had their range between M_w 6.5 to M_w 7.6. Bidirectional linear dynamic time history analyses are performed on the models to evaluate the torsional amplification in the floor response.

Table 3.1: Time history data considered

S.No.	**Earthquake**	**Year**	**Magnitude (M_w)**
1	San Fernado	1971	6.6
2	Friuli Italy	1976	6.5
3	Imperial Valley	1979	6.5
4	Loma Prieta Oakland	1989	6.9
5	Northridge Sylmar	1994	6.7
6	Chi Chi Taiwan	1999	7.6
7	Nenana Mt. Alaska	2002	6.7
8	San Simeon CA	2003	6.6
9	Chuetsu-oki	2007	6.8
10	Iwate	2008	7.2
11	Tottori Japan	2000	7.3
12	Niigata Japan	2004	6.8
13	Darfield, NZ	2010	7.1
14	El Mayor-Cucapah	2010	7.2
15	Taiwan	1999	7.6

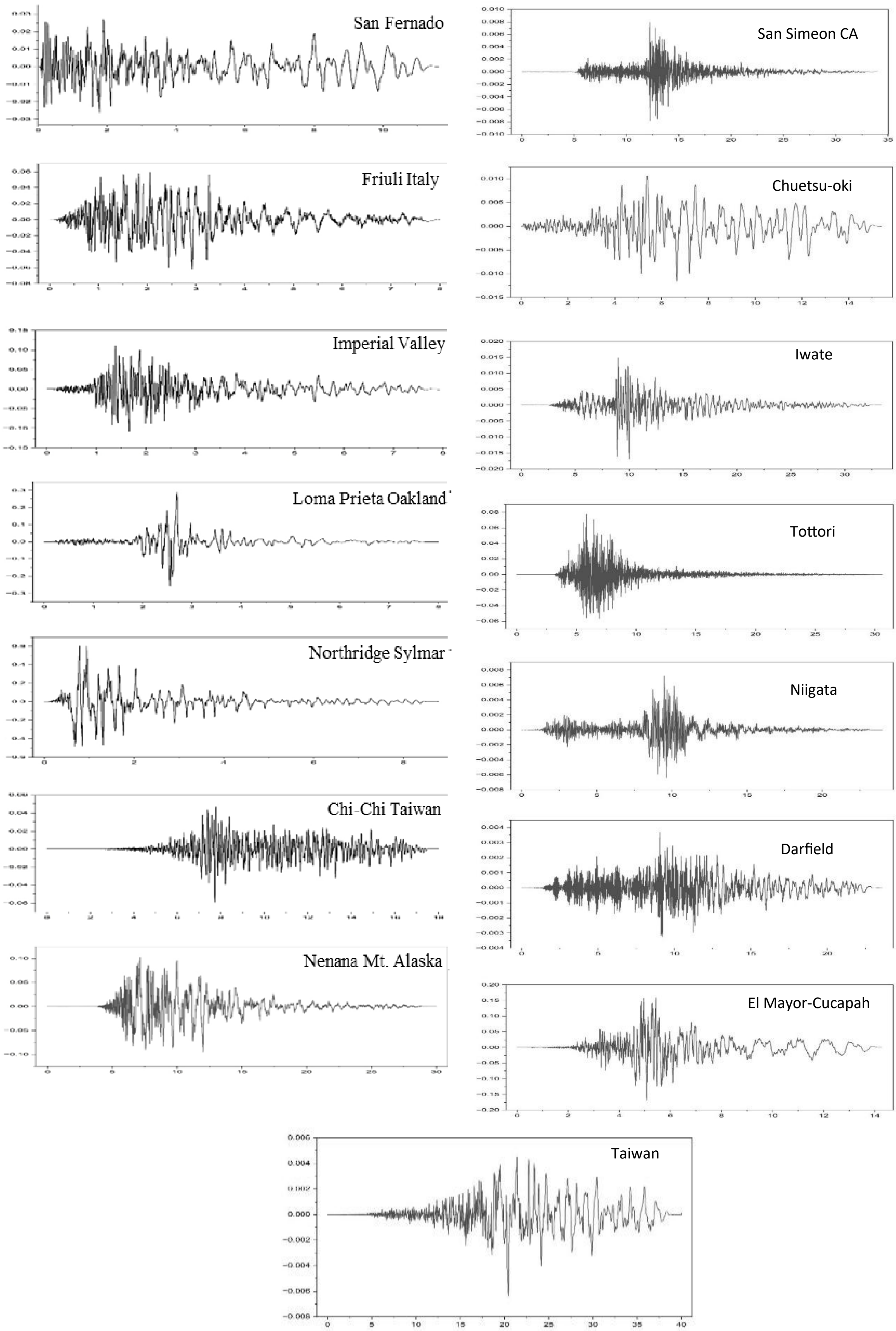


Figure 3.5: Spectral acceleration graph for time history data

Spectral Acceleration graph for the data considered is plotted in figure 3.5.Time history data for the corresponding time history case is imported into the ETABS for the particular direction and is matched to the response spectrum function as shown in figure 3.6 and 3.7

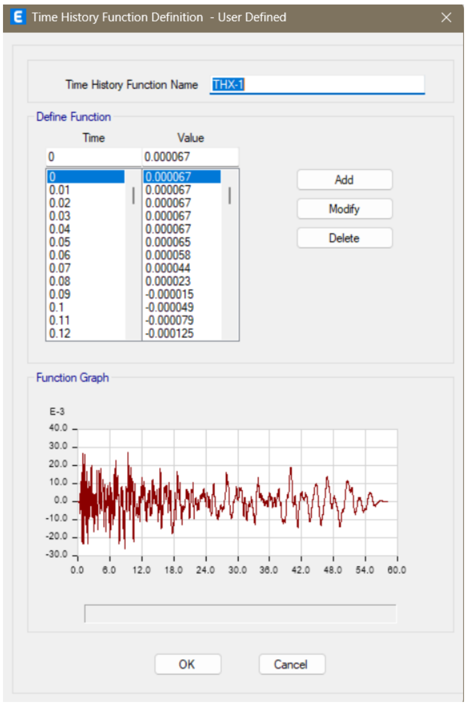


Figure 3.6: Time history data applied in ETABS

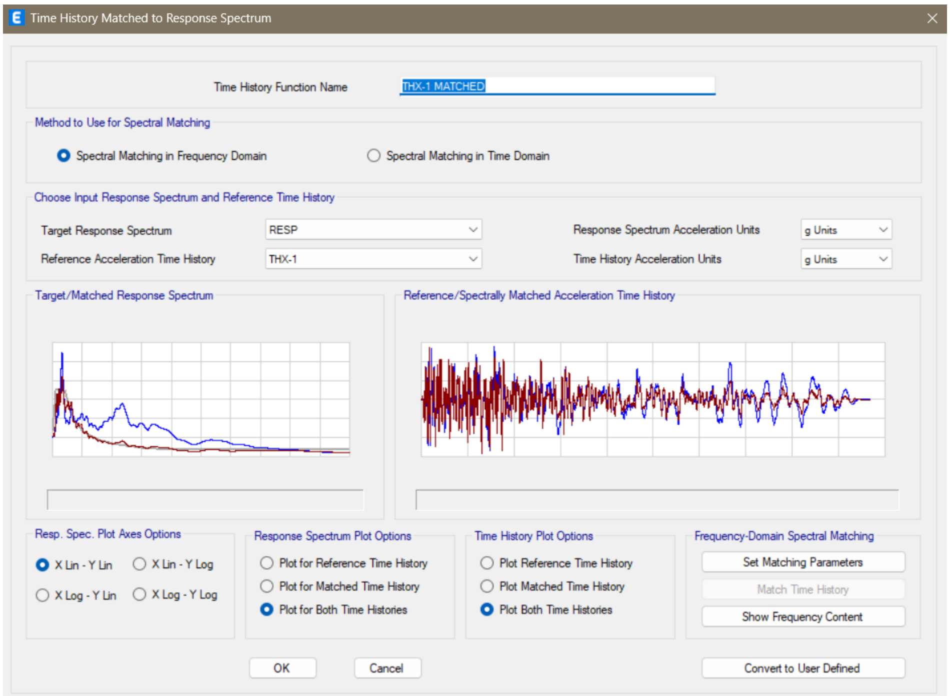


Figure 3.7: Time history data matched to response spectrum

3.5 Eccentricity

Eccentricity in buildings refers to the horizontal offset between the center of mass (CM) and the center of rigidity (CR) of a floor. When these centers do not coincide, seismic forces induce torsional moments in addition to translational motions, leading to uneven distribution of lateral displacements across the structure. This condition is defined as torsional irregularity, which can significantly amplify seismic demand on certain structural elements, increasing the risk of localized damage or failure. Eccentricity for the buildings under consideration is shown in table. The eccentricity comes maximum at base floor level of each configuration and reduces as we go above as well as below it. The eccentricity reduces at roof level in each configuration as we increase the slope angle, while it increases at base floor level as shown in table 3.2 and 3.3. The eccentricity at each level is more in type-A configuration than that in type-B configuration, this could be attributed to reduced floor area in type -B buildings.

Table 3.2: Eccentricity at different floor for buildings with angles 5-15

ANGLE	FLOOR	CONFIGURATION	
		A	B
5	ROOF	2.01	1.23
	THIRD FLOOR	2.13	1.90
	SECOND FLOOR	1.97	1.68
	FIRST FLOOR	1.22	0.91
	BASE FLOOR	4.35	3.80
10	ROOF	1.96	1.17
	THIRD FLOOR	2.00	1.79
	SECOND FLOOR	1.68	1.45
	FIRST FLOOR	0.53	0.45
	BASE FLOOR	6.15	5.59
	FIRST FLOOR BELOW BASE	3.90	3.30
15	ROOF	1.84	1.11
	THIRD FLOOR	1.82	1.68
	SECOND FLOOR	1.40	1.22
	FIRST FLOOR	0.01	0.01
	BASE FLOOR	7.04	6.55
	FIRST FLOOR BELOW BASE	3.31	2.98

Table 3.3: Eccentricity at different floor for buildings with angles 20-30

ANGLE	FLOOR	CONFIGURATION	
		A	B
20	ROOF	1.82	1.06
	THIRD FLOOR	1.75	1.61
	SECOND FLOOR	1.23	1.08
	FIRST FLOOR	0.40	0.25
	BASE FLOOR	7.58	7.02
	FIRST FLOOR BELOW BASE	4.45	3.70
	SECOND FLOOR BELOW BASE	2.80	2.40
25	ROOF	1.72	0.99
	THIRD FLOOR	1.58	1.49
	SECOND FLOOR	0.94	0.84
	FIRST FLOOR	0.96	0.69
	BASE FLOOR	8.23	7.67
	FIRST FLOOR BELOW BASE	4.30	3.60
	SECOND FLOOR BELOW BASE	2.40	2.80
	THIRD FLOOR BELOW BASE	1.90	1.70
30	ROOF	1.71	0.99
	THIRD FLOOR	1.57	1.48
	SECOND FLOOR	0.92	0.82
	FIRST FLOOR	0.99	0.70
	BASE FLOOR	8.27	7.70
	FIRST FLOOR BELOW BASE	4.50	4.00
	SECOND FLOOR BELOW BASE	2.70	2.45
	THIRD FLOOR BELOW BASE	1.40	1.25

3.6 Dynamic behavior and torsional irregularity in buildings

Previous studies have highlighted that the floor response of buildings is strongly influenced by their dynamic characteristics. Hence, modal analysis was carried out using ETABS. The results revealed that the selected setback–step-back buildings exhibit dynamic behavior distinct from that of regular configurations.

Table 3.4: Modal participation mass ratio for A-5

Mode	Period sec	U_X	U_Y	R_X	R_Y
1	0.589	0	0.5918	0.3295	0
2	0.509	0.6699	0	0	0.3283
3	0.421	0	0.1007	0.0009	0
4	0.209	0	0.1132	0.3169	0
5	0.193	0.108	0	0	0.2963
6	0.183	0	0.0001	0.0025	0
7	0.129	0	0.0484	0.0689	0
8	0.119	0.0384	0	0	0.0402
9	0.111	0	8.90E-06	0.0018	0
10	0.096	0	0.0272	0.0565	0

Table 3.5: Modal participation mass ratio for A-10

Mode	Period sec	U_X	U_Y	R_X	R_Y
1	0.664	0	0.5984	0.3452	0
2	0.572	0.6435	0	0	0.3543
3	0.484	0	0.0919	0.0006	0
4	0.242	0	0.1264	0.3071	0
5	0.216	0.113	0	0	0.2447
6	0.205	0	0.0003	0.0041	0
7	0.15	0	0.0636	0.1018	0
8	0.131	0.0503	0	0	0.0455
9	0.123	0	0.0002	0.009	0
10	0.111	0	0.0348	0.0697	0

Table 3.6: Modal participation mass ratio for A-15

Mode	Period sec	U_X	U_Y	R_X	R_Y
1	0.666	0	0.613	0.3504	0
2	0.575	0.6189	0	0	0.3775
3	0.485	0	0.0798	0.0066	0
4	0.232	0	0.1303	0.3064	0
5	0.199	0.1132	0	0	0.2005
6	0.189	0	0.0015	0.0048	0
7	0.148	0	0.0614	0.1114	0
8	0.122	0.0582	0	0	0.045
9	0.118	0	0.0012	0.0213	0
10	0.108	0	0.0223	0.0382	0

Table 3.7: Modal participation mass ratio for A-20

Mode	Period sec	U_X	U_Y	R_X	R_Y
1	0.679	0	0.5928	0.3719	0
2	0.58	0.5731	0	0	0.4105
3	0.512	0	0.0682	0.0074	0
4	0.264	0	0.1311	0.234	0
5	0.219	0.1102	0	0	0.1246
6	0.21	0	0.0014	0.0016	0
7	0.169	0	0.0628	0.0973	0
8	0.134	0.0681	0	0	0.0351
9	0.131	0	0.0031	0.0324	0
10	0.119	0	0.0213	0.0265	0

Since the buildings fall under the category of torsionally irregular structures, a considerable portion of the mass participation ratio is concentrated within the first three modes. This indicates a strong likelihood of torsional irregularity in their seismic response. Furthermore, the modal analysis confirmed that each model shows significant mass participation ratios associated with rotational modes in different directions, as summarized in table 3.4 to 3.15. As the slope angle increases there is increase in participation of rotation across both types of configurations.

Table 3.8: Modal participation mass ratio for A-25

Mode	**Period sec**	**U_X**	**U_Y**	**R_X**	**R_Y**
1	0.692	0	0.6301	0.3424	0
2	0.583	0.5554	0	0	0.4141
3	0.532	0	0.0541	0.0313	0
4	0.29	0	0.1436	0.2825	0
5	0.222	0.134	0	0	0.1144
6	0.215	0	0.0004	0.0014	0
7	0.19	0	0.047	0.1008	0
8	0.152	0.2549	0	0	0.3005
9	0.146	0	0.0002	0.0285	0
10	0.131	0.0013	0	0	0.0363

Table 3.9: Modal participation mass ratio for A-30

Mode	**Period sec**	**U_X**	**U_Y**	**R_X**	**R_Y**
1	0.694	0	0.6016	0.3651	0
2	0.584	0.5153	0	0	0.4443
3	0.534	0	0.0547	0.0285	0
4	0.297	0	0.1671	0.2917	0
5	0.222	0.119	0	0	0.07
6	0.221	0	0.0027	0.0313	0
7	0.201	0	0.046	0.1018	0
8	0.16	0	6.66E-06	0.0259	0
9	0.149	0.3209	0	0	0.4251
10	0.134	0.0008	0	0	0.0346

Table 3.10: Modal participation mass ratio for B-5

Mode	Period sec	U_X	U_Y	R_X	R_Y
1	0.652	0	0.5832	0.3453	0
2	0.573	0.6542	0	0	0.345
3	0.479	0	0.0845	0.0027	0
4	0.244	0	0.1004	0.2356	0
5	0.221	0.1037	0	0	0.2619
6	0.204	0	0.0068	0.033	0
7	0.132	0	0.0778	0.1121	0
8	0.123	0.0447	0	0	0.0525
9	0.117	0	0.0101	0.0324	0
10	0.098	0	0.0235	0.0484	0

Table 3.11: Modal participation mass ratio for B-10

Mode	Period sec	U_X	U_Y	R_X	R_Y
1	0.661	0	0.5822	0.3636	0
2	0.581	0.6262	0	0	0.3717
3	0.493	0	0.0787	0.0001	0
4	0.253	0	0.1146	0.2348	0
5	0.224	0.1065	0	0	0.213
6	0.206	0	0.0053	0.0191	0
7	0.152	0	0.1129	0.1858	0
8	0.125	0.0626	0	0	0.0627
9	0.123	0	1.06E-05	0.0049	0
10	0.103	0	0.018	0.0318	0

Overall, the building type B had higher rotational modes participation as compared to other type of building across all angles. As shown in below tables every building selected had significant rotational mode contribution in first two modes, so the selected set could be referred to be of torsionally irregular buildings.

Table 3.12: Modal participation mass ratio for B-15

Mode	Period sec	U_X	U_Y	R_X	R_Y
1	0.668	0	0.5854	0.3761	0
2	0.586	0.5972	0	0	0.399
3	0.507	0	0.0706	0.0015	0
4	0.263	0	0.1284	0.2424	0
5	0.226	0.1065	0	0	0.1687
6	0.208	0	0.0032	0.0079	0
7	0.172	0	0.0987	0.1718	0
8	0.127	0.0798	0	0	0.0676
9	0.125	0	0.0008	0.0035	0
10	0.107	0	0.0166	0.034	0

Table 3.13: Modal participation mass ratio for B-20.

Mode	Period sec	U_X	U_Y	R_X	R_Y
1	0.673	0	0.5635	0.3965	0
2	0.588	0.5521	0	0	0.4296
3	0.515	0	0.0624	0.0022	0
4	0.272	0	0.1359	0.1931	0
5	0.227	0.1024	0	0	0.1035
6	0.209	0	0.0009	0.0008	0
7	0.183	0	0.0887	0.1278	0
8	0.129	0.0957	0	0	0.0544
9	0.127	0	3.08E-05	0.0114	0
10	0.112	0	0.0244	0.0505	0

Table 3.14: Modal participation mass ratio for B-25

Mode	Period sec	U_X	U_Y	R_X	R_Y
1	0.683	0	0.5943	0.3718	0
2	0.592	0.5337	0	0	0.4322
3	0.534	0	0.0544	0.0178	0
4	0.297	0	0.1657	0.2579	0
5	0.229	0.1224	0	0	0.0919
6	0.22	0	0.0049	0.0245	0
7	0.202	0	0.0477	0.074	0
8	0.152	0.286	0	0	0.319
9	0.144	0	0.0009	0.0434	0
10	0.126	0.0001	0	0	0.0228

Table 3.15: Modal participation mass ratio for B-30

Mode	Period sec	U_X	U_Y	R_X	R_Y
1	0.685	0	0.5643	0.3943	0
2	0.592	0.4928	0	0	0.4622
3	0.535	0	0.0559	0.0157	0
4	0.304	0	0.1917	0.2654	0
5	0.229	0.1082	0	0	0.0546
6	0.229	0	0.0147	0.0654	0
7	0.205	0	0.038	0.0544	0
8	0.166	0	0.0012	0.0524	0
9	0.149	0.3494	0	0	0.4363
10	0.13	0.0077	0	0	0.0213

3.7 Non-structural component

Non- structural components were considered on the basis of damping ratios varying from 1%,2%,5%,10% and 20% as mentioned in earlier section. The non-structural components were modelled as joint objects in the building models assigning load, damping and stiffness corresponding to various damping ratio of non-structural components considered as shown in figure 3.8.

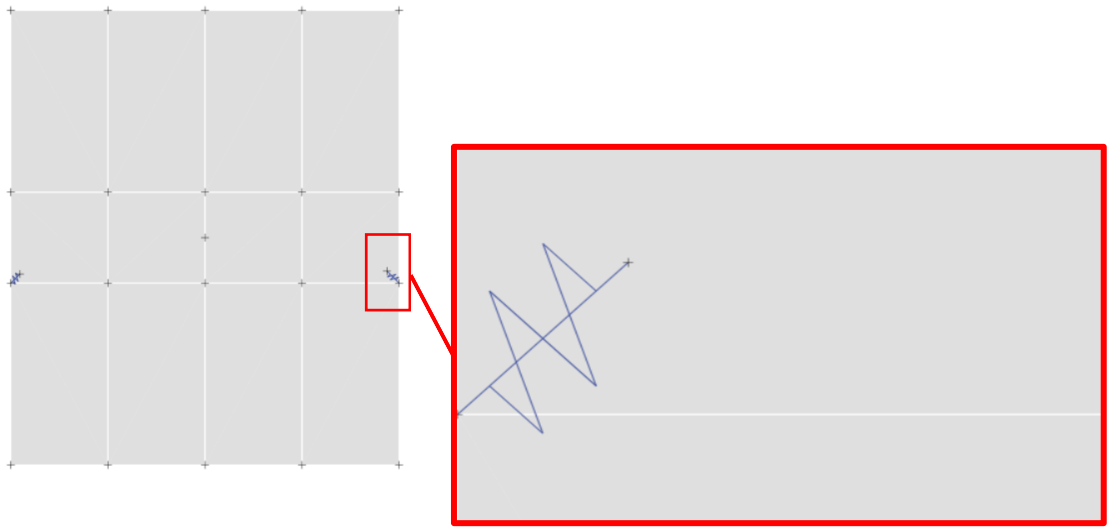


Figure 3.8: Non-structural component modelled as joint object and linked with link at flexible and stiff edge of the floor.

The time period of the component representing as time period of non-structure component time period is used in calculating the tuning ratio, which is the ratio of time period of the non-structural component to the fundamental period of the building. Mass of non-structural component is varied along with the time period of the non-structural component corresponding to the varied tuning ratio as per required. Mass and time period of the model is used to calculate the stiffness and damping constant for the non-structural component.

$$k = \frac{4 \times \pi^2 \times m}{T^2} \tag{1}$$

$$c = 2 \times \zeta \times \sqrt[2]{k \times m} \tag{2}$$

Where m is mass of non-structural component, T being the time period of the component, ζ is damping ratio and k is stiffness of the non-structural component

These parameters are used to define the link properties for the model in ETABS. Taking Building on 5-degree slope case where the fundamental time period of building is 0.589 sec and mass of non-structural component is taken as 500 N. The values calculated for the stiffness and damping constant by using equation (1) and (2) are 25.30 kN/m and 71.14kN-s/m respectively. Similarly, 20 different cases are considered for each type of building and each type of non-structural component considered. In total 200 different cases were considered for each type of non-structural components.

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 General

This chapter presents the results of the analytical investigation and their interpretation with respect to torsional behaviour of the building. The variation of the torsion irregularity index is examined to assess the extent of irregularity under different conditions. The torsional amplification factor is then evaluated to understand its influence on seismic response. Further, the correlation between the torsional amplification factor and the displacement-based torsion irregularity index is explored to establish their interrelationship and highlight the consistency or variation in capturing torsional effects. The discussion is aimed at providing insights into the significance of these parameters in characterising torsional response and their implications for seismic design.

4.2 Torsion irregularity index

The displacement-based torsional irregularity index, which is the ratio of the maximum edge displacement to the average edge displacement at a specific floor level ($\Delta_{max}/\Delta_{avg}$), is used to assess the torsional irregularity of step-back buildings. Figure 4.1 represents torsional irregularity variances where Z and H stand for floor height and total building height, respectively, and are both measured from the lowest foundation level. This index is calculated along the building's height, and Fig. 48 shows the variances.

The findings show that the greatest torsional impacts always take place at the base floor level and gradually decrease from there both upward and downward. Torsional irregularity is confirmed in line with the fact that the displacement-based index ($\Delta_{max}/\Delta_{avg}$) is detected to surpass the crucial threshold of 1.40. Significant differences are observed at the intermediate levels, which extend from the first floor above the foundation to the base floor below it, even though the torsional demand at the roof level is generally equivalent throughout the step-back buildings under investigation. One significant finding from the analysis is that, for a given plan arrangement and slope, torsional effects at the roof level decrease.

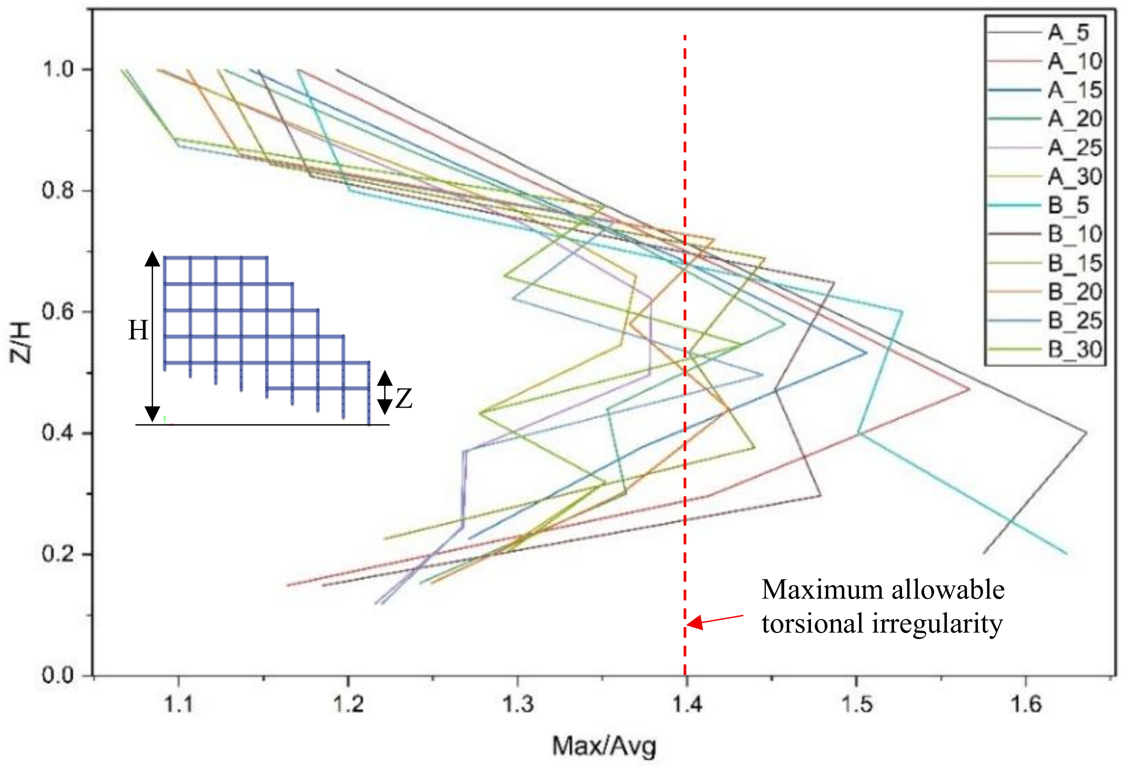


Figure 4.1: Torsion irregularity index along the height of building

4.3 Torsional amplification factor

Step-back buildings absolute floor acceleration responses are measured in this study at various floor levels and locations on the floor plan in an across-slope direction. The floor acceleration response spectra corresponding to non-structural component (NSC) damping ratios (ξ_s) of 1%, 2%, 5%, 10%, and 20% are then constructed using these responses. For a given NSC damping ratio, the torsional amplification factors (TAFs) are the ratio of the floor spectral acceleration at the flexible edge (FE) or stiff edge (SE) to that at the centre of rigidity (CR). These TAFs are expressed as a function of the tuning ratio (T_s/T_1), where T_1 is the fundamental building period and T_s is the NSC period. These relationships are evaluated in the across-slope direction for all cases considered. The resulting torsional amplification spectra, obtained at the base floor level for NSC damping ratios of 2%, 5%, and 10%, are illustrated in figure 4.2, 4.3 and 4.4 below.

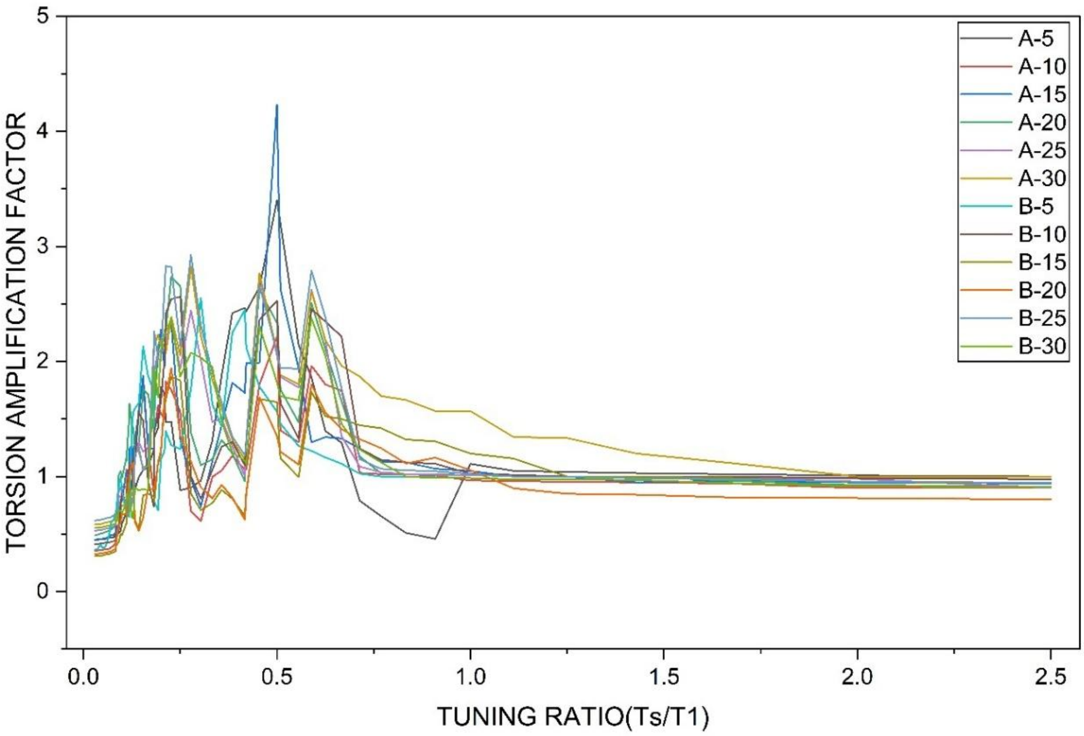


Figure 4.2: Spectral amplification factor graph at base floor corresponding to 2%

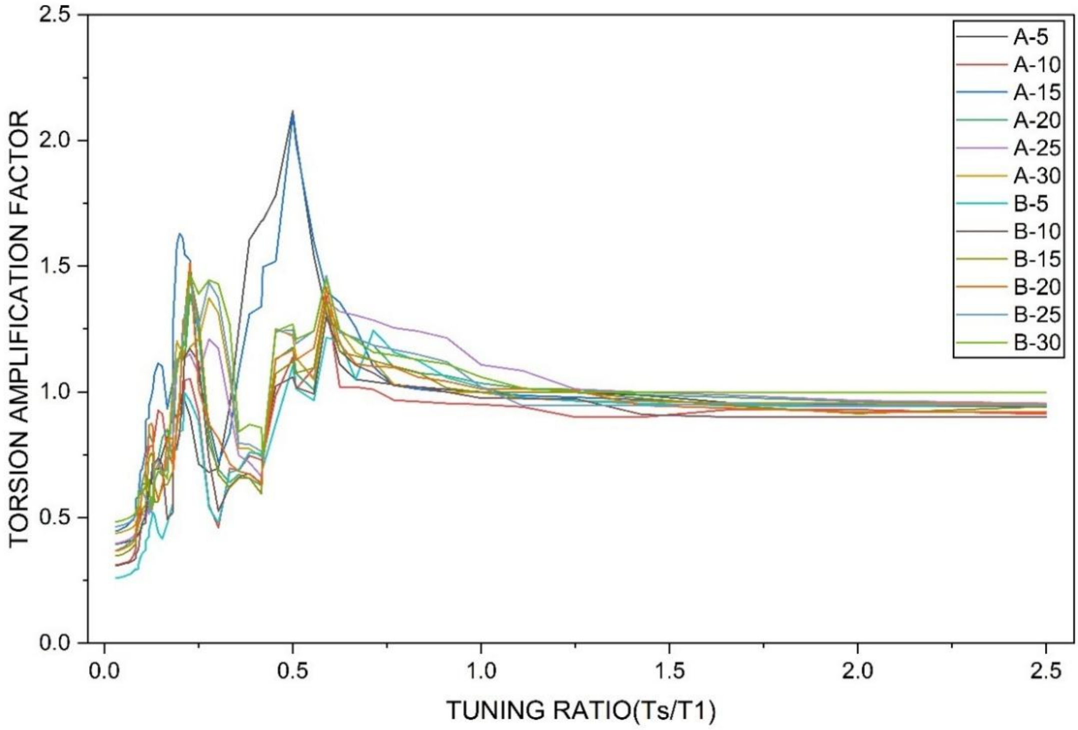


Figure 4.3: Spectral amplification factor graph at base floor corresponding to 5%

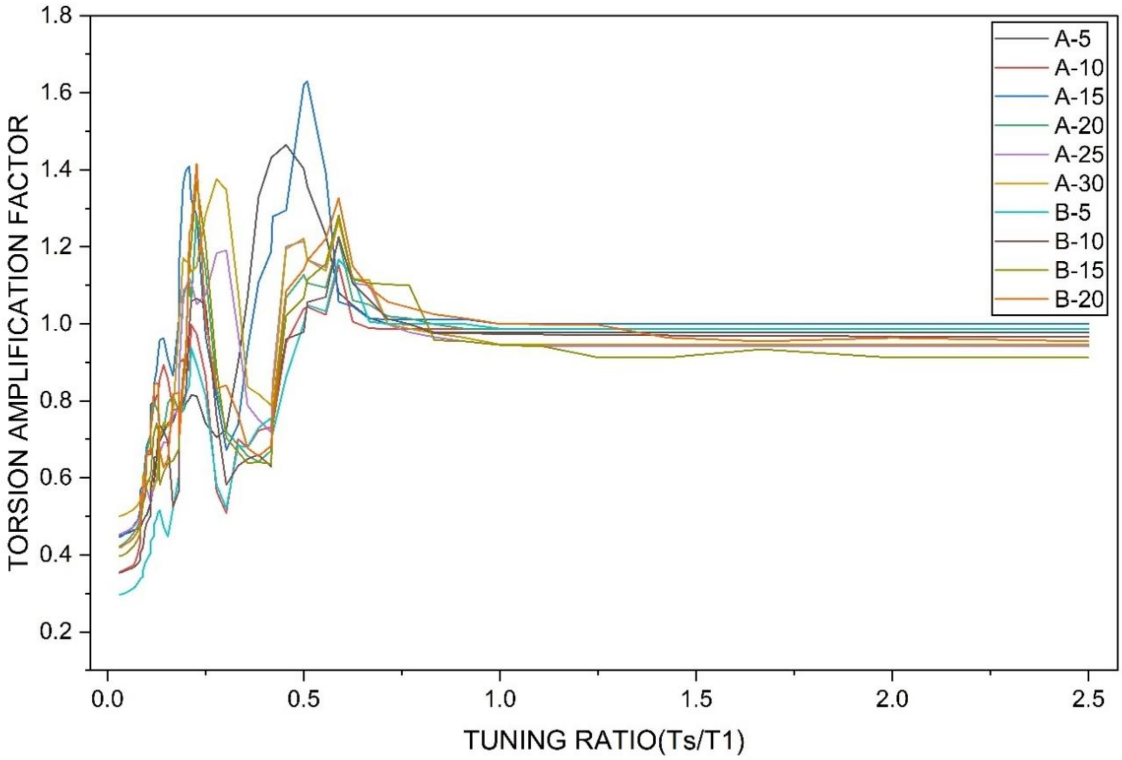


Figure 4.4: Spectral amplification factor graph at base floor corresponding to 10%

The findings demonstrate that torsional amplification always takes place at the FE and torsional de-amplification at the SE with a tuning ratio of $T_s/T_1=0$. Regardless of the NSC damping ratio, both amplification and de-amplification take place for $T_s/T_1 \leq 0.50$ for the FE. Depending on the building configuration and NSC damping, the peak (median) TAFs in this range from 1.5 to 4.0. Torsional amplification is consistently seen at the base floor FE for all tuning and damping ratios, with peak (median) TAFs in the higher-mode influence zone ranging from 1.5 to 4.0. On the other hand, torsional de-amplification is shown for all modes and damping ratios near the southeast corner of the base floor, where TAFs are getting close to unity. This is explained by the short, ground-supported columns that are present, which bring the SE close to the centre of stiffness.

For low NSC damping ratios, both the FE and SE exhibit torsional amplification at the base floor, which corresponds to torsional vibration modes. At larger damping ratios, however, the SE exhibits de-amplification (with TAFs approaching unity). The FE confirms the previous finding that torsional effects at the roof diminish with increasing building height. Maximum torsional amplification is seen in the proximity of various modal influence zones, while minimum values are also detected.

The median TAFs for very flexible NSCs ($T_s/T_1 \geq 1.50$) converge toward unity in all the examples examined, indicating that the combined impact of the NSC characteristics and inherent building torsion becomes insignificant for such systems. These results serve as the foundation for creating a simpler prediction model for the seismic design of acceleration-sensitive NSCs in torsionally irregular step-back buildings and offer crucial insights into the behavior of torsional amplification spectra.

4.4 Correlation between torsional amplification factor and displacement based torsional irregularity indices

Correlations between the TAFs for the rigid and flexible NSCs and their floor displacement-based torsional irregularity index ($\Delta_{max} / \Delta_{avg}$), which is advised in seismic design codes, IS 1893, and ASCE 7, are examined to create a simplified torsional amplification prediction model. As was previously mentioned, there are three unique features present in the torsional amplification/de-amplification spectra:

(i) the TAF at $Ts / T_1 = 0$ (i.e., corresponding to the rigid NSC),

(ii)the peak TAF in the influence zone of different modes of vibration (i.e., corresponding to the flexible NSC), and

(iii) the TAF for the very flexible NSCs.

Among the three identified characteristics, the TAF corresponding to very flexible NSCs is found to be independent of both building and NSC properties. Hence, in this section, attention is restricted to the other two cases: (i) the TAF at $T_s/T_1=0$ and (ii) the peak TAF within the influence zone of higher building modes, obtained as the maximum TAF value from the torsional amplification/de-amplification spectra for tuning ratios between 0.05 and 1.10. Δ_{avg} is the average of the largest displacements at the flexible edge (FE) for a specific floor, Δ_{max} is the absolute maximum displacement at the FE. The torsional irregularity index ($\Delta_{max}/\Delta_{avg}$) is compared to the variation of TAFs for stiff NSCs ($T_s/T_1=0$) at both FE and SE, across all floors. All step-back building configurations and ground motion data have these results depicted. The graphs are valid for all NSC damping ratios since damping has no effect on rigid NSC behavior. Both amplification and de-amplification are feasible at the SE for rigid NSCs, according to the findings. levels above the foundation level are usually related with amplification, whilst levels below it is associated with de-amplification. Most of the time, torsional amplification is seen at the FE, but occasionally, de-amplification is also seen. Crucially, in accordance with IS 1893 and ASCE 7 requirements, TAFs at the FE exhibit a definite positive association with the displacement-based torsional irregularity index ($\Delta_{max}/\Delta_{avg}$). TAFs at the SE, on the other hand, have little to no association with this index. Exponent of 1.49 are obtained via regression analysis of TAFs at the FE for rigid NSCs, with matching goodness-of-fit values of 0.52 as shown in figure 4.5.

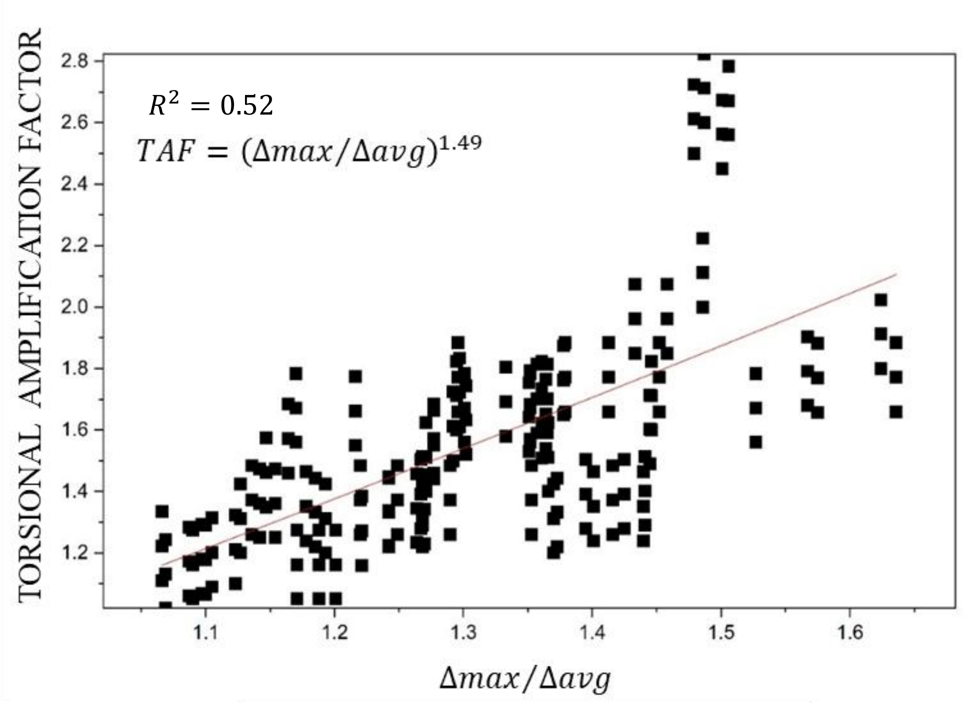


Figure 4.5: Torsional amplification factors corresponding to rigid NSC's at flexible end

The highest spectral value for flexible NSCs ($0.05 \le T_s/T_1 \le 1.10$) is determined by extracting the peak TAF at each floor. The findings show that peak TAFs at the FE show a substantial positive association with the displacement-based torsional irregularity score, whereas those at the SE show no link at all. The torsional irregularity index ($\Delta_{max}/\Delta_{avg}$) causes peak TAFs to rise sharply at the FE; regression analysis yield goodness-of-fit values between 0.7 and 0.81 and exponents between 2.54 and 3.42. As the NSC damping ratio increases, the exponent decreases as shown in figure 4.6 to 4.10.

Overall, the findings show that for both rigid and flexible NSCs, displacement-based torsional irregularity indices offer a strong association with TAFs at the FE. Torsional amplification is the most important factor for flexible NSCs, according to the comparison of regression exponents. Furthermore, the comparatively minor impact of the NSC damping ratio on the predictive exponent implies that peak torsional amplification in floor responses can be accurately predicted using a single displacement-based index that is not reliant on damping. Lastly, the values obtained for the FE can also be used for the SE in creating a simplified torsional amplification prediction model because TAFs at the SE are often lower or equivalent to those at the FE.

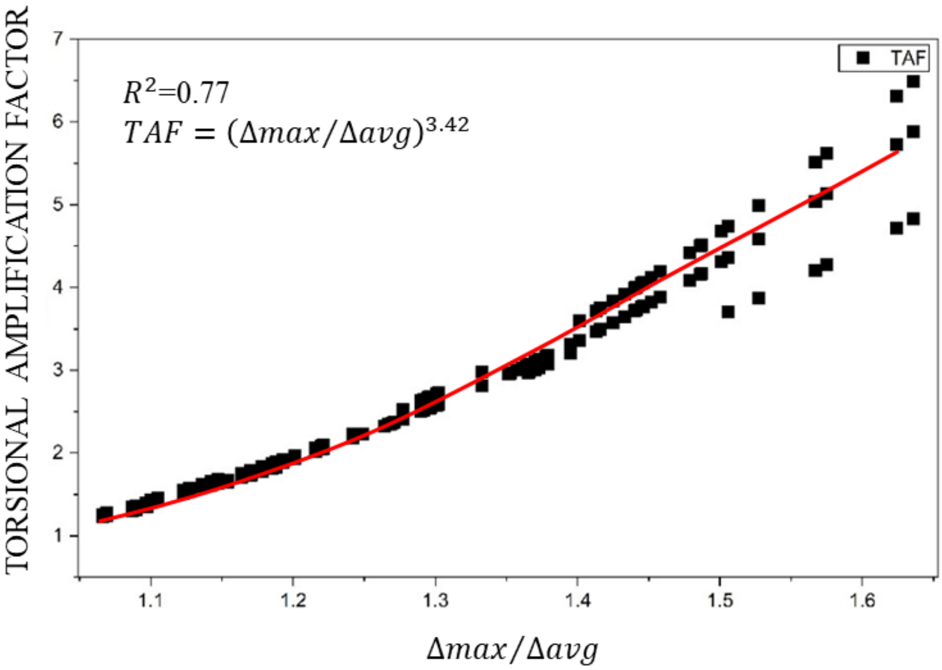


Figure 4.6: Torsional amplification factors corresponding to flexible NSC’s at flexible end 1% damping

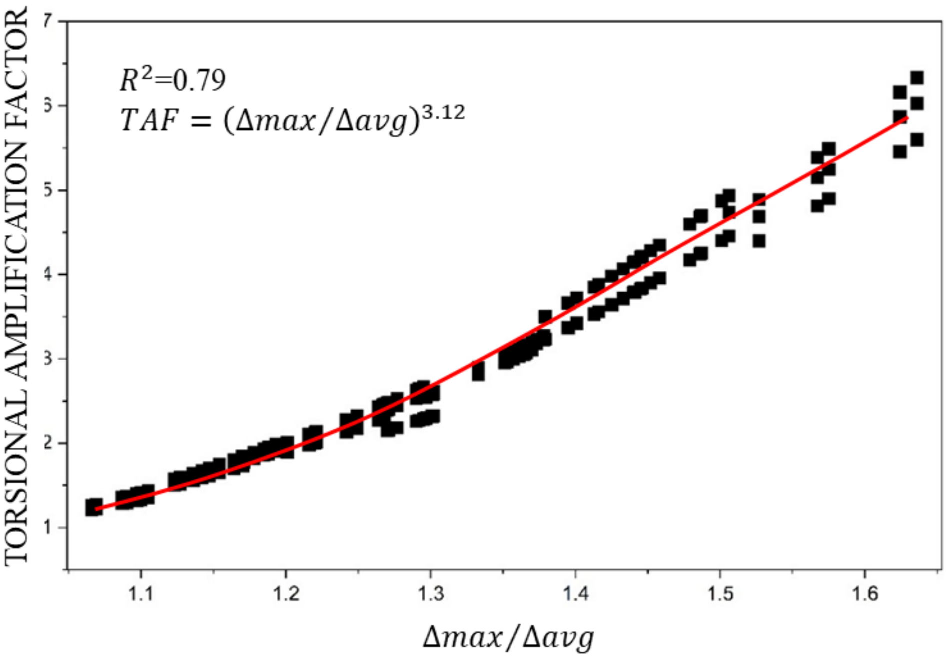


Figure 4.7: Torsional amplification factors corresponding to rigid NSC’s at flexible end 2% damping

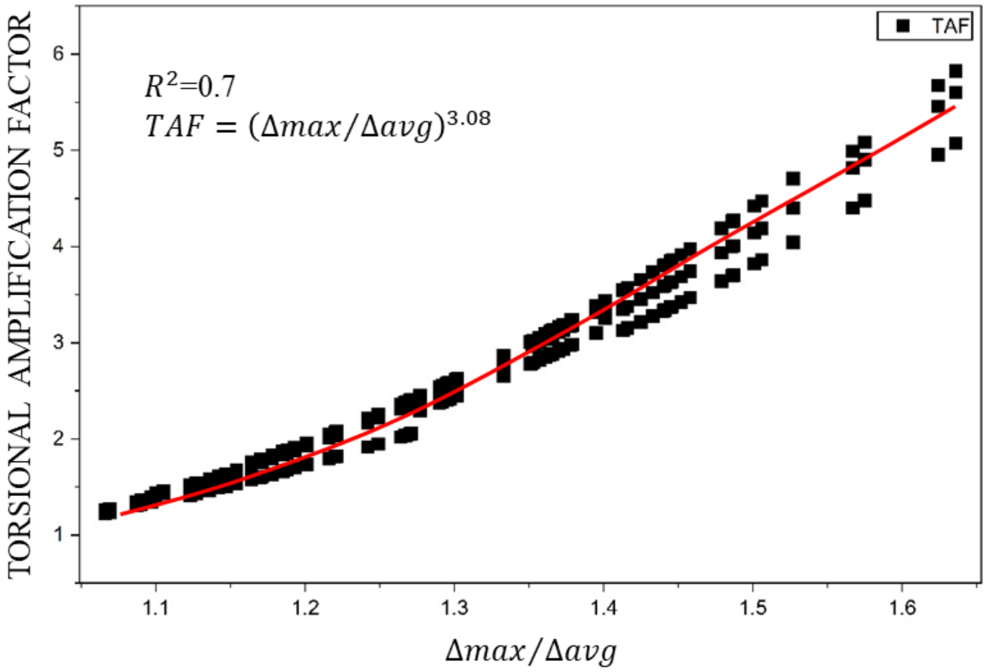


Figure 4.8: Torsional amplification factors corresponding to rigid NSC's at flexible end 5% damping

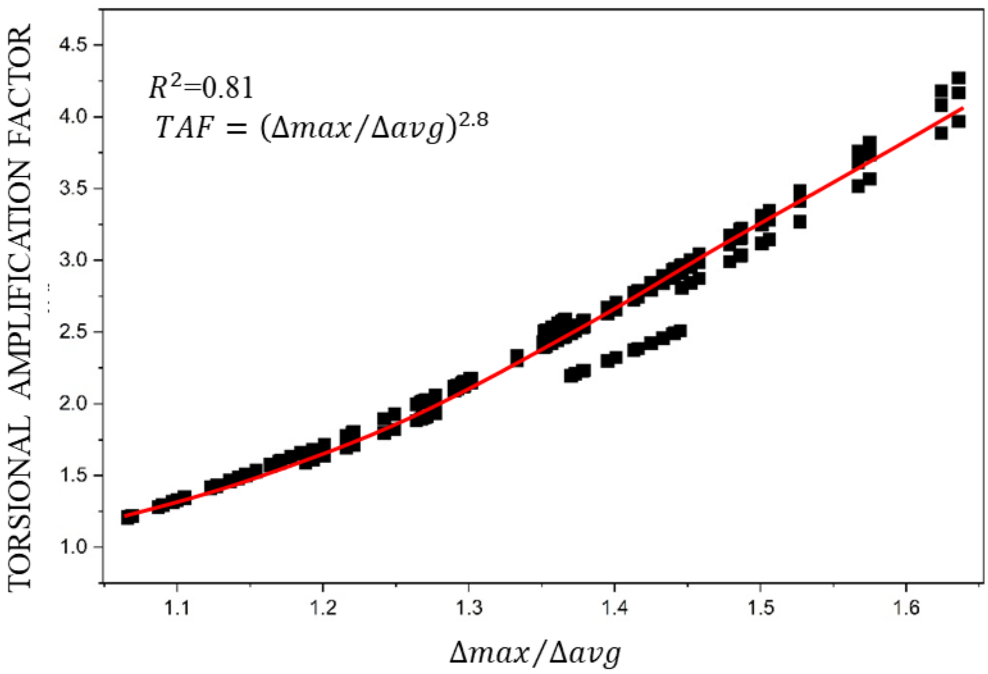


Figure 4.9: Torsional amplification factors corresponding to rigid NSC's at flexible end 10% damping

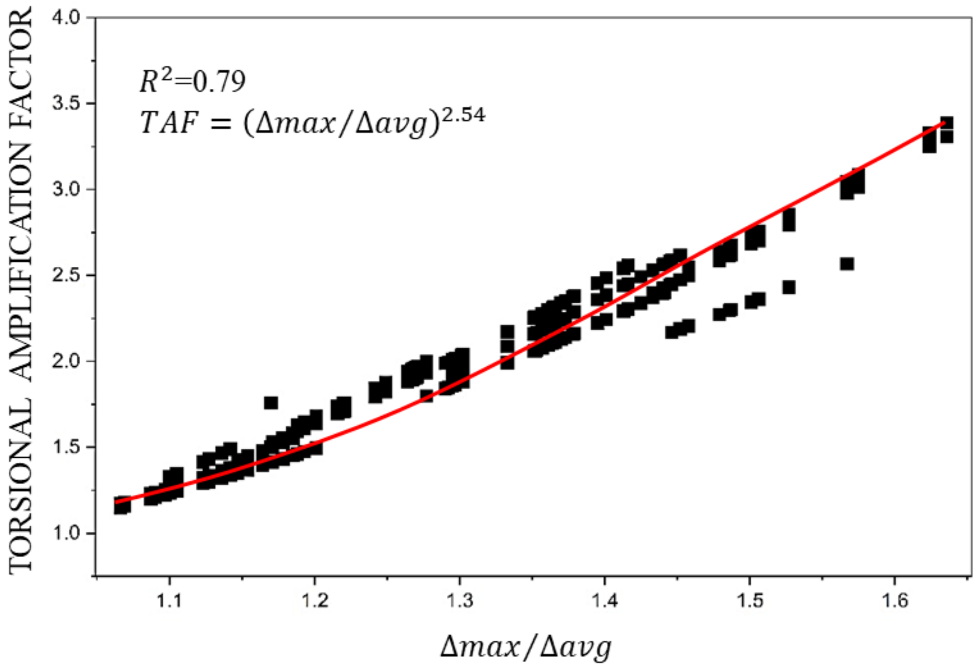


Figure 4.10: Torsional amplification factors corresponding to rigid NSC's at flexible end 20% damping

Non-structural components (NSCs) are categorized as flexible if their vibration periods (T_s) are significantly greater than the building's fundamental period (T_1), and as very flexible if their Ts are greater than $1.5T_1$. Torsional amplification factors (TAFs) for extremely flexible NSCs that correspond to tuning ratios of 1.5, 2.0, 3.0, and 4.0 are shown in Figures 4.11 and 4.12. The plots contain information from every building under investigation, assessed at each floor level's stiff edge (SE) and flexible edge (FE). The median TAF values are found to lie between 0.92 and 0.98 at the SE and between 1.08 and 1.2 at the FE. These results indicate that, for very flexible NSCs, the TAF approaches unity at both the SE and FE, implying that torsional irregularity has negligible influence on their seismic response.

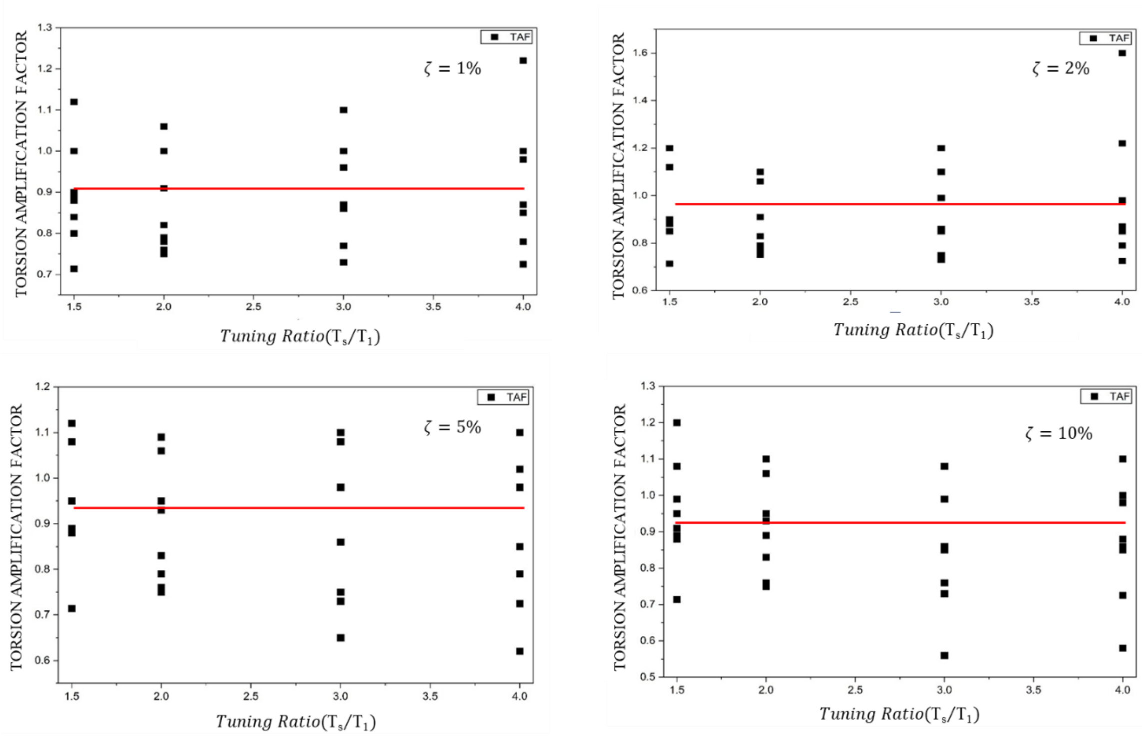


Figure 4.11: Torsional amplification factors corresponding to very flexible NSC’s at rigid end

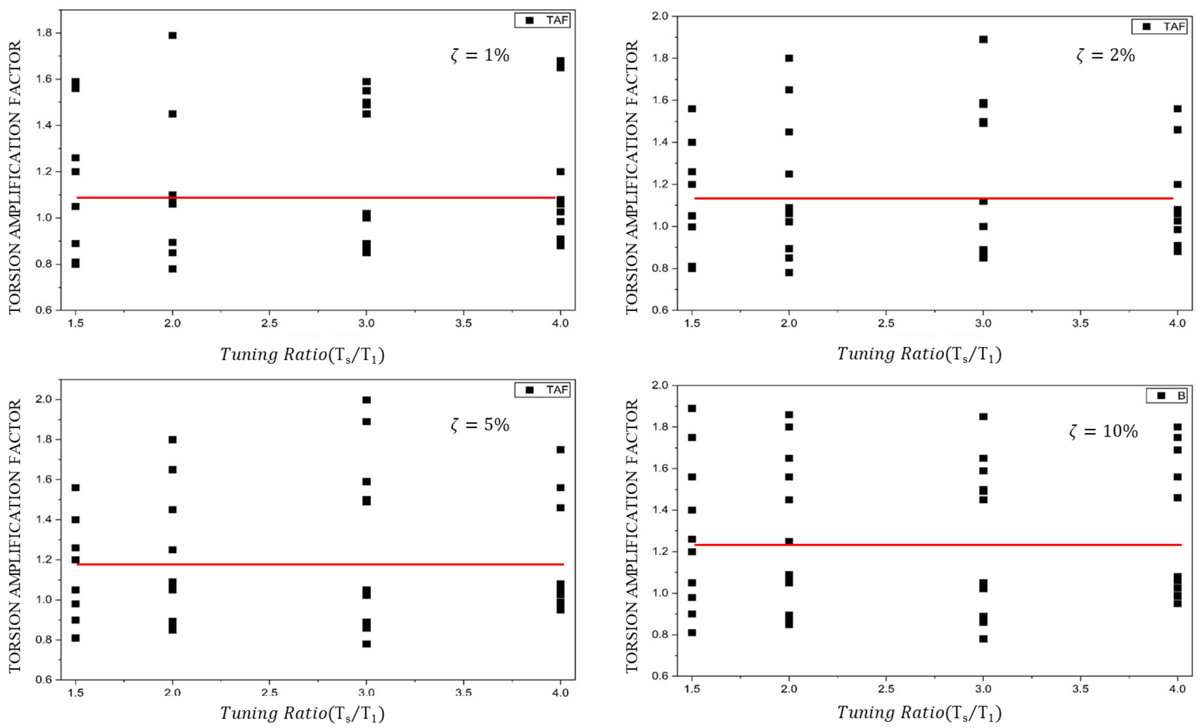


Figure 4.12*:* Torsional amplification factors corresponding to very flexible NSC’s at flexible end

CHAPTER 5

PROPOSED MODEL

5.1 General

This chapter presents the proposed model developed for the torsionally irregular buildings, along with its formulation and underlying assumptions. The modelling approach is detailed to highlight its applicability in capturing the key aspects of structural response. The accuracy of the proposed model by Surana [24] is then examined through validation against benchmark results or existing studies, ensuring its reliability for further analysis.

5.2 Model

Based on the identified characteristics of the torsional amplification spectra and the findings presented in the preceding sections, a simplified multi-linear model is developed to estimate TAFs for the seismic design of NSCs in torsionally irregular buildings. Since torsional amplification may occur even at the stiff edge (SE), the model conservatively disregards the possibility of torsional de-amplification. Consequently, it is applicable to NSCs located at any position on the floor plan of a torsionally irregular building. The formulation of the proposed model involves determining three characteristic TAF values, which together define its multi-linear variation as a function of the tuning ratio (T_s/T_1). These three different values of the TAFs are

(i) TAF (rigid) at $T_s/T_1 = 0$, which represents the torsional amplification factor (corresponding to the rigid NSC),

(ii) TAF (flexible) corresponding to T_s/T_1 between 0.05 and 1.10, which represents the peak value of torsional amplification factor in the influence zone of different modes of vibration of the building, and

(iii) TAF (very flexible) for $T_s/T_1 \geq 1.50$, which represents the torsional amplification factor for the very flexible NSCs.

For tuning ratios between the defined points, specifically within the ranges $0<T_s/T_1<0.05$ and $1.10<T_s/T_1<1.5$, the proposed model assumes a linear variation of the TAFs. The overall shape of the torsional amplification prediction model is derived from the trends observed in the previous sections. However, instead of replicating the sharp peaks present in the torsional amplification spectra, the model adopts a flat plateau as shown in figure 5.1. This simplification accounts for the uncertainty associated with the building-to-building variability in the location of these peaks, which are influenced by the complex dynamic characteristics of the structures and differ across floor levels and configurations.

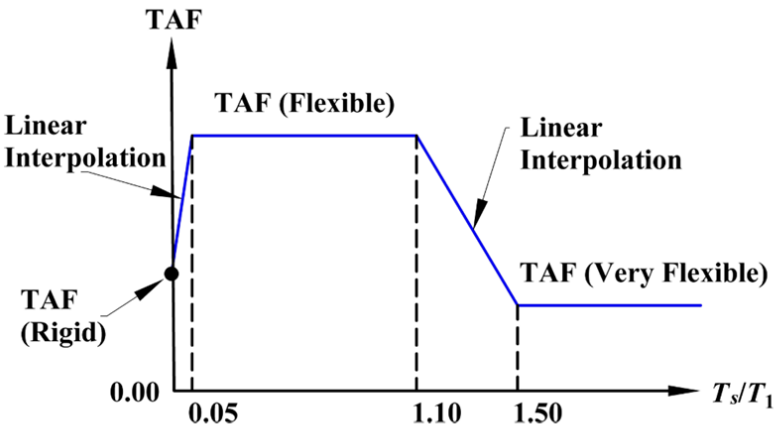


Figure 5.1: Proposed multi-linear model for predicting floor torsional amplification factor

As demonstrated in the preceding sections, the TAFs at $T_s/T_1 = 0$ (i.e., the rigid NSC) and for T_s/T_1 between 0.05 and 1.10 (i.e., the flexible NSC) are found to be nearly independent of the damping ratio of the NSCs and to have a strong correlation with the displacement-based torsional irregularity index, or $\Delta_{max}/\Delta_{avg}$. However, the TAFs for the extremely flexible NSCs are shown to be unaffected by the damping ratio of the NSCs, their location inside the building, and the torsional characteristics of the building. Since the torsional irregularity index, or $\Delta_{max}/\Delta_{avg}$, at the floor level under consideration for the building of interest is known, Eq. are given to calculate the TAFs as a function of tuning ratio (T_s/T_1) based on the regression studies described in the previous section. It should be mentioned that there may be slight variations in the predicted TAFs at a specific floor level of a torsionally irregular building when employing the two distinct torsional irregularity indices, $\Delta_{max}/\Delta_{avg}$. The following are the suggested formulas for calculating the TAFs as a function of the tuning ratio:

$$\mathrm{TAF} = \begin{cases} \left(\dfrac{\Delta_{max}}{\Delta_{avg}}\right)^{1.50} & \text{At } T_s/T_1 = 0 \\ \left(\dfrac{\Delta_{max}}{\Delta_{avg}}\right)^{3.00} & 0.05 \leqslant T_s/T_1 \leqslant 1.10 \\ \text{linear interpolation} & 0 < T_s/T_1 \leqslant 0.05 \text{ and } 1.10 < T_s/T_1 \leqslant 1.50 \\ 1.20 & 1.50 \leqslant T_s/T_1 \end{cases}$$

5.3 Validation of model with results:

Surana [24] proposed the model for the torsion amplification factor as a function of ratio of delta max/ delta avg i.e. torsion irregularity index. The goodness fit curve is derived for each graph and equation is derived with torsion amplification factor (TAF) being a function dependent on torsional irregularity indices as in in graphs shown. The equation being:

$$TAF = \left(\frac{\Delta max}{\Delta avg}\right)^{n}$$

The value of n varies according to the type of non-structural component considered. The values for the same are shown in table for comparison between the results achieved for different models but torsionally irregular building.

Table 5.1: Result validation

Type of NSC	Surana [24]	Present Author	% Error
Rigid NSCs	1.54	1.47	4.76%
Flexible NSCs (1%)	3.53	3.42	3.21%
Flexible NSCs (2%)	3.33	3.12	6.73%
Flexible NSCs (5%)	2.96	3.08	3.89%
Flexible NSCs (10%)	2.69	2.8	3.92%
Flexible NSCs (20%)	2.39	2.54	5.90%

The table 5.1 shows that results are in good comparison with respect to reference research paper (Surana [24]). Based on the analysis conducted in ETABS, it could be asserted that the proposed model by Surana [24] is valid across torsionally irregular building having setback and stepback configurations.

CHAPTER 6

CONCLUSION

6.1 Conclusion:

The impacts of the building's torsional response can be easily incorporated into the seismic design of lightweight acceleration-sensitive NSCs using torsional amplification factors (TAFs). Therefore, bi-directional linear dynamic time-history studies are carried out on various torsionally irregular RC buildings in order to examine TAFs for acceleration-sensitive NSCs. At various floor levels of the buildings under investigation, floor acceleration response spectra are assessed for a range of NSC damping ratios, including 1%, 2%, 5%, 10%, and 20%. Using the floor displacement-based torsional irregularity indices ($\Delta_{max}/\Delta_{avg}$) suggested by the national building codes of the United States and India, the correlations of the TAFs at various floors for the rigid and flexible NSCs are examined. The following noteworthy findings are derived from the numerical analysis that is carried out:

1. TAFs are shown to be dependent on tuning ratios and building torsional features; the flexible NSCs exhibit their most critical value.
2. For both stiff and flexible NSCs, a torsional amplification typically takes place at the FE. However, regardless of the building's and the NSC's characteristics, TAFs approach unity for the highly flexible NSCs.
3. Since the damping ratio of NSCs has little effect on the peak TAFs, TAFs can be taken to be independent of the damping ratio of NSCs for all practical seismic design and evaluation applications.
4. TAFs at various floor levels for the rigid and flexible NSCs are advised and found to be highly associated with the corresponding floor displacement-based torsional irregularity index, i.e., $\Delta_{max} / \Delta_{avg}$. This discovery emphasizes how the flexible and stiff edge displacement response may be used to directly predict the torsional amplification in the floor response of a torsionally irregular building.

6.2 Scope of future study

The present study shows the correlation between the torsional amplification factor and the torsional irregularity index for the acceleration sensitive non-structural components varied across different setback-stepback configurations with varying ground slopes. Building on these findings, several avenues for future research can be pursued to enhance the seismic design and understanding of NSCs:

1. Future studies could investigate the effects of sloping terrains and soil-structure interaction on torsional amplification factors.
2. While this study focuses on acceleration-sensitive NSCs, subsequent research could incorporate other types of NSCs, including displacement-sensitive and non-linear components, to develop more generalized TAF recommendations.
3. Although damping was found to have minimal impact in the present study, further parametric investigations could examine extreme damping values and variable NSC tuning conditions to refine design recommendations.
4. Experimental studies on scaled models or full-scale buildings could validate the numerical results and enhance confidence in using TAFs for seismic design of NSCs.

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