FEUP FACULDADE DE ENGENHARIA
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INFLUENCE OF THE VERTICAL IRREGULARITIES IN THE SEISMIC DESIGN AND PERFORMANCE OF RC BUILDINGS

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To my family, for their support and unconditional love throughout this journey

Wisdom is not a product of
schooling but of the lifelong
attempt to acquire it.

Albert Einstein

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ABSTRACT

The research aims to advance the field of seismic design by examining the structural behaviour of reinforced concrete (RC) structures in accordance with the first and second generations of Eurocode 8, mainly focusing on the vertical irregularity criteria. The document integrates substantial theoretical, comparative, and numerical research on seismic codes, vertical irregularity criteria comparison and structural behaviour under seismic actions.

The introduction outlines the thesis's scope and objectives, highlighting the need to advance seismic-resistant design methodologies in response to the continual evolution of building codes, with a specific focus on structural vertical irregularities. It contextualizes the critical role of seismic structural engineering in mitigating earthquake-induced damage to infrastructure.

The thesis conducts a review of international seismic design standards. It compares them, mainly focusing on vertical irregularity criteria and the regularity classification impact in the seismic behaviour factor. It was presented the lessons learned from experiences of structural damages in seismic events associated with structural irregularities in China, Italy, Spain, Nepal, Mexico and Turkey. Additionally, this section compared the vertical irregularity criteria through several seismic codes, including those from Europe, Brazil, Mexico, Chile, USA and New Zealand.

The chapters dedicated to numerical studies present a comparative evaluation of the effects of the first- and second-generation vertical irregularity criteria outlined in Eurocode 8 on the seismic performance of Ductility Class Medium (DCM) RC moment-resisting frames and shear wall systems. It evaluates the impact of imposed vertical irregularity due to increasing height, column cross-section, inter-storey mass and resistance shift. Furthermore, it was assessed the combined influence of irregularity in elevation and corrosion in seismic response. Through comprehensive numerical simulations and structural analyses, the study elucidates the extent to which these parameters influence the overall structural integrity and safety of buildings subjected to seismic loading.

From those contributions, an assessment was conducted to examine the changes in the methodology for calculating the behaviour factor between the first and second generations of Eurocode 8 as applied to the developed structural models. The main purpose of this chapter is to evaluate different levels of vertical irregularity and propose the correspondent intermediate corrections levels of behaviour factor. The findings highlight the differential impact of vertical irregularity on seismic response and support the proposed amendments for inclusion in the draft of the second generation of Eurocode 8.

Finally, the document concludes by synthesizing the research findings and outlining prospective directions for the continued development of seismic design standards and practices. It identifies key areas where further investigation is warranted to enhance the understanding and assessment of structural irregularities in RC buildings, with the aim of improving their seismic performance. Emphasis is placed on the necessity for ongoing innovation and adaptability within the field of structural engineering to address the evolving challenges presented by seismic hazards.

KEYWORDS: Vertical irregularity, RC structures, Eurocode 8, seismic codes, behaviour factor.

RESUMO

A investigação visa o avanço na área de projeto sísmico, examinando o comportamento estrutural de estruturas de betão armado (BA) de acordo com a primeira e a segunda gerações do Eurocódigo 8, com foco principal nos critérios de irregularidade em altura. O documento integra pesquisa teórica, comparativa e numérica sobre códigos sísmicos, comparação de critérios de irregularidade em altura e comportamento estrutural sob ações sísmicas.

A introdução descreve o escopo e os objetivos da tese, destacando a necessidade de avançar em metodologias de projeto resistentes a sismos em resposta à evolução contínua dos códigos de construção, com foco específico em irregularidades verticais estruturais, contextualizando o papel crítico da engenharia estrutural sísmica na mitigação de danos à infraestrutura induzidos por terremotos.

A tese realiza uma revisão das normas internacionais de projeto sísmico. Ela as compara, com foco principal nos critérios de irregularidade em altura e no impacto da classificação de regularidade no coeficiente de comportamento sísmico. São apresentadas as lições aprendidas com as experiências de danos estruturais em eventos sísmicos associados a irregularidades estruturais na China, Itália, Espanha, Nepal, México e Turquia. Além disso, esta seção comparou os critérios de irregularidade vertical em diversas normas sísmicas, incluindo as da Europa, Brasil, México, Chile, EUA e Nova Zelândia.

Os capítulos que abordam o trabalho numérico apresentam uma avaliação comparativa dos efeitos dos critérios de irregularidade em altura de primeira e segunda geração, descritos no Eurocódigo 8, sobre o desempenho sísmico de estruturas tipo pórtico de betão armado de Classe de Ductilidade Média (DCM) e sistemas tipo parede. Avalia-se o impacto da irregularidade em altura imposta devido ao aumento da altura, da seção transversal do pilar, da variação de massa e de resistência entre pisos. Além disso, avaliou-se a influência combinada da irregularidade em altura e da corrosão na resposta sísmica. Por meio de simulações numéricas abrangentes e análises estruturais, o estudo elucida em que medida esses parâmetros influenciam a integridade estrutural geral e a segurança de edifícios submetidos a sismos.

A partir dessas contribuições, foi realizada uma avaliação para examinar as mudanças na metodologia de cálculo do coeficiente de comportamento entre a primeira e a segunda geração do Eurocódigo 8, aplicadas aos modelos estruturais desenvolvidos. O principal objetivo deste capítulo é avaliar diferentes níveis de irregularidade em altura e propor os correspondentes níveis intermédios de correção do coeficiente de comportamento. Os resultados destacam o impacto diferencial da irregularidade em altura na resposta sísmica e fundamentam as alterações propostas para inclusão no projeto da segunda geração do Eurocódigo 8.

Por fim, o documento conclui sintetizando os resultados do trabalho desenvolvido e delineando as direções prospetivas para o desenvolvimento contínuo de padrões e práticas de projeto sísmico. Identificou-se áreas-chave nas quais se justifica uma investigação mais aprofundada para aprimorar a compreensão e a avaliação de irregularidades estruturais em edifícios de concreto armado, com o objetivo de aprimorar seu desempenho sísmico. Enfatiza-se a necessidade de inovação e adaptabilidade contínuas no campo da engenharia estrutural para enfrentar os desafios em evolução apresentados pelos riscos sísmicos.

PALAVRAS-CHAVE: Irregularidade em altura, estrutura de betão armado, Eurocódigo 8, normas sísmicas, coeficiente de comportamento.

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1 Introduction

1.1 MOTIVATION

Seismic Structural Engineering is a specialized discipline within structural engineering focused on minimizing damage resulting from seismic events. It encompasses the prediction of earthquake-induced impacts on physical structures, the design and development of resilient structural systems, and the evaluation and retrofitting of existing buildings. This field also ensures adherence to seismic design provisions and structural building codes to enhance safety and performance during ground shaking [1].

Given the inherently unpredictable nature of seismic activity, quantifying damage control within the framework of code provisions remains a complex task, primarily due to the need to define acceptable levels of structural risk. The central challenge in the efficient design of earthquake-resistant structures lies in achieving an optimal balance between seismic demand and the structural capacity to withstand seismic effects without experiencing catastrophic failure. This requires careful consideration of multiple parameters that characterize structural behaviour under dynamic loading conditions [1].

Observations on buildings' performance during strong earthquakes have served as means of teaching builders and engineers on proper and improper construction of earthquake load resisting systems [2–4]. In regions that have long been inhabited and subjected to relatively frequent firm ground shaking, design procedures have evolved, resulting in relatively good performance of engineered structures. Although such design procedures are not universally applicable due to regional differences, structural engineers can learn much by studying those procedures in construction materials and techniques [5].

Several damage mechanisms in buildings have been repeated and identified as inappropriate structural configuration, among them: torsion produced by the inclusion of asymmetrical masonry infills, first soft-storey buildings and poor longitudinal and transverse reinforcement detailing [6].

A torsionally unbalanced system, popularly known as a plan asymmetric system, occurs due to the noncoincidence of centre of mass (CM) and centre of stiffness (CS). To understand the behaviour of structures, they were ideally conceptualized as a representative single-story system with the same fundamental period, or at most a low-rise multistorey system whose CM need not lie in the same vertical line in only a few studies [7]. However, a multistorey torsionally unbalanced system having a CM and CS of each storey on the same vertical line is rare in practice [8].

Almost all the buildings have a lack of symmetry and irregularity, to some extent, at least for functional and/or architectural purposes. Most of the code provisions are neglected about multistorey asymmetric buildings, perhaps due to the complications involved in the definition of CM and CS, even though issues related to asymmetry were recognized much earlier [9–13]. It`s important to highlight that efforts have been made to locate the CS of each level story [14] and also to propose a methodology for the analysis of multistorey asymmetric structures without calculating the CS of each story [15].

Although considerable research effort has been made to study the seismic behaviour of asymmetric structure, relatively less effort has been made toward understanding the same for various types of irregular structures [8]. This study highlighted that very little research has been conducted toward structures with irregularities along both principal directions. Further research is needed with the various soil movements, degraded nature of the material properties, effectiveness of the location of the shear walls and many other related issues to perform a better seismic risk assessment.

The elevation of buildings may exhibit geometric irregularities along their height, often resulting from nonuniform vertical distributions of mass, stiffness, or structural strength. Such irregularities contribute to vertical discontinuities that can significantly affect a structure's seismic performance. A notable example is the open-storey configuration, which falls within this category of vertical irregularity. Variations in geometry along the building height can lead to localized stress concentrations, thereby increasing structural vulnerability. Common instances of this phenomenon include buildings with vertical setbacks and those constructed on sloped terrains, such as in hilly regions [8]. General forms of this type of irregularity are demonstrated in Figure 1-1.

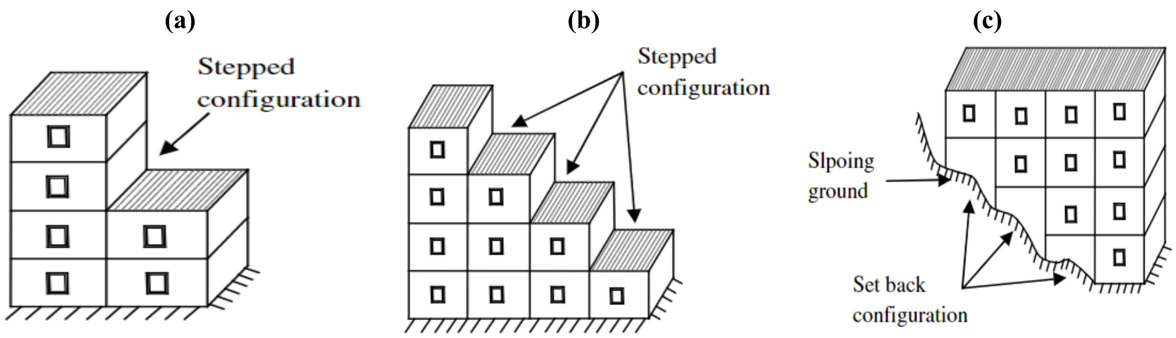


Figure 1-1: General forms of vertical irregularity. Adapted from Das et al. [8]

EN 1998-1:2004 [16] constitutes the current state-of-the-art reference for seismic design within Europe. Nevertheless, several of its provisions require revision to reflect recent advances in research and insights gained from practical application. The forthcoming second generation of Eurocode 8, prEN 1998-1-2:2024 [17], is currently under review and incorporates substantial updates across multiple domains. These include refinements to the ductility class framework, revised criteria for assessing structural irregularities in both plan and elevation, and the decomposition of behaviour factors, among other significant modifications.

A notable innovation in prEN1998-1-2:2024 [17] is the incorporation of displacement-based design, which complements the traditional force-based approach with reduced equivalent forces. This code introduces the necessary resistance models for applying this approach to reinforced concrete structures, thereby enhancing the accuracy and performance-based nature of seismic design.

EN 1998-1:2004 [16] permits the determination of the design seismic base shear force through elastic analysis, with subsequent reduction by a constant factor (behaviour factor) to account for the ductile, nonlinear behaviour of the structural system. The behaviour factor reflects the structure's capacity to dissipate energy through hysteresis, as well as its inherent overstrength and redundancy. However, the quantification of parameters related to structural behaviour remains an area of ongoing investigation. These issues continue to be the subject of active research and discussion within both the academic and professional engineering communities.

The behaviour factor (q), according to EN 1998-1:2004 [16] represents an approximation of the ratio between the seismic forces a structure would be subjected to under fully elastic response conditions, with 5% viscous damping, and the reduced seismic forces permitted for design purposes using a conventional elastic analysis model. This reduction is intended to ensure an adequate structural response, as outlined in the provisions of the code. The behaviour factor significantly influences the shape of the design response spectrum, particularly affecting the spectral acceleration values. Consequently, the magnitude of the design forces is highly sensitive to the value assigned to the behaviour factor.

In the other hand, prEN1998-1-2:2024 [17] introduce a behaviour factor composed by the product of the behaviour factors components:

1. q_{R}: component that accounts for overstrength due to the redistribution of seismic action effects in redundant structures;
2. q_{S}: component that accounts for overstrength due to all other sources;
3. q_{D}: component that accounts for the deformation and energy dissipation capacity.

q_{R} exceeding unity is only achievable in structures exhibiting redundancy, however, q_{R} is not a redundancy measure, nor is plastic deformation capacity. Rather, it quantifies the discrepancy between the internal force distribution obtained through linear elastic analysis and that which occurs near ultimate limit states, incorporating all relevant sources of nonlinearity. q_{S} primarily reflects material overstrength and the additional resistance arising from inevitable overdesign of structural elements. The q_{D} component captures the structure's capacity for deformation and energy dissipation, which is influenced by various factors including the deformation capacity of walls, the degree of wall coupling via slabs and ring beams, the spatial distribution of wall lengths, and torsional effects. These components of the behaviour factor play a critical role in displacement-based design approaches, particularly in relation to chord rotation analyses, inter-storey drift limits, and second order effects.

The subsequent chapters undertake a detailed examination of the aforementioned aspects, encompassing in-depth evaluations, comparative analyses, and a synthesis of the results. This discussion aims to develop a comprehensive understanding of the structural implications associated with vertical irregularities in RC buildings.

1.2 GOALS AND ORIGINALITY

This research addresses critical aspects of vertical irregularities in seismic design and detailing in RC structures. It is internationally recognized that the issue of structural irregularities is not yet fully developed or studied, with many current regulations having a simplified approach to the problem, so investing in research in this area is crucial.

The study aims to advance the understanding of irregularity in elevation criteria consideration and impact in design provisions for seismic-resilient structures by conducting a comprehensive investigation encompassing various chapters.

The specific objectives of this study are:

1. Comparative analysis of vertical irregularity criteria established by first and second generation of Eurocode 8 and other international seismic codes;
2. Assess the effect of each irregularity in elevation criteria in the RC structures seismic response;
3. Evaluate the combined effect of corrosion and vertical irregularity in the seismic response;
4. Proposal of a new scale to affect the behaviour coefficient of Eurocode 8 according to the level of irregularity;
5. Assess several existing buildings regarding vertical irregularities criteria from different seismic codes;
6. Evaluate the influence of irregularity in elevation classification, seismic standard considered in the design and differences in structural properties in seismic response of the existing buildings.

This work may contribute to the improvement of Eurocode 8 in the assessment of different levels of vertical irregularities and its impacts on the structural performance during seismic events, therefore it is intended to propose additional irregularity classification, such as irregular and strongly irregular structure. This new scale proposal will affect the behavior factor, q, according to the irregularity level. These can better adequate the use of the Eurocode 8 requirements for the seismic design of RC structures and consequently contribute to risk mitigation.

1.3 DOCUMENT ORGANIZATION

Figure 1-2 presents a structured overview of the document's organization, illustrating the logical flow between chapters. This visual representation facilitates navigation and enhances comprehension of the topics and tasks progression throughout the document. Additionally, it highlights the hierarchical relationship among the chapters, providing readers with a clear and coherent framework to follow.

Chapter 1, "*Introduction*", offers essential context and articulates the rationale underpinning the study. The motivations and objectives of the current investigation are outlined, thereby establishing a foundation for appreciating its significance. By delineating the driving factors that initiated the research, the introduction facilitates a deeper engagement with the topic. Additionally, it identifies gaps within existing knowledge or practice, highlights areas warranting further exploration and states the specific aims the study seeks to accomplish. Finally, the introduction serves as a conceptual guide, orienting the reader to the trajectory of inquiry pursued throughout the research.

Chapter 2, entitled "*Code requirements for the seismic design of irregular elevation RC structures*", describes and compares seismic codes from:

1. Europe: First and second generation;
2. Brazil;
3. Mexico;
4. Chile;
5. United States of America;
6. New Zealand.

This description and comparison are focused on structural irregularities in elevation criteria, regarding their quantitative or qualitative nature and their limitations. This chapter also discusses the most common types of structural damage due to seismic events and the documented experiences of structural damages in earthquakes associated with structural irregularities in China, Italy, Spain, Nepal, Mexico and Turkey. Finally, there are discussed the level of development of each assessed seismic code, considering the specific matter of vertical irregularities.

In Chapter 3, "*Comparative analysis of the impact of vertical irregularities on reinforced concrete moment resisting frame structures according to Eurocode 8*", a study is conducted concerning the influence of the irregularity in elevation, imposed by different elevations, on the ground and intermediate storeys of buildings, and variations in the cross-sections of columns in the seismic response of reinforced concrete (RC) structures. The research aims to evaluate the impacts of the vertical irregularity criteria in seismic response of framed RC structures (lateral stiffness, mass and resistance shift).

This examination was carried out by assessment of thirteen DCM (Ductility class medium) five-storey moment-resisting frame (MRF) buildings, regarding the first and second generation of Eurocode 8 vertical irregularity criteria. Firstly, those structures were designed considering the Eurocode 8 guidelines and classified as regular or irregular in elevation. All structures were conceived to be regular in plan.

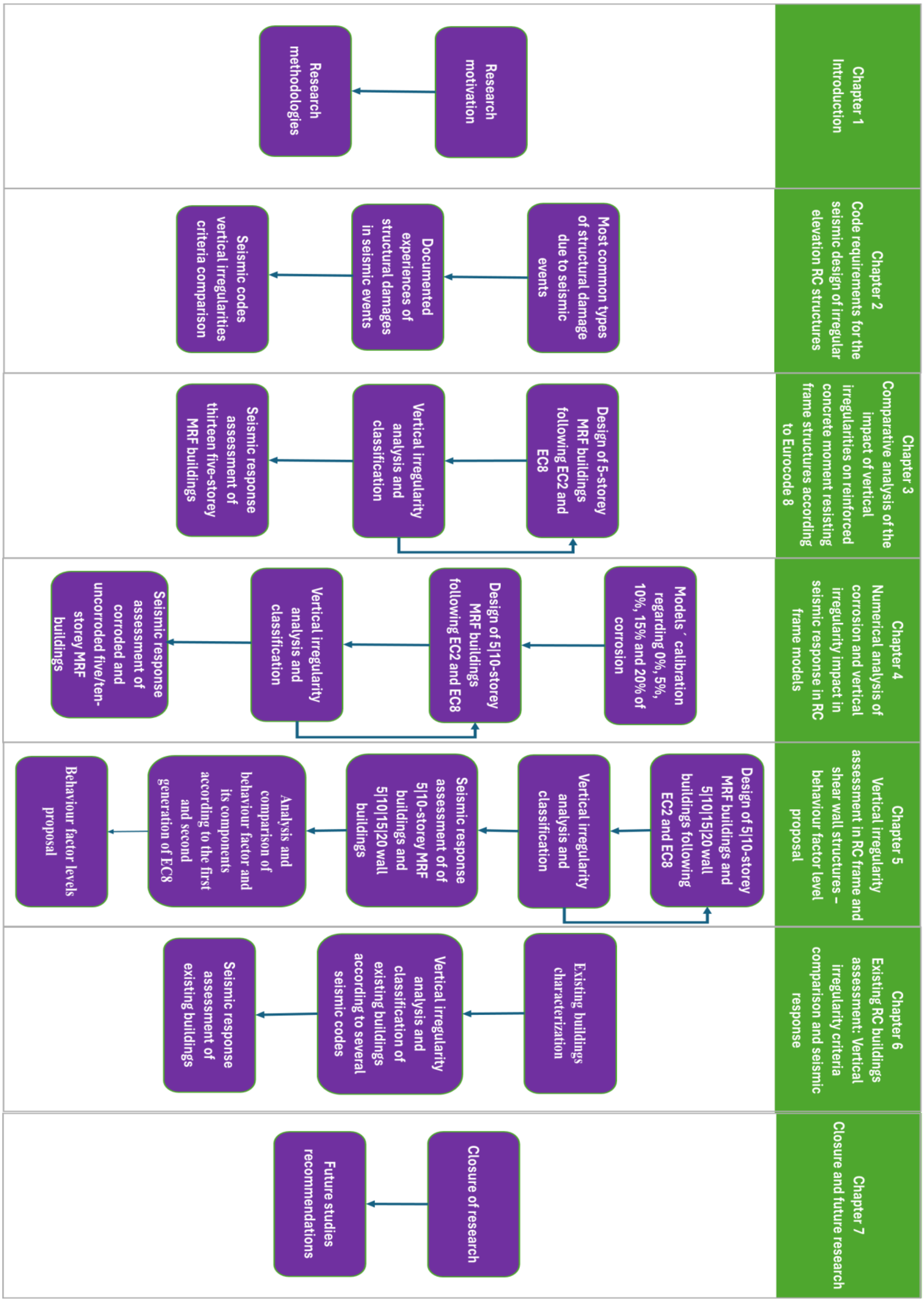


Figure 1-2: Document organization

The outcomes demonstrated the impact of increasing the heights of ground or middle floor, columns cross-sections reductions and mass and resistance shifts in buildings substantially impacts, in different levels, the seismic response.

The findings offer valuable information on vertical irregularities criteria impact in seismic response, providing the opportunity to better choose the control variable to assess this matter in further chapters.

Chapter 4, denominated "*Numerical analysis of corrosion and vertical irregularity impact in seismic response in RC frame models*", discusses the combined impact of rebars corrosion and vertical irregularity in MRF RC structures. To address this matter, it has been carried out the assessment of several experimental studies regarding corrosion in columns. With that information, it was performed the definition of the models' settings to each level of corrosion.

With the models' calibration, the 5-10 storey MRF RC structures were designed according to the first and second generation of Eurocode 8 guidelines. There was performed the vertical irregularity classification of each structure and, when necessary, the model was redesigned. Finally, with the corrosion levels settings and the designed structures, it was assessed the seismic response of those buildings with 0%, 5%, 10%, 15% and 20% of corrosion.

Chapter 5, named "*Vertical irregularity assessment in RC frame and shear wall structures – behaviour factor level proposal*", has the main goal the discussion of behaviour factor levels established by the first and second generation of Eurocode 8, specifically regarding vertical irregularities. Firstly, it was carried out the design and seismic response assessment of DCM frame and shear walls RC parametric models, considering vertical irregularity aspects. In this matter, the lateral stiffness was the control variable for irregularity in elevation.

Secondly, it was performed an analysis and comparison of behaviour factor and its components according to the first and second generation of Eurocode 8. From that assessment, it was proposed the different levels of irregularity in elevation and their corresponding behaviour factor limits to Eurocode 8.

Chapter 6, "*Existing RC buildings assessment: Vertical irregularity criteria comparison and seismic response*", considering all the discussion performed in the chapters regarding vertical irregularities, assesses several existing buildings regarding vertical irregularity criteria from different seismic codes and their seismic response.

Firstly, it was carried out the characterisation of each building and the vertical irregularity classification according to Eurocode 8. Furthermore, this classification was compared to several international seismic codes' criteria. It was evaluated the influence of irregularity in elevation classification, seismic standard considered in the design and differences in structural properties.

Secondly, considering the structural design information availability, it was carried out the seismic response assessment of part of the existing buildings under study. It was observed the impact of seismic design consideration or not on structural response in capacity curves, inter-storey and global drifts.

Finally, chapter 7," *Closure and future research*", draws general conclusions about the research and discusses future research that could be explored for a better understanding of structural irregularities impact in RC structures' seismic response.

1.4 REFERENCES

[1] Maranhão, H. J. de O. 2024. "Seismic Design of Reinforced Concrete Buildings According to the Modern Codes." Porto: Faculty of Engineering of the University of Porto. Accessed September 10, 2024. https://hdl.handle.net/10216/160480.

[2] Braga, F., V. Manfredi, A. Masi, A. Salvatori, and M. Vona. 2011. "Performance of Non-Structural Elements in RC Buildings during the L'Aquila, 2009 Earthquake." *Bulletin of Earthquake Engineering* 9 (1): 307–24. https://doi.org/10.1007/s10518-010-9205-7.

[3] Verderame, G. M., F. De Luca, P. Ricci, and G. Manfredi. 2011. "Preliminary Analysis of a Soft-Storey Mechanism after the 2009 L'Aquila Earthquake." *Earthquake Engineering and Structural Dynamics* 40 (8): 925–44. https://doi.org/10.1002/eqe.1069.

[4] Palermo, M., R. R. Hernandez, S. Mazzoni, and T. Trombetti. 2014. "On the Seismic Behavior of a Reinforced Concrete Building with Masonry Infills Collapsed during the 2009 L'aquila Earthquake." *Earthquake and Structures* 6 (1): 45–69. https://doi.org/10.12989/eas.2014.6.1.045.

[5] Varum, H., J. Melo, A. Furtado, and A. Lima. 2022. "Irregularities in RC Buildings: Perspectives in Current Seismic Design Codes, Difficulties in Their Application and Further Research Needs." In *Seismic Behaviour and Design of Irregular and Complex Civil Structures IV*, edited by Rita Bento, Mario De Stefano, Dietlinde Köber, and Zbigniew Zembaty. Vol. 50. Geotechnical, Geological and Earthquake Engineering. Cham: Springer International Publishing. https://doi.org/10.1007/978-3-030-83221-6.

[6] Jara, J. M., E. J. Hernández, B. A. Olmos, and G. Martínez. 2020. "Building Damages during the September 19, 2017 Earthquake in Mexico City and Seismic Retrofitting of Existing First Soft-Story Buildings." *Engineering Structures* 209 (April). https://doi.org/10.1016/j.engstruct.2019.109977.

[7] Dutta, S. C., and R. Roy. 2012. "Seismic Demand of Low-Rise Multistory Systems with General Asymmetry." *Journal of Engineering Mechanics* 138 (1): 1–11. https://doi.org/10.1061/(asce)em.1943-7889.0000303.

[8] Das, P. K., S. C. Dutta, and T. K. Datta. 2021. "Seismic Behavior of Plan and Vertically Irregular Structures: State of Art and Future Challenges." *Natural Hazards Review*. https://doi.org/10.1061/(ASCE)NH.1527.

[9] Hoerner, J. B. 1971. "*Modal Coupling and Earthquake Response of Tall Buildings*." Pasadena: California Institute of Technology.

[10] Housner, G. W. 1941. "*An Investigation of the Effects of Earthquakes on Building*." Pasadena: California institute of Technology.

[11] Kan, C. L., and A. K. Chopra. 1979. "Linear and Nonlinear Earthquake Responses of Simple Torsionally Coupled Systems." Berkeley. Accessed January 14, 2022. https://nehrpsearch.nist.gov/static/files/NSF/PB298262.pdf.

[12] Kan, C. L., and A. K. Choprat. 1977. "Elastic Earthquake Analysis of Torsionally Coupled Multistorey Buildings." *Earthquake Engineering and Structural Dynamics* 5: 395–412.

[13] Rosenblueth, E. 1979. "Seismic Design Requirements in a Mexican 1976 Code." *Earthquake Engineering and Structural Dynamics* 7: 49–61.

[14] Poole, R. A. 1977. “Analysis for Torsion Employing Provisions of NZRS 4203:1974.” *Bulletim of New Zealand Society for Earthquake Engineering*.

[15] Goel, R. K., and A. K. Chopra. 1993. “Seismic Code Analysis of Building without Locating Centers of Rigidity.” *Journal of Structural Engineering* 119 (10): 3039–55.

[16] EN 1998-1. “Eurocode 8: Design of Structures for Earthquake Resistance – Part 1: General Rules, Seismic Actions and Rules for Buildings.” European Comittee for Standardization: Brussels, Belgium, 2004.

[17] prEN 1998-1-2. “Eurocode 8: Design of Structures for Earthquake Resistance - Part 1-2: Rules for New Buildings, Draft Post-ENQ 48th Meeting CEN/TC250/SC8.” European Comittee for Standardization: Brussels, Belgium, 2024.

1.5 NOMENCLATURE

CM	Centre of mass
CS	Centre of stiffness
DCM	Ductility class medium
MRF	Moment-resisting frame
q_R	Behaviour factor component: considers overstrength of structure due to redistribution of action effects in redundant structural systems
q_S	Behaviour factor component: considers overstrength of structure due to all other sources, except to, redistribution of action effects in redundant structural systems
q_D	Behaviour factor component: consider the deformation and energy dissipation capacity
RC	Reinforced concrete

2 Code requirements for the seismic design of irregular elevation RC structures

Santos, D., J. Melo, and H. Varum. 2024. "Code Requirements for the Seismic Design of Irregular Elevation RC Structures." *Buildings* 14 (5): 1351. https://doi.org/10.3390/buildings14051351.

2.1 Abstract

The recent seismic activity highlights the crucial need to enhance seismic design and safety assessment methods, particularly for irregular structures, in both new and existing constructions. The present study focuses on structural irregularities in elevation for buildings, as the design of structural systems involves multiple variables that often result in irregularities in many buildings. This work aims to perform a comparative assessment of the criteria adopted for the evaluation of the structural irregularities in elevation present in European and international seismic codes. This paper is structured as follows: Firstly, it discusses structural irregularities and more specifically the most common types of structural damage due to seismic events. Then, it shows the documented experiences of structural damages in seismic events associated with structural irregularities in China, Italy, Spain, Nepal and Mexico. Additionally, it discusses the requirements of the standards on irregularities and their limitation in that matter. At the end of this section, the different approaches of each code in irregularities in elevation are compared. All assessed seismic codes addresses the structural irregularity issue, attributing the desired characteristics of a seismic-resistant structure. However, there are considerable development differences between norms, demonstrated on ambiguity of few codes on criteria of vertical irregularities.

2.2 Introduction

Building performance during strong earthquakes has been observed and has been used to instruct engineers and builders on how to properly design and construct seismic load-resisting structures [1–5]. In regions characterised by prolonged inhabitation and frequent seismic activity, the refinement of design methodologies has yielded satisfactory structural performance outcomes. Notwithstanding their limited applicability owing to regional differences, the study of these procedures offers valuable insights for structural engineers [6].

Several structural damage mechanisms have been recurrently identified as inappropriate structural configurations, including torsion induced by asymmetrical masonry infills, first soft-storey buildings and deficiencies in reinforcement detailing [7].

Concerning the floor deformation influence on structural behaviour, Ruggieri et al. [8] disclosed that the type of diaphragm notably impacts the likelihood of structural damage. This could also be effective, considering evolution in height, where the de-formability reduces on the higher storeys.

Regarding the effect of the irregularity on the demand amplification along the height of the buildings, Ruggieri and Vukobratović [9] concluded that for low-rise buildings, in the perpendicular direction of the analysis, the torsion induces a further acceleration component, which heightened with the floor eccentricity. The results were related to the floor eccentricity, the elevation of the considered node and, regarding the nonlinear field, the ductility and the yielding force became significant. Similar results have been taken from ref. [10].

Despite extensive research endeavours dedicated to assessing the seismic behaviour of asymmetric structures, relatively less effort has been undertaken toward the various types of irregular structures [11]. This study emphasised the reduced research focusing on structures exhibiting irregularities along both principal directions. Some authors suggest that the implementation and adherence to seismic design codes are crucial factors in mitigating earthquake community risk [12]. Further research is required on soil movements, the deterioration of the material properties, the efficacy of the shear walls positioning and many other related issues to perform a better seismic risk assessment.

In the paper, the following seismic codes were assessed:

1. EN 1998-1:2005 (Europe);
2. prEN 1998-1-2:2024 (Europe);
3. NBR 15421:2023 (Brazil);
4. NTC-RSEE from Mexico City (Mexico);
5. Manual de diseño obras civiles: diseño por sismo (Mexico);
6. ASCE/SEI7-22—minimum design loads for buildings and other structures (USA);
7. Diseño sísmico de edifícios (NCh 433) (Chile);
8. NZS 1170-5 structural design actions (New Zealand).

Regarding the irregularity in elevation criteria, it was referenced quantitatively, qualitatively or not referenced on each seismic code. The following criteria are considered in the seismic codes comparisons:

1. Continuity of structural elements;
2. Variation of lateral stiffness;
3. Existence of weak floors (soft storey);
4. Mass variation;
5. Ratio between actual and required resistance;
6. Geometry criteria;
7. Resistance variation (weak storey);
8. Storey area variation;
9. Ratio between elevation and smallest dimension in plan.

2.3 STRUCTURAL IRREGULARITIES IN ELEVATION

A proper structural conception is essential to ensure optimal performance under any loading conditions. Regular structures with redundant resistant systems for horizontal load demands typically exhibit better behaviour. Conversely, complex structural systems often present deficiencies in the dimensions and detailing of structural elements [6].

Over the past decade, numerous researchers have worked on the seismic vulnerability analysis of irregular building structures. Mouhine and Hilali [13] scrutinised the seismic vulnerability of building structures characterised by vertical geometric irregularities positioned at various heights along the buildings. Their findings underscored the significant impact of setback percentage and its placement on both the performance and lateral stiffness of these structures. Setback causes the non-uniform distribution of mass and stiffness in the vertical direction, which may have a significant influence on seismic response [14].

Similarly, Ruggieri and Uva [15] conducted a study focusing on the performance evaluation of eight low-rise reinforced concrete building structures, of which seven featured in-plane irregularities. Using pushover analysis, they highlighted the influence of seismic motion spatial variability on the structural seismic behaviour. Furthermore, Men et al. [16] explored the seismic vulnerability of RC structures exhibiting vertical irregularities. Their research emphasised the interplay between seismic intensity, structural performance levels and the vulnerability of such structures.

Irregularities in elevation can affect different characteristics of the structural system, such as stiffness, mass and strength. These can come from a variety of sources, including the following:

1. Stiffness irregularity—soft storey: Checked when there are variations of stiffness from one floor to another, resulting in a weak floor, which may give rise to mechanisms such as soft storey [17]. The lateral stiffness of buildings is also strongly affected by the presence of setbacks [13,18,19];
2. Strength irregularities—weak storey:
   i. Due to the existence of different heights of the constituent parts of the building and different heights between buildings: occur when adjacent buildings have different heights, resulting in restrictions on the movements of the lower floors [20];
   ii. Irregularities in the bearing capacity of a floor: due to the different resistance of the elements that support the action seismic activity in a direction on a given floor, in relation to the next floor [21];
   iii. Irregularities due to discontinuities in the load paths: verified with the absence of continuity of the resistant elements from one floor to the next [22].
3. Mass irregularities: Checked when the mass of a floor is much higher or lower than the others. An example of a practical aspect of this situation is the need for floor technicians with heavy machinery [21].

Vertical irregularity is characterised by abrupt shifts in mass, stiffness and strength along the height, leading to soft/weak storey behaviour. These storeys are susceptible to an earlier collapse by concentration in element forces and ductility demands [23]. However, these discontinuities, treated by current seismic codes as vertical irregularities, undoubtedly do not lead to increases in plastic demands and poor seismic performance [24,25].

The most common type of possible damages due to seismic events, based on the published work about this matter, are listed below [6]:

1. Stirrups and hoops: These stem from inadequate detailing, insufficiency or absence of transverse reinforcement, poor detailing of hoops and wide spacing between stirrups;
2. Longitudinal reinforcement detailing (bond, anchorage and lap splices): Smooth bars reduce the strength capacity of RC elements. Significant deterioration of the bond conditions is observed along the longitudinal bars, exceeding and deviating the plane sections theory;
3. Shear and flexural capacity of elements;
4. Inadequate shear capacity of structural joints;
5. Strong-beam weak-column mechanism;
6. Short-column mechanism;
7. Irregularities in plan and/or in elevation;
8. Pounding;
9. Damages in secondary elements;
10. Damages in non-structural elements.

Furthermore, in the conclusions of the published work about the assessment of modern seismic codes about the structural efficiency in earthquake events by Athanatopoulou and Manoukas [26], it is highlighted:

1. Torsional sensitivity of a building is avoided only if a minimum torsional radius is ensured. However, this condition does not imply that adequate static torsional stiffness is available either;
2. In general, displacement or drift ratios are not reliable either as torsional sensitivity criteria or as a measure of displacement amplification at the perimeter of buildings. Therefore, a revision of the relevant code provisions should be examined.

It is necessary for a constant assessment and update of seismic codes to ensure structural safety during those events.

2.4 DAMAGES ASSOCIATED WITH STRUCTURAL IRREGULARITIES

The seismic behaviour of RC structures is closely tied to the characteristics of their structural elements, including stiffness, ductility, strength and energy dissipation capacity. The overall strength is a result of the combined effort of individual structural components and their interactions. Properly designing and detailing the connections between these elements is crucial, as inadequate transverse reinforcement has led to various failures, both at local and global scales. In the case of structures with substantial redundancy, their seismic performance hinges on their ability to redistribute loads effectively [27].

The detailing of the RC elements stands out as a prevalent cause of damage documented in recent seismic events. The confinement effect from the transverse reinforcement, related to the spacing and diameter of the stirrups, longitudinal reinforcement arrangement and quality of the steel employed, is crucial in preventing concrete detachment. Based on the observations made in situ, especially on Sichuan (China) [28–31], L'Aquila (Italy) [3,32], Lorca (Spain) [33], Emilia (Italy) [34–36], Gorkha (Nepal) [37–39], Puebla (Mexico) [5,40] and Eastern Anatolia region (Turkey) [4], it can be inferred that this approach was not implemented in buildings constructed during earlier decades.

Another common type of damage noticed in recent earthquakes is related to insufficient shear capacity of RC elements. The shear strength limit is before the onset of yielding, indicating a constraint on energy dissipation. Generally, corner columns encounter challenges associated with their shear strength capability and inadequate confinement, particularly when the building's structure features disparities between its mass and stiffness centres. Figures 2-1–2-3 present examples of structural damages due to seismic events.

(**a**) (**b**) (**c**)

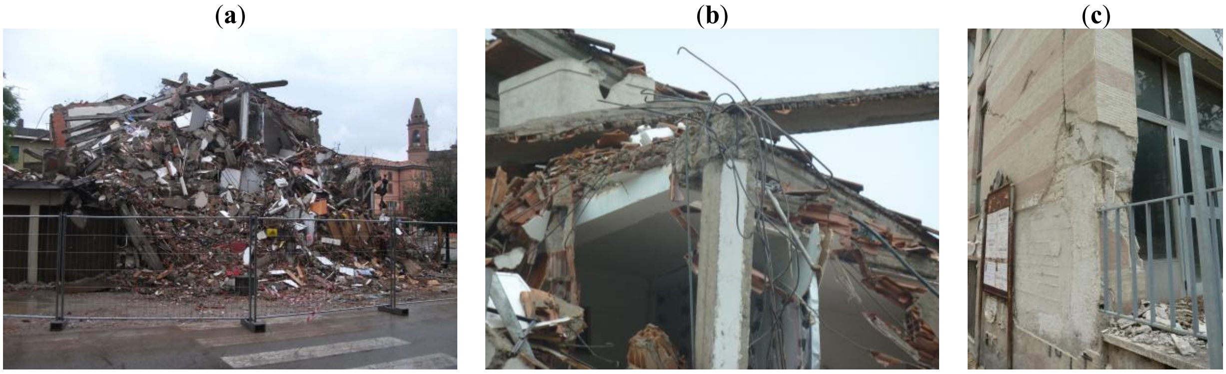


Figure 2-1: Emilia, Italy—2012: (a) RC structure collapsed in Cavezzo; (b) inadequate reinforcement joint de-tail; (c) damage of columns due to inadequate detailing for confinement

(a) **(b)**

Figure 2-2: L'Aquila, Italy—2009: (a) damages on RC structure in L'Aquila; (b) node damage of reinforced concrete elements

Ductile behaviour should be the premise of the seismic design of columns and beams. This strategy involves a potential reduction in demand within the compression zone and/or enhancing capacity. This can involve restricting axial loading, enlarging cross-sectional areas or controlling tensile reinforcement while balancing forces with compression loads. Furthermore, strengthening the compression region can be achieved by employing higher quality concrete and effective confinement [27].

(a) **(b)** **(c)**

Figure 2-3: Lorca, Spain—2011: (a) building collapse after the earthquake; (b) building with a soft-storey mechanism; (c) column failure due to inadequate detailing of the transverse reinforcement

Among the paramount concerns regarding the seismic behaviour of RC structures is the interaction of the masonry infill walls with the structural components. These infill walls can alter the lateral stiffness, strength, ductility and energy dissipation capacity of buildings. As a result, changes in the natural frequencies of the building and its corresponding vibration mode may occur, leading to seismic action surpassing the anticipated levels of the design. This phenomenon can elevate the seismic vulnerability of the buildings.

The distribution of infill walls, typically determined by the architectural team, can significantly influence the global seismic performance of a building, notably when the building is irregular in height and/or in plan. Considerable local failures have been documented in the seismic performance of masonry infill walls, attributed to in-plane, out-of-plane and combined seismic loadings.

2.5 Discussion and Limitation of Code Requirements on Irregularities

2.5.1 EN 1998-1:2004—Europe

Building structures are classified by Eurocode 8 [41] as regular and non-regular, in plan and in elevation. This distinction has implications for the following aspects of seismic design, as demonstrated in Table 4.1 of Eurocode 8 [41]:

1. The structural model: a planar model or a spatial model;
2. The method of analysis: response spectrum analysis or a modal analysis;
3. The value of the behaviour factor “q” must be reduced by 20% for buildings that are not regular in elevation and it also reduces the presence of irregularities in plan, depending on the structural system and the ductility class, as explained in 5.2.2.2 (6) of Eurocode 8 [41].

A building classified as regular in elevation must satisfy all the conditions indicated in the following paragraphs [41]:

1. All systems resistant to lateral actions, such as cores, load-bearing walls or frames, are continuous from the foundation to the top of the building, or, if there are floor setbacks at different heights, to the top of the area considered in the building;

2. The lateral stiffness and the mass of each floor remain constant or show a gradual reduction, without abrupt changes, from the base to the top of the considered building;
3. In buildings with a frame structure, the ratio between the actual floor strength and the strength required by the design must not vary disproportionately between adjacent floors. In this context, the particular aspects of frame structures with masonry infills are dealt with in 4.3.6.3.2 of this code;
4. When the building has setbacks, the following additional conditions must be applied:
   i. In the case of successive setbacks that maintain an axial symmetry, the setback on any floor must not exceed 20% of the plan dimension of the lower level in the direction of the setback;
   ii. In the case of a single setback placed in the lower 15% of the overall height of the primary structural system, it must not exceed 50% of the plan dimension of the lower level. In this case, the structure of the lower area located inside the vertical projection of the upper floors must be calculated to resist at least 75% of the horizontal force that would act at that level in a similar building without widening the base;
   iii. In the case of non-symmetrical setbacks, the sum, on each side of the setbacks of all floors, must not exceed 30% of the dimension in plan at floor level above the foundation or above the upper level of a rigid basement, and each setback must not be more than 10% of the dimension in plan of the lower level.

The European standard presents three levels of detailing of concrete structures de pending on the actions applied: DCL, DCM and DCH levels (low, medium and high capacity to dissipate energy, respectively) [41]. When there is no DCL detail specification, Eurocode 2 [42] recommendations are used.

2.5.2 PREN 1998-1-2:2024—EUROPE

The second generation of Eurocode 8 is currently ongoing to improve and update the regulation. Among the changes, such as restructuring the standard and increasing content relevant to the topic, this topic aims to analyse the differences regarding the treatment of irregularities between the two versions, the current one and its draft version (draft).

The prEn 1998-1-2:2024 [43], like the current version, maintains the same classification of each unit regardless of the regular and irregular structures. The criteria for regularity in elevation present some changes which stand out, as follows:

1. As for the lateral stiffness and the mass of each floor, while in the current version, these factors remain constant or have small gradual reductions in height (without specifying a percentage), in the draft version, these reductions should not exceed, respectively, 30% and 150% in relation to the floor bottom, without sudden changes from consecutive storeys. In the case of mass, that difference is considered until one storey below the top storey;

2. Regarding the ratio between the real strength of the floor and that required for calculation, in the current version, it is said that this ratio should not vary disproportionately (but without setting a proportion) between the floors of framed structures, whereas in the draft version, it states that this ratio must not exceed 30% and does not refer only to framed structures;
3. The current version addresses the presence of setbacks and characterises additional conditions in the treatment of buildings, whereas the version under study also includes such a requirement.

2.5.3 NBR 15421:2023—PROJETO DE ESTRUTURAS RESISTENTES A SISMOS—PROCEDIMENTO—BRAZIL

This code defines seismic categories exclusively in terms of their seismic zones, as shown in 7.3. These categories are used to establish the allowed seismic-resistant structural systems, the limitations on irregularities of the structures, the components of the structure that must be designed in terms of seismic resistance and the types of seismic analysis (methods of calculation) that can be adopted [44].

This code allows the use of three calculation methods for seismic analysis: the method of equivalent horizontal forces (for seismic categories A, B and C), the spectral method (for seismic categories B and C observing the limitation of pavements and structural irregularities) and the method of historical accelerations in time (for seismic categories B and C for seismic categories B and C observing the limitation of pavements and structural irregularities) [44].

Building structures are classified by NBR 15421 [44] as regular and non-regular, in plan and in elevation. Structures showing one or more of the irregularity types listed below must be designed as having structural irregularities in elevation [44]:

1. Type4: Discontinuities in the vertical seismic resistance path, such as consecutive vertical resistance elements in the same plane but with axes greater than their length apart, or when resistance between consecutive elements is greater in the upper element;
2. Type 5: Characterisation of an "extremely weak pavement" as one in which its lateral resistance is less than 65% of the resistance of the pavement immediately above. The lateral resistance is computed as the total resistance of all earthquake-resistant elements present in the considered direction. In case the seismic forces are not multiplied by the over resistance coefficient Ω_0, structures cannot be more than two stories and not more than 9 m of elevation.

It addresses that columns, beams, slabs and trusses supporting porticos and seismic resistant wall columns that present Type 2 or Type 4 irregularity should be designed considering the seismic effects in the vertical direction (E_v) and the consequences of the horizontal earthquake with the effect of over resistance (E_{mh}), according to 8.4 of this code [44]. If the structure presents structural irregularity in plan types 1, 2 or 3, a three-dimensional model should be used. In this model, each node must have at least three degrees of freedom, two translations in a horizontal plane and one rotation around a vertical axis [44]. Also, in the case of seismic category C structures, in which there is a structural irregularity in plan type 1, the accidental torsional moments M_{ta} at each elevation should be multiplied by the torsional amplification factor Ax [44].

This code does not deal with the structural detailing drawing of structures subjected to seismic actions, leaving all discussion about the structural detail drawing for Code NBR6118 [45], which does not specifically address the matter considering seismic actions. This is a limitation of those codes regarding seismic actions on structural detailing matter when compared, for instance, with Eurocode 8.

2.5.4 NTC-RSEE FROM MEXICO CITY—MEXICO

The Mexican code from the Public Administration of Mexico City classifies the building structures as regular, irregular and strongly irregular, without distinction in criteria of irregularity in plan and in elevation. A structure is classified as regular when it satisfies the requirements indicated below [46]:

1. The different walls, frames and other earthquake-resistant vertical systems are visibly parallel to the main orthogonal axes of the building. An earthquake-resistant plane or element shall be visibly parallel to one of the orthogonal axes when the angle it forms in plan with the axis in question does not exceed 15 degrees;
2. The ratio between its height and its smallest size in plan is not greater than 4;
3. The ratio of length to width in plan is no more than 4;
4. In plan has no setbacks or projections of dimensions greater than 20% of the dimension in plan measured parallel to the direction in which the setback or projection is considered;
5. Each level has a floor system whose rigidity and strength in its plan satisfy the requirements for a rigid diaphragm;
6. The floor system has no openings that, at some level, exceed 20% of its flat area on that level, and the empty areas do not differ in position from one floor to another. This requirement does not apply to the roof;
7. The weight of each level, including the overload that must be considered for the seismic project, is not greater than 120% of that corresponding to the floor immediately below;
8. In each direction, no floor has a floor dimension greater than 110% of the dimension of the floor immediately below. In addition, no floor has a floor dimension greater than 125% of the smallest of the lower floor dimensions in the same direction;
9. All columns are restricted on all floors in both directions of analysis by horizontal diaphragms or by beams. Consequently, no column passes through a floor without being attached to it;
10. All the columns of each floor have the same height, although it can vary from one floor to another. Except for this item, the top floor of the building;
11. The lateral stiffness of no floor differs by more than 20% from that immediately below. Except for this item, the top floor;

12. On no floor does the lateral displacement of any point on the floor exceed the average lateral displacement of the floor ends by more than 20%;
13. In systems designed for “Q” (coefficient of seismic behaviour) equal to 4, the ratio of lateral strength and design action shall be higher than 85% of the average of these ratios for all floors. In systems designed for “Q” equal to or less than 3, on no floor shall the ratio indicated above be less than 75% of the average of these ratios for all floors. To verify compliance with this requirement, the resistance capacity of each floor will be calculated considering all the elements that can contribute significantly to it. The top floor is excluded from this requirement.

A structure is classified as irregular when it does not satisfy one of the requirements “5”, “6”, “9”, “10”, “11”, “12” and “13” or does not satisfy two or more of the requirements “1”, “2”, “3”, “4”, “7” and “8”, as listed above. A structure is considered strongly irregular if it does not meet two or more of the requirements “5”, “6”, “9”, “10”, “11”, “12” and “13” listed above, or if any of the following conditions are met:

1. The lateral displacement of any point of one of the plants exceeds by more than 30% of the average of the displacements of the extremities;
2. The lateral stiffness or shear strength of any floor exceeds that of the floor immediately below by more than 40%. To verify compliance with this requirement, the strength and lateral stiffness of each floor will be calculated considering all the elements that can contribute significantly to them;
3. More than 30% of the columns located on one floor do not meet requirement “9” listed above.

2.5.5 MANUAL DE DISEÑO OBRAS CIVILES: DISEÑO POR SISMO—MEXICO

The code developed by the Mexican Federal Electricity Commission (CFE) classifies the building structures as regular, irregular and strongly irregular, without distinction in criteria of irregularity in plan and in elevation. A structure is classified as irregular when it does not satisfy three of the requirements indicated below [47]:

1. The distribution in plan of the mass, walls and other resistant elements of the building must be symmetrical with respect to two orthogonal axes;
2. The ratio of elevation and smallest dimension in plan cannot be greater than 2.5;
3. The ratio between the length and width of the base cannot exceed 2.5;
4. Existing setbacks and overhangs cannot exceed 20% of the dimension in plan, measured parallel to the direction in which the setback or overhang is located;
5. On each floor, there must be a rigid diaphragm behaviour;
6. There cannot be any opening in the slab that exceeds 20% of the dimension in plan, measured parallel to the opening. Hollow zones must not appear staggered between adjacent floors and the total opening area must not exceed 20% of the floor area;

7. The mass of each floor cannot exceed 110% of the corresponding floor immediately below, except for the top floor, nor may it be less than 70% of the mass of the floor immediately below. The top floor is exempt from this condition.
8. No floor may have an area greater than 110% nor less than 70% of the floor area immediately below;
9. All columns are restricted by floors, in orthogonal directions, by horizontal diaphragms;
10. Stiffness cannot differ between floors by more than 50% from that of the floor immediately below;
11. On no floor can the statically calculated torsional eccentricity exceed 10% of the plan dimension of that floor, measured parallel to the mentioned eccentricity.

It can also be classified as strongly irregular if it meets any of the following requirements [47]:

1. The statically calculated torsional eccentricity exceeds 20% of the floor plan dimension measured parallel to the mentioned eccentricity;
2. The rigidity or shear resistance of the tread exceeds more than 100% that of the tread immediately below;
3. At the same time, fulfil the conditions “10” and “11” described above. Does not fulfil four or more regularity conditions described above.

For seismic analysis of building structures, three types of methods can be used [47]: a simplified method, a static method and a dynamic method. The simplified method is simpler to use. However, it is only applicable to regular structures that meet all the requirements indicated by the method. The static method is more suitable for regular structures with height conditioned by the type of terrain where they are inserted. The dynamic method can be used on any structure, regardless of its characteristics.

According to the code [47], the structure must be analysed under seismic action considered in two independent orthogonal directions. As a combination of the effects of seismic action in two orthogonal directions, the standard proposes that the value of the action in each direction is increased by 30% of the value of the corresponding orthogonal direction.

2.5.6 ASCE/SEI7-22—MINIMUM DESIGN LOADS FOR BUILDINGS AND OTHER STRUCTURES—USA

The American regulation [48] classifies structures as regular or irregular in elevation according to a set of criteria related to the structural configurations of the building that result in a classification divided by types.

This code also divides types according to a Seismic Project Category, which is a classification assigned to a structure based on its occupation category and the seismicity of that area. Where, as indicated on the (ISAT) web pages, the category assignment can vary from A-F and can be defined as follows:

1. Seismic category A: it corresponds to buildings located in areas where little seismic activity is expected and with good soil;

2. Seismic category B: it corresponds to buildings belonging to occupation type I, II or III where seismic activity is expected to be moderate, located on stratified soil consisting of good and bad soils;
3. Seismic category C: it corresponds to buildings belonging to occupation type IV where seismic activity is expected to be moderate, or buildings belonging to occupation type I, II or III where seismic activity is expected to be more severe;
4. Seismic Category D: it corresponds to buildings located in areas where they are expected to experience severe and destructive seismic action, but not located close to a major fault;
5. Seismic category E: it corresponds to buildings belonging to occupation type I, II or III located a short distance from large active faults;
6. Seismic category F: it corresponds to buildings belonging to occupation type IV located a short distance from major active faults.

Further, the type of occupation can be defined as follows:

1. Type of occupancy I: buildings that pose a small risk to human life in the event of failure;
2. Type of occupancy II: buildings that do not fit into any of the other types of occupancy;
3. Type of occupancy III: buildings that pose a substantial risk to human life and in the event of failure;
4. Type of occupation IV: buildings considered essential for human life.

To classify structures as regular or irregular in elevation, ASCE/SEI7-22 [48] defines structures as follows:

1. Type 1a—Stiffness–soft storey irregularity: it exists where there is a floor in which the lateral stiffness is below 70% of that in the storey above or below 80% of the mean stiffness of the three floors above;
2. Type 1b—Stiffness–extreme soft storey irregularity: it exists where there is a floor in which the lateral stiffness is below 60% of that in the storey above or below 70% of the average stiffness of the three floors above;
3. Type 2—Vertical geometric irregularity: it exists where the horizontal dimension of the seismic force-resisting system in any floor exceeds 130% of that in an adjacent storey;
4. Type 3—In-plane discontinuity in vertical lateral force-resisting element irregularity: it occurs where there is an in-plane offset of a vertical seismic force-resisting element, leading to overturning demands on supporting structural elements;
5. Type 4a—Discontinuity in lateral strength–weak storey irregularity: it occurs where the storey lateral strength is below 80% of that in the floor above;
6. Type 4b—Discontinuity in lateral strength extreme weak storey irregularity: it occurs where the storey lateral strength is below 65% of that in the storey above.

This code [48] also provides two exceptions on vertical structural irregularities of types 1a, 1b and 2 listed below:

1. They do not apply where no storey drift ratio, regarding design lateral seismic force, is higher than 130% of the next floor above the storey drift ratio. Torsional effects are excluded from the computation of storey drifts. Assessment of the storey drift ratio for the upper two storeys is unnecessary;
2. These considerations are not mandatory for single-storey buildings in any Seismic Design Category nor for two-storey buildings categorised under Seismic Design Categories B, C or D.

Furthermore, this code provides constraints and additional requirements for systems with structural irregularities:

1. Prohibited vertical irregularities for seismic design categories D through F: Structures designated under Seismic Design Category E or F with vertical irregularities Type 1b, 4a, 4b or Seismic Design Category D structures with irregularity in elevation Type 4b are not allowed. However, it is permitted if the E and F Seismic Design Category structures present irregularity in elevation Type 4a where the storey lateral strength is not less than 80% of the floor above;
2. Extreme weak storeys: Structures featuring a vertical irregularity Type 4b must not exceed 2 storeys or 9 m in structural height. However, this limit does not apply if the "weak" storey can resist a total seismic force equivalent to Ω0 times the design force;
3. Elements supporting discontinuous walls or frames: Structural elements carrying discontinuous walls or structural frames with horizontal and/or vertical irregularity Type 4 must be designed to resist the seismic load effects, considering overstrength. The connections of these elements to the supporting members must be sufficient to transmit the forces for which they were designed;
4. Increase in forces as a consequence of irregularities for seismic design categories D through F: For structures categorised under Seismic Design Category D, E or F and presenting a horizontal structural irregularity of Type 1a, 1b, 2, 3 or 4 or a vertical structural irregularity of Type 3, the design forces for some components of the seismic force-resisting system, such as connections of diaphragms to vertical elements and collector and connection to vertical elements, must be increased 25%.

Forces calculated using the seismic load effects, including overstrength, do not re quire augmentation.

This code [48] allows structures to be analysed using several methods. However, the model must incorporate the stiffness and strength characteristics of components that are significant to forces and deformation distributions within the structure. It also has to represent the spatial distribution of mass and stiffness throughout the structure. Additionally, it should consider, for RC structures, the stiffness properties of concrete and masonry elements regarding the effects of cracked sections.

2.5.7 DISEÑO SÍSMICO DE EDIFÍCIOS (NCH 433)—CHILE

The seismic assessment in Chilean standards is provided by the following:

1. NCh 433:1996 [49], modified in 2012—Diseño sísmico de edifícios;
2. NCh 2369:2003 [50] - Diseño sísmico de estructuras e instalaciones industriales;
3. NCh 2745:2013 [51] - Análisis y diseño de edificios con aislación sísmica;
4. NCh 3411:2017 [52] - Diseño sísmico de edificios con sistemas pasivos de disipación de energia;
5. NCh 3357:2015 [53] - Diseño sísmico de componentes y sistemas no estructurales.

The Chilean code NCh 433 does not explicitly address structural irregularities. It only mentions that irregular buildings can only be designed as a single structure when the diaphragms are calculated and built so that the structure behaves during earthquakes as a single one.

This code suggests a separation of the building into regular parts (INN 2012). This code also highlights two things [49]:

1. If an irregular building in plan is designed as a single structure, special care must be taken in dimensioning the connections between the different parts that make up the floor;
2. In floors where there is rigidity discontinuity in the resistant planes or other vertical substructures, it must be verified that the diaphragm is able to redistribute forces.

Regarding structural analysis methods, the code allows a static analysis or a spectral modal analysis, depending on certain geometric conditions of the structure [49]. It presents two response modification factors, one for equivalent static analysis (R) and another for spectral analysis (R_0). This factor reflects the energy absorption and dissipation characteristics of the resistant structure, as well as the experience in the seismic behaviour of the different types of structures and materials used.

2.5.8 NZS 1170.5 STRUCTURAL DESIGN ACTIONS—NEW ZEALAND

The New Zealand Code addresses structural irregularity and encourages structural designers to design regular structures due to seismic behaviour. Any structure is considered irregular as long as it meets one of the irregularity criteria present in the code. Single-storey houses or roofs weighing less than 10% of the level below will not be considered when applying these criteria [54].

A structure might be considered irregular in elevation if, according to the New Zealand Code [50]:

1. One floor significantly exceeds the height of adjoining storeys, leading to a reduction in stiffness;
2. The ratio of mass to stiffness in adjoining storeys differs significantly;

3. The smaller dimension was below the larger dimension, resulting in an inverted pyramid effect;
4. The structure presents, at one or more levels, substantial horizontal offsets in the vertical elements of the horizontal force-resisting system, even though the structure has a symmetrical geometry about the vertical axis;
5. The structure exhibits abrupt changes in strength capacity between storeys, resulting in the concentration of energy demand in the resisting elements of a specific storey.

The method of seismic analysis is conditioned by the presence of irregularities and the method of equivalent forces is indicated only for regular structures that comply with certain conditions. For irregular structures, the modal response spectrum method or historical accelerations in time is used.

2.5.9 Comparative analyses of the studied codes

Table 2-1 presents a summary of the regularity criteria in elevation addressed by the analysed codes. Table 2-2 presents the reduction of the behaviour coefficient due to irregularities considered in each code standard studied.

Table 2-1: Synthesis of regularity criteria in elevation

Criteria	Europe	Europe (Draft)	Brazil	Mexico NTC-RSEE	Mexico CFE	USA	Chile	New Zealand
Continuity of structural elements	R	R	R	N	N	R	N	N
Variation of lateral stiffness	R	Q	Q	Q	Q	Q	N	Q
Existence of weak floors (soft storey)	N	N	Q	N	N	Q	N	N
Mass variation	R	Q	N	Q	Q	N	N	Q
Ratio between actual and required resistance	Q	Q	N	N	N	N	N	N
Geometry criteria	Q	R	N	Q	Q	Q	N	Q
Resistance variation (weak storey)	N	N	R	N	Q	Q	N	Q
Storey area variation	N	N	N	Q	Q	N	N	N
Ratio between elevation and smallest dimension in plan	N	N	N	Q	Q	N	N	N

N: no reference; R: referenced, but without quantification criteria; Q: presents quantification criteria.

Table 2-2: Expected reduction in the behaviour factor (q)/structural response modification coefficient (R-factor) due to irregularity in elevation

Region	Behaviour factor/R-Factor
Europe	A 20% reduction in the presence of irregularities in elevation. Reduction in the presence of irregularities in the plan, depending on the structural system and of the ductility class, as explained in 5.2.2.2 (6) of this standard. With this reduction, the structure must be further strengthened. It can be at the level of the geometry of the elements and/or the amount of reinforcement.
Brazil	R-factor is mentioned, but no reductions in the presence of irregularities are numerically determined.
Mexico (NTC-RSEE)	Seismic behaviour factor Q is mentioned. A 20% reduction in the presence of irregularities. A 30% reduction for heavily irregular structures.
Mexico (CEF)	Seismic factor Q is mentioned. Reduction of 10% when the regularity requirements listed from “a” to “i” are not met. Reduction of 20% when two or more regularity requirements are not met and/or the regularity requirements “j” or “k” are not met. A 30% reduction when the structure is classified as strongly irregular.
USA	The redundancy factor ρ can be 1,0 or 1,3 for the structures in the category of irregularity D through F. This factor reduces the response modification coefficient, R, for less redundant structures.
Chile	R-factor is mentioned, but no reductions in the presence of irregularities are numerically determined. Topic 5.5.2.2, however, specifically states that structures with irregularities in plan only can be designed as a single structure when the rigid diaphragm is guaranteed, according to topic 5.5.2.1 of this code. Otherwise, each part should be designed separately, according to topic 5.10 of this code.
New Zealand	The structural performance factor and structural ductility factor are mentioned, but no reductions in the presence of irregularities are numerically determined.

These comparative assessments of those codes relative to elevation regularity criteria led to the following conclusions:

1. The importance given to the mass variation and stiffness in elevation is notorious, as practically all standards contemplate this type of criterion;
2. There are different procedures to verify the criteria of variation of stiffness elevation: some codes make this verification based on the stiffness of the vertical elements on each floor, others through the relationship of the drift between successive floors and, for the latter, there is a need for a previous model;
3. The Chilean code does not address any reference or quantification methods to all regularity criteria in elevation;
4. The European code is the only one that requires that the actual resistance is proportional to the required resistance and that these do not differ too much, which is a difficult criterion to verify;
5. The Mexican code presents two specific criteria which do not appear in the other analysed codes: storey area variation and the ratio between elevation and smallest dimension in plan.

Numerous studies indicate that all the codes that discuss that matter limit themselves from classifying certain common irregularities, suggesting the permissible extent of those typical cases. Nevertheless, it is essential to explore the nature of the high seismic vulnerability to enhance their structural behaviour during seismic events [11].

All assessed seismic codes do not directly incorporate the contribution of infill walls in vertical irregularities criteria, even though several studies demonstrate this phenomenon. However, for stiffness and mass irregularities, it is considered, indirectly by the impact of absence of infill walls in any floor, that contribute to soft-storey behaviour or the infill wall mass contribution to the shift of that parameter between consecutive floor.

The EC8 is the most demanding standard because it is not enough to meet one of the regularity criteria for the coefficient of behaviour to be affected. In other codes, the correction of the behaviour coefficient depends on the type of criterion that is violated.

The EC8, the American and Mexican standards adopt the reduction of the coefficient of behaviour or R-Factor in irregular buildings, ranging between 20% and 30%. The biggest penalty comes in the Mexican code, which can reach 30% in cases of heavily irregular structures.

New Zealand and Brazilian standards have some quantified criteria for regularity in plan and elevation. However, these do not lead to any reduction in the coefficient of behaviour or R-Factor. The Chilean standard has no restrictions on buildings with structural irregularities. In these cases, structural irregularities only have implications for seismic analysis methodology.

When the structure is regular in plan and in elevation, all standards allow a seismic analysis to be carried out by the method of equivalent forces. Differences arise when the structure is classified as irregular, either in plan or in elevation, or both, in which a modal dynamic analysis is predominantly suggested.

Not all the analysed seismic codes present the same degree of development and approach to the theme. However, the first conclusion that can be drawn from the analysis is that all seismic standards address the theme of structural irregularities. However, Chile's standard is the only one that does not have well-established criteria for irregularities. In addition, some norms become confusing, making it difficult to understand what is affected by the existence of irregularities. In addition, although there is a set of criteria that are repeated in different standards, the limits allowed are not always the same in the different standards since these are related to the seismic activity to which the country may be subject.

2.6 SUMMARY AND CONCLUSIONS

The standards indicate a consensus regarding the desired characteristics of a seismic resistant structure, which include simplicity, symmetry, uniformity and redundancies. However, a comparative analysis conducted in this study has revealed significant variations in the representation of seismic hazard.

Eurocode 8 sets out certain criteria that do not involve processes to quantify their influence, which may excessively penalise the final classification for regularity in elevation and in plan and reduce the maximum permissible coefficient of behaviour. It is important to note that the new version of Eurocode 8 provides quantification of some irregularity criteria and clarifies the calculation procedures to address these irregularities.

The draft version of the second generation of Eurocode 8 includes proposed changes that address certain challenges and issues highlighted by the designers. Specifically, the changes aim to mitigate potential errors arising from varied interpretations by introducing quantification for some regularity criteria and verification limits. However, the application and interpretation of these criteria are subject to the designer's sensitivity to the problem, level of experience and knowledge, which may affect the resulting outcomes.

The Mexican codes and the American codes are as strict as Eurocode 8, although they have some differences in the assessments of some irregularity criteria. Both countries are quite traditional in the field of study of seismic engineering, a factor evidenced by the evolution of their standards.

The standards of New Zealand and Brazil incorporate specific criteria to assess regularity in plan and elevation of structures. However, these criteria do not result in any reduction in the coefficient of behaviour or R-Factor. On the other hand, the Chilean standard imposes no restrictions on buildings with structural irregularities. In such cases, these irregularities only impact the seismic analysis methodology.

When a structure is regular in both plan and elevation, all assessed standards permit a seismic analysis using the method of equivalent forces. However, differences arise when the structure is classified as irregular, either in plan, elevation or both, where a modal dynamic analysis is predominantly recommended.

Although not all analysed seismic codes display the same level of development and approach to the subject matter, it is evident that all seismic standards address the issue of structural irregularities. Nevertheless, Chile's standard stands out as the only one lacking well-defined criteria for irregularities. Furthermore, some norms are ambiguous, making it challenging to comprehend the extent of the impact caused by irregularities. Additionally, while certain criteria are repeated across different standards, the allowed limits may vary in accordance with the seismic activity expected in each respective country.

2.7 REFERENCES

[1] Braga, F., V. Manfredi, A. Masi, A. Salvatori, and M. Vona. 2011. "Performance of Non-Structural Elements in RC Buildings during the L'Aquila, 2009 Earthquake." *Bulletin of Earthquake Engineering* 9 (1): 307–24. https://doi.org/10.1007/s10518-010-9205-7.

[2] Verderame, G. M., F. De Luca, P. Ricci, and G. Manfredi. 2011. "Preliminary Analysis of a Soft-Storey Mechanism after the 2009 L'Aquila Earthquake." *Earthquake Engineering and Structural Dynamics* 40 (8): 925–44. https://doi.org/10.1002/eqe.1069.

[3] Palermo, M., R. R. Hernandez, S. Mazzoni, and T. Trombetti. 2014. "On the Seismic Behavior of a Reinforced Con-crete Building with Masonry Infills Collapsed during the 2009 L'aquila Earthquake." *Earthquake and Structures* 6 (1): 45–69. https://doi.org/10.12989/eas.2014.6.1.045.

[4] Vuran, E., C. Serhatoğlu, M. Ö. Timurağaoğlu, E. Smyrou, İ. E. Bal, and R. Livaoğlu. 2024. "Damage Observations of RC Buildings from 2023 Kahramanmaraş Earthquake Sequence and Discussion on the Seismic Code Regulations." *Bulletin of Earthquake Engineering*, 1–30. https://doi.org/10.1007/s10518-023-01843-3.

[5] Tena-Colunga, A. 2021. "Conditions of Structural Irregularity. Relationships with Observed Earthquake Damage in Mexico City in 2017." *Soil Dynamics and Earthquake Engineering* 143 (April). https://doi.org/10.1016/j.soildyn.2021.106630.

[6] Varum, H., J. Melo, A. Furtado, and A. Lima. 2022. "Irregularities in RC Buildings: Perspectives in Current Seismic Design Codes, Difficulties in Their Application and Further Research Needs." In *Seismic Behaviour and Design of Irregular and Complex Civil Structures IV*, edited by Rita Bento, Mario De Stefano, Dietlinde Köber, and Zbigniew Zembaty. Vol. 50. Geotechnical, Geological and Earthquake Engineering. Cham: Springer International Publishing. https://doi.org/10.1007/978-3-030-83221-6.

[7] Jara, J. M., E. J. Hernández, B. A. Olmos, and G. Martínez. 2020. "Building Damages during the September 19, 2017 Earthquake in Mexico City and Seismic Retrofitting of Existing First Soft-Story Buildings." *Engineering Structures* 209 (April). https://doi.org/10.1016/j.engstruct.2019.109977.

[8] Ruggieri, S., F. Porco, and G. Uva. 2020. "A Practical Approach for Estimating the Floor Deformability in Existing RC Buildings: Evaluation of the Effects in the Structural Response and Seismic Fragility." *Bulletin of Earthquake Engineering* 18 (5): 2083–2113. https://doi.org/10.1007/s10518-019-00774-2.

[9] Ruggieri, S., and V. Vukobratović. 2024. "The Influence of Torsion on Acceleration Demands in Low-Rise RC Buildings." *Bulletin of Earthquake Engineering*, March. https://doi.org/10.1007/s10518-024-01873-5.

[10] Landge, M. V., and R. K. Ingle. 2021. "Comparative Study of Floor Response Spectra for Regular and Irregular Buildings Subjected to Earthquake." *Asian Journal of Civil Engineering* 22 (1): 49–58. https://doi.org/10.1007/s42107-020-00297-1.

[11] Das, P. K., S. C. Dutta, and T. K. Datta. 2021. "Seismic Behavior of Plan and Vertically Irregular Structures: State of Art and Future Challenges." *Natural Hazards Review*. https://doi.org/10.1061/(ASCE)NH.1527.

[12] Ainuddin, S., U. Mukhtar, and S. Ainuddin. 2014. "Public Perception about Enforcement of Building Codes as Risk Reduction Strategy for Seismic Safety in Quetta, Baluchistan." *International Journal of Disaster Risk Reduction* 9: 99–106. https://doi.org/10.1016/j.ijdrr.2014.04.007.

[13] Mouhine, M., and E. Hilali. 2022. "Seismic Vulnerability Assessment of RC Buildings with Setback Irregularity." *Ain Shams Engineering Journal* 13 (1). https://doi.org/10.1016/j.asej.2021.05.001.

[14] Georgoussis, G. K. 2011. "Simplified Dynamic Analysis of Eccentric Buildings with a Setback. 1: The Effect of Mass Irregularity." *The Structural Design of Tall and Special Buildings* 20 (8): 911–27.

[15] Ruggieri, S., and G. Uva. 2020. "Accounting for the Spatial Variability of Seismic Motion in the Pushover Analysis of Regular and Irregular RC Buildings in the New Italian Building Code." *Buildings* 10 (10): 1–22. https://doi.org/10.3390/buildings10100177.

[16] Men, J. J., Q. Zhou, and Q. X. Shi. 2009. "Fragility Analysis Method for Vertically Irregular Reinforced Concrete Frame Structures." *Key Engineering Materials* 400: 587–92.

[17] Bento, R., and J. Azevedo. 2000. "Behaviour Coefficient Assessment for Soft Storey Structures." In *Proceedings of the Twelfth World Conference on Earthquake Engineering*, 1–8. Auckland, New Zealand.

[18] Kassem, M. M., F. Mohamed Nazri, L. J. Wei, C. G. Tan, S. Shahidan, and S. S. Mohd Zuki. 2019. "Seismic Fragility Assessment for Moment-Resisting Concrete Frame with Setback under Repeated Earthquakes." *Asian Journal of Civil Engineering* 20 (3): 465–77. https://doi.org/10.1007/s42107-019-00119-z.

[19] Oggu, P., and K. Gopikrishna. 2020. "Assessment of Three-Dimensional RC Moment Resisting Frames under Repeated Earthquakes." *Structures* 26 (August): 6–23. https://doi.org/10.1016/j.istruc.2020.03.039.

[20] Kappos, A. J., and S. G. Scott. 1998. "Seismic Assessment of a R/C Building with Setbacks Using Nonlinear Static and Dynamic Analysis Procedures." In *6th SECED Conference, Seismic Design Practice into the Next Century*, 1–8. Oxford, UK.

[21] Lima, A. G., J. Melo, and H. Varum. 2021. "Structural Irregularities in the Seismic Design of Reinforced Concrete Buildings: Discussion of Regulatory Requirements and Case Studies." *Revista Portuguesa de Engenharia de Estruturas* 3 (16): 75–86.

[22] Das, S. 2000. *Seismic Design of Vertically Irregular Reinforced Concrete Structures*. North Carolina State University.

[23] Alecci, V., and M. De Stefano. 2019. "Building Irregularity Issues and Architectural Design in Seismic Areas." *Frattura Ed Integrita Strutturale* 13 (47): 161–68. https://doi.org/10.3221/IGF-ESIS.47.13.

[24] Stefano, M. De, and B. Pintucchi. 2008. "A Review of Research on Seismic Behaviour of Irregular Building Struc-tures since 2002." *Bulletin of Earthquake Engineering* 6 (2): 285–308. https://doi.org/10.1007/s10518-007-9052-3.

[25] Bhosale, A., R. Davis, and P. Sarkar. 2018. "New Seismic Vulnerability Index for Vertically Irregular Buildings." *ASCE-ASME Journal of Risk and Uncertainty in Engineering Systems, Part A: Civil Engineering* 4 (3): 04018022. https://doi.org/10.1061/ajrua6.0000973.

[26] Athanatopoulou, A. M., and G. E. Manoukas. 2021. "Torsional Sensitivity Criteria In Seismic Codes." *Earthquake and Structures* 21 (1): 1–10. https://doi.org/10.12989/eas.2021.21.1.001.

[27] Varum, H., A. Furtado, and J. Melo. 2022. "Insights on the Seismic Design of Current RC Buildings: Field Lessons, Codes and Research Needs." In *Proceedings of the European Conference on Earthquake Engineering and Seismology*, 4–9, Bucharest, Romania. https://doi.org/10.1007/978-3-031-15104-0_19

[28] Ferreira, M. M. L. de S. A. 2012. "*Risco Sísmico Em Sistemas Urbanos*." Lisboa: Universidade Técnica de Lisboa.

[29] Zhao, C. P., Z. L. Chen, L. Q. Zhou, Z. X. Li, and Y. Kang. 2010. "Rupture Process of the Wenchuan M8.0 Earthquake of Sichuan China: The Segmentation Feature." *Chinese Science Bulletin* 55 (3): 284–92. https://doi.org/10.1007/s11434-009-0425-7.

[30] Liu-Zeng, J., Z. Zhang, L. Wen, P. Tapponnier, J. Sun, X. Xing, G. Hu, et al. 2009. "Co-Seismic Ruptures of the 12 May 2008, Ms 8.0 Wenchuan Earthquake, Sichuan: East-West

Crustal Shortening on Oblique, Parallel Thrusts along the Eastern Edge of Tibet." *Earth and Planetary Science Letters* 286 (3–4): 355–70. https://doi.org/10.1016/j.epsl.2009.07.017.

[31] Costa, A. A., M. A. Ferreira, and C. S. Oliveira. 2010. "O Grande Sismo de Sichuan: Impactos e Lições Para o Futuro." In *8º Congresso de Sismologia e Engenharia Sísmica*, 1–31. Aveiro, Portugal.

[32] Rodrigues, H, X. Romão, A. G. Costa, A. Arêde, H. Varum, J. Guedes, R. Vicente, A. A. Costa, and E. Pauperio. 2010. "Sismo de L`Aquila de 6 de Abril de 2009 Ensinamentos Para Portugal." In *8º Congresso de Sismologia e Engenharia Sísmica*, 1–14.

[33] Romão, X., A. A. Costa, E. Paupério, H. Rodrigues, R. Vicente, H. Varum, and A. Costa. 2013. "Field Observations and Interpretation of the Structural Performance of Constructions after the 11 May 2011 Lorca Earthquake." *Engineering Failure Analysis* 34 (December): 670–92. https://doi.org/10.1016/j.engfailanal.2013.01.040.

[34] Magliulo, G., M. Ercolino, C. Petrone, O. Coppola, and G. Manfredi. 2014. "The Emilia Earthquake: Seismic Performance of Precast Reinforced Concrete Buildings." *Earthquake Spectra* 30 (2): 891–912. https://doi.org/10.1193/091012EQS285M.

[35] Bournas, D. A., P. Negro, and F. F. Taucer. 2014. "Performance of Industrial Buildings during the Emilia Earth-quakes in Northern Italy and Recommendations for Their Strengthening." *Bulletin of Earthquake Engineering* 12 (5): 2383–2404. https://doi.org/10.1007/s10518-013-9466-z.

[36] Novelli, V., D. Alexander, and E. Verrucci. 2012. "The 29th May 2012 Emilia Romagna Earthquake EPICentre Field Observation Report No. EPI-FO-290512." UCL EPICentre: London, UK.

[37] Varum, H., A. Furtado, H. Rodrigues, J. Dias-Oliveira, N. Vila-Pouca, and A. Arêde. 2017. "Seismic Performance of the Infill Masonry Walls and Ambient Vibration Tests after the Ghorka 2015, Nepal Earthquake." *Bulletin of Earthquake Engineering* 15 (3): 1185–1212. https://doi.org/10.1007/s10518-016-9999-z.

[38] Rodrigues, Hugo, A. Furtado, N. Vila-Pouca, H. Varum, and A. R. Barbosa. 2018. "Seismic Assessment of a School Building in Nepal and Analysis of Retrofitting Solutions." *International Journal of Civil Engineering* 16 (11): 1573–89. https://doi.org/10.1007/s40999-018-0297-9.

[39] Dizhur, D., R. P. Dhakal, J. Bothara, and J. M. Ingham. 2016. "Building Typologies and Failure Modes Observed in the 2015 Gorkha (Nepal) Earthquake." *Bulletin of the New Zealand Society for Earthquake Engineering* 49 (2). https://doi.org/10.5459/bnzsee.49.2.211-232.

[40] Conde, F. R. 2018. "Avaliação Da Capacidade Sismo-Resistente de Edifícios Em Alvenaria Segundo a NTC-2017." Porto: Faculdade de Engenharia da Universidade do Porto.

[41] EN 1998-1. "Eurocode 8: Design of Structures for Earthquake Resistance – Part 1: General Rules, Seismic Actions and Rules for Buildings." European Comittee for Standardization: Brussels, Belgium, 2004.

[42] EN 1992-1-1. "Eurocode 2 - Design of Concrete Structures - Part 1–1: General Rules and Rules for Buildings." European Comittee for Standardization: Brussels, Belgium, 2004.

[43] prEN1998-1-2:2024. "Eurocode 8: Design of Structures for Earthquake Resistance - Part 1-2: Rules for New Buildings, Draft Post-ENQ 48th Meeting CEN/TC250/SC8." European Comittee for Standardization: Brussels, Belgium, 2024.

[44] NBR 15421. *"Projeto de Estruturas Resistentes a Sismos - Procedimento."* 2023. Associação Brasileira de Normas Técnicas, Rio de Janeiro, Brazil. 2023.

[45] NBR 6118. *"Projeto de Estruturas de Concreto - Procedimento."* Associação Brasileira de Normas Técnicas, Rio de Janeiro, Brazil. 2023.

[46] NTC-RSEE. *"Administración Pública de la Ciudad de México Norma Técnica Complementaria Para La Revisión de La Seguridad Estructural de Las Edificaciones."* Órgano de Difusión del Gobierno de la Ciudad de México: México City, Mexico, 2017.

[47] MOC-2015. *"Manual de Diseño de Obras Civiles: Diseño Por Sismo."* Comisión Federal de Electricidad: México City, Mexico, 2015.

[48] ASCE/SEI7-22. *"Minimum Design Loads and Associated Criteria for Buildings and Other Structures."* American Society of Civil Engineers: Reston, VA, USA, 2021.

[49] NCh 433.Of1996 Modificada En 2012. *"Diseño Sísmico de Edificios."* Instituto Nacional de Normalizacion: Santiago, Chile, 2012.

[50] NCh2369. *"Diseño Sísmico de Struturas e Instalaciones Industriales."* Instituto Nacional de Normalizacion: Santiago, Chile, 2003.

[51] NCh2745. *"Análisis y Diseño de Edificios con Aislación Sísmica."* Instituto Nacional de Normalizacion: Santiago, Chile, 2013.

[52] NCh3411. *"Diseño Sísmico de Edificios con Sistemas Pasivos de Disipación de Energia."* Instituto Nacional de Normalizacion: Santiago, Chile, 2017.

[53] NCh 3357. *"Diseño Sísmico de Componentes y Sistemas no Estructurales."* Instituto Nacional de Normalizacion: Santiago, Chile, 2015.

[54] NZS1170-5. *"Structural Design Actions—Part 5: Earthquake Actions—New Zealand Commentary."* Standards New Zealand: Wellington, New Zealand, 2004.

3 Comparative analysis of the impact of vertical irregularities on reinforced concrete moment resisting frame structures according to Eurocode 8

Santos D, Melo J, Varum H. "Comparative Analysis of the Impact of Vertical Irregularities on Reinforced Concrete Moment-Resisting Frame Structures According to Eurocode 8." *Buildings* 14 (9). https://doi.org/10.3390/buildings14092982.

3.1 Abstract

Eurocode 8 is undergoing a revision process encompassing novel ductility classes, damage limitation limits, local ductility conditions corresponding to newly defined prescriptions and structural irregularity criteria. In this paper, we specifically assessed the influence of an irregularity in elevation, imposed by different elevations, on the first and third storeys of buildings, and variations in the cross-sections of columns during the seismic response of reinforced concrete (RC) structures. To assess this impact, an extensive examination was conducted on thirteen five-storey moment-resisting frame (MRF) buildings. The design of those structures was carried out on the Robot Structural Analysis Professional framework following the current generation of Eurocodes 2 and 8, and the seismic response analysis was carried out using the SeismoStruct v2024 software. The results were compared to evaluate the influence of imposed irregularities in elevation due to the increasing height, column cross-section, mass, and resistance variation. The study's outcomes revealed that, for DCM structures, the imposed irregularities in elevation have different impacts on the seismic response. Increasing the heights of ground or middle floor have substantial deleterious effects on the building's seismic response. The planned geometry and variations in the cross-sections of columns substantially impact inter-storey drift and base shear. The effects of mass and resistance irregularities were neglected in this study. As such, more studies on those matters are necessary to allow our results to be further generalised.

3.2 Introduction

The performance of structures during significant seismic events has been evaluated, providing useful knowledge to guide engineers and builders on how to effectively design and build seismic load-resisting structures [1–6]. With the evolution of design techniques, engineered structures have performed relatively well in areas that have been inhabited for a long time and are subject to quite regular firm ground shaking. Structural engineers can discern the most appropriate construction materials and techniques by studying such design procedures, although not all of them will be appropriate due to regional variations [7].

The mechanisms behind considerable amounts of structural damage are seen often and have been determined to be due to improperly configured structures. These include torsion caused by asymmetrical masonry infills, single-storey soft-storey buildings, and inadequate longitudinal and transverse reinforcement detailing [8].

Non-uniform distributions of mass, stiffness, and strength, either separately or in various combinations throughout the height of structures, are common types of irregularities. These vertical irregularities have different impacts on the seismic responses of structures. This topic has been extensively addressed in numerous studies [4,9–18], and their major conclusions can be highlighted as follows:

1. Vertical irregularity plays a significant role in RC buildings' vulnerability [12];
2. Most of the energy input to the frames with irregular elevations due to setbacks was dissipated through beams with plastic hinges [9];
3. An increase in the number of slab openings leads to greater energy consumption and enhances the formation of cracks in columns, beams, and their joints, and additionally, 25% is considered a safe threshold regarding rigid diaphragm behaviour [11];
4. Mass irregularities usually have limited effects on seismic response [13];
5. Stiffness irregularities have considerable effects on storey shear, capacity, and storey drift demands [13];
6. The use of RC walls in structures with weak/soft-storey irregularities has a positive effect on the seismic response [15];
7. The magnitude of the peak inter-storey drift ratio is more pronounced at the soft-storey level in buildings with that irregularity [16];
8. Non-structural elements with short periods experience high seismic demands when attached to the lower floors of a building lacking vertical stiffness irregularities [16];
9. Middle soft-storey buildings exhibit the greatest amplification of component acceleration [16].

However, it is necessary to study this matter further because the seismic behaviour of structures is far from being understood sufficiently [19], especially due to the different levels of development of and the approaches within seismic codes regarding vertical irregularity that may produce different interpretations of the criteria for such irregularities, impacting the structural design of buildings.

The goal of contemporary design processes is to create a structural system that per forms well enough to withstand seismic occurrences while maintaining the required stiff ness, strength, and deformation capacities. For a building to achieve its performance targets under earthquake loading, the structural system needs to be properly sized and designed in line with the specifications.

EN1998-1 [20] and EN 1992-1-1 [21] define structural analysis and design methodologies for reinforced concrete structures, including stiffness, strength, and ductility requisites. The development of a second generation of Eurocode 8 prEN 1998-1-2 [22] is ongoing and new requirements for irregular structures are being proposed. The Eurocodes delineate the specifications for member design strengths and detailing requirements.

To assess the effect of an irregularity in elevation on a structure's seismic response, this article compares thirteen five-storey MRF buildings. These structures highlight the fact that, according to the most recently available census data on Portuguese buildings [23], the most common types of buildings are those with four or five storeys. The models were developed using the structure studied by Ref. [24]. They have three distinct plan configurations, variable storey height configurations (from 3 m to 5.5 m of height), and non-torsional flexibility properties.

The design calculations for DCM structures were conducted in accordance with the requirements of EN 1998-1 [20]. The method employed was a force-based approach, which included the response spectrum method. This methodology is characterized by a linear analysis integrating overstrength and nonlinear response through the behaviour factor (q). A pushover analysis approach has been applied to the designed structures to assess their seismic response and check the impact of the irregularity in elevation.

This research intends to contribute to a better understanding of the impact of an irregularity in elevation in the seismic responses of MRF structures, considering a wide range of structural inter-storey heights of the ground and middle floors, variations in the columns' cross-sections, mass and resistance discrepancies between storeys, and three different geometric configurations.

3.3 MATERIALS AND METHODS

Thirteen RC structures with distinct compositions were considered in our analysis. The buildings' attributes were based on invariant characteristics (frames, span length, construction materials, vertical loads, and seismicity) as well as specific variables unique to each building (inter-storey height and inertial masses).

Based on the symmetrical and isotropic assumptions outlined in EN 1998-1 [20] and prEN 1998-1-2 [22], thirteen models of structures were constructed. These structures were assigned to DCM ductility classes based on the maximum response spectral acceleration within the constant acceleration range (S_α), defined as being $2.5\,.\,a_g.S.\eta$, as per EN 1998-1 [20].

3.3.1 STRUCTURAL CHARACTERIZATION

The structures assessed consist of three configurations, all with five storeys and with differences in their planned geometries, as shown in Figures 3-1 and 3-2.

The structural system comprises frames spaced 6.0 m apart on the x-axis and 7.0 m apart along the y-axis, typical of residential buildings, as illustrated in Figures 3-1 and 3-2. The height between floors is 3.0 m for all storeys. The models had their heights in-creased by 0.5 m to a maximum of 5.5 m of elevation on the ground storey or on the 3rd storey. The solid slab thickness was 15.0 cm on all floors. Table 3-1 presents the geometrical characteristics of each analysed structure.

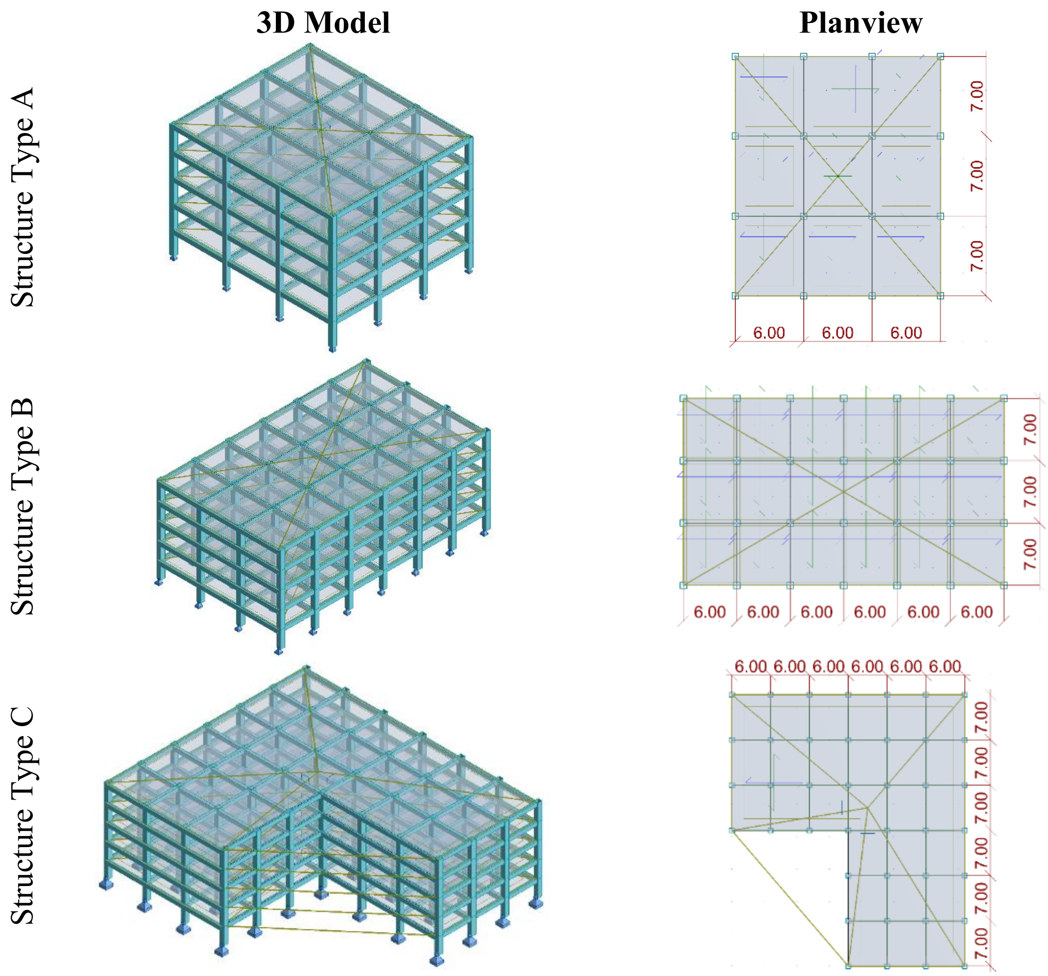


Figure 3-1: Frame building system configurations (Robot Structural Analysis Professional 2024)

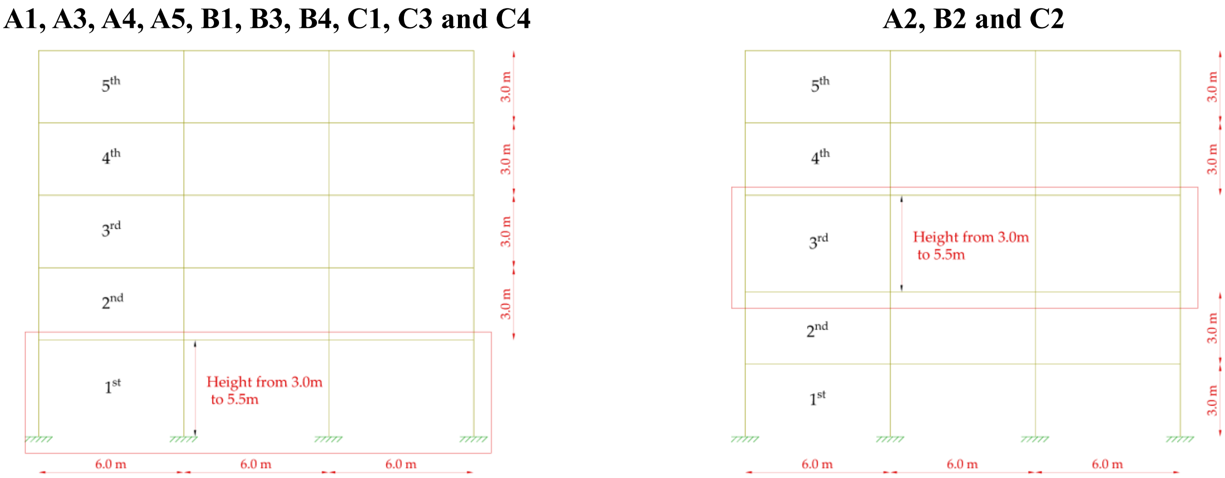


Figure 3-2: Frame building cutaway

Table 3-1: Structural elements dimensions of the MRF structures

Structure	Storey	Columns (cm × cm)	Beams (cm × cm)	Storey Elevation Increase	Height of Storey with Elevation Increase (m)
A1	All	55 × 55	40 × 55	1	3.0
A2	All	55 × 55	40 × 55	3	9.0
A3	Storeys 1 and 2	55 × 55	40 × 55	1	3.0
	Storeys 3 and 4	50 × 50	40 × 50		
	Storey 5	45 × 45			
A4	Storeys 1 and 2	55 × 55	40 × 55	1	3.0
	Storeys 3 and 4	50 × 50			
	Storey 5	45 × 45			
A5	Storeys 1 and 2	55 × 55	40 × 55	1	3.0
	Storeys 3, 4 and 5	45 × 45			
B1	All	75 × 75	45 × 65	1	3.0
B2	All	75 × 75	45 × 65	3	9.0
B3	Storey 1 and 2	75 × 75	45 × 65	1	3.0
	Storeys 3 and 4	70 × 70			
	Storeys 5	65 × 65			
B4	Storeys 1, 2 and 3	75 × 75	45 × 65	1	3.0
	Storeys 4 and 5	65 × 65			
C1	All	75 × 75	45 × 65	1	3.0
C2	All	75 × 75	45 × 65	3	9.0
C3	Storeys 1 and 2	75 × 75	45 × 65	1	3.0
	Storeys 3 and 4	70 × 70			
	Storey 5	65 × 65			
C4	Storeys 1, 2 and 3	75 × 75	45 × 65	1	3.0
	Storeys 4 and 5	65 × 65			

3.3.2 MATERIALS

The beams and columns in the assessed structures were considered to be constructed using C30/37 concrete and B500 steel reinforcements.

The C30/37 concrete's properties were defined according to EN 1992-1-1 [21] and EN 206-1 [25], and its mechanical properties are detailed in Table 3-2. By choosing Exposure Class XC1, as defined by the environmental conditions in EN 206-1 [25] as stated in EN 1992-1-1 [21], the nominal cover was set at 30 mm. The steel reinforcement considered was B500. Its mechanical properties were defined in accordance with EN 10080 [26], as shown in Table 3-2.

Table 3-2: Concrete and steel mechanical properties

C30/37 Concrete		B500 Steel Reinforcements	
f_{ck} (MPa)	30	f_{yk} (MPa)	500
f_{cm} (MPa)	38	f_{ym} (MPa)	555
f_{cd} (MPa)	20	f_{yd} (MPa)	435
$f_{ctk,0.95}$ (MPa)	2.0	E_s (GPa)	200
f_{ctm} (MPa)	2.9	ε_{sy} (‰)	2.5
f_{ctd} (MPa)	1.3		
E_{cm} (GPa)	33		
$\varepsilon_{cu,1}$ (‰)	3.5		

3.4 DESIGN PRINCIPLES

3.4.1 STRUCTURAL CONCEPTION CRITERIA

The EN 1998-1 [20] and prEN 1998-1-2 [22] specifications were adhered to within the structural design. To evaluate the deformations and internal forces, Finite Element models were developed in Robot Structural Analysis Professional [27]. The models were designed to comply with the EN 1998-1 [20] and prEN 1998-1-2 [22] requisites, and their characteristics can be described as follows:

1. The buildings were modelled to resist earthquake forces in two directions, X and Y;
2. The modelling accounted for global torsional effects;
3. Slabs were treated as rigid diaphragms;
4. Columns and beams were modelled as linear elastic bar elements, but there was no specific modelling of joint stiffness in this analysis;
5. There was no redistribution of bending moments in this modelling approach;
6. The connection between beams and columns was assumed to be concentric;
7. Accidental eccentricities and second-order effects were considered in the analysis;
8. The contribution of infills to the structural response was not considered; however, their weight was included in the overall loads;
9. The foundation support was restrained in all directions.

Notably, the chosen reinforcement design for each kind of structure (A1-5, B1-4 and C1-4) was the worst scenario from all heights. In the case of the A1 structure, for instance, the worst case was the A1 structure with 5.0 m and 5.5 m of height in the first storey.

3.4.2 VERTICAL LOADS AND SEISMIC ACTIONS

The vertical loads of office buildings were considered in all assessed buildings according to EN 1990 [28] and EN 1991-1-1 [29]. The experiments can be summarized as follows:

1. Structure self-weight ($G_{k,1}$): All storeys: RC columns, beams, and slabs have a density of 25 kN/m^3;
2. Other permanent loads ($G_{k,2}$):
   i. First to fourth storeys: External walls and partitions have a uniformly distributed load of 1.2 kN/m^2, and these storeys have a final a load of 1.3 kN/m^2;
   ii. Fifth storey: Has a final load of 1.0 kN/m^2.
3. Live loads (Q_k):
   i. First to fourth storey: Buildings for use as civil dwellings, falling into the usage category A, have a live load of 2.0 kN/m^2;
   ii. Fifth storey: H roofs considered inaccessible, except for normal maintenance.

To develop the design spectra for the elastic analysis considering the structural systems, the behaviour factor q and its components were defined in accordance with EN 1998-1 [20] and prEN 1998-1-2 [22] for regular structures both in plans and elevations. The analysis took into account the DCM ductility class recommended in EN 1998-1 [20].

The definition of the seismic action follows EN 1998-1 [20]. The recommendation in the quoted standard for seismic intervention for local collapse prevention with an exceedance probability of 10% in 50 years is for structures of ordinary importance. In this work, ground type B and importance class II with a correspondence factor of $\gamma_I = 1.0$ were used to build up the type 1 elastic and design spectra for the elastic analysis in accordance with EN 1998-1 [20]. The values of the parameters characterizing the elastic response spectra (ξ = 5%) for the structure analyzed are displayed in Table 3-3.

Table 3-3: Horizontal response spectra properties

Ductility Class	S_α (m/s^2)	a_g (m/s^2)	q
DCM	5.00	1.67	3.90

3.4.3 Load combination

The structures were designed for the following:

1. ULS – Fundamental combinations

$$\sum_{j\geq 1} \gamma_{G,j} \cdot G_{k,j} \text{“+”} \gamma_{Q,1} \cdot Q_{k,1} \qquad (1)$$

2. ULS – Seismic combinations

$$\sum_{j\geq 1} G_{k,j} \text{ “+”} A_{Ed} \text{“+”} \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} \qquad (2)$$

Wind, snow, and thermal actions were not considered as the seismic demand is paramount in influencing the design. The combinations of actions are outlined in Table 3-4. The seismic combinations account for accidental eccentricities.

Table 3-4: Load combinations

<table>
<tr><th>Load Combination</th><th>Nature</th><th>$G_{k,1}$</th><th>$G_{k,2}$</th><th>Q_k</th><th>$E_{Ed,x}$</th><th>$E_{Ed,y}$</th><th>$A_{cce,i,x}$</th><th>$A_{cce,i,y}$</th></tr>
<tr><td>1</td><td>ULS-Fundamental</td><td>1.35</td><td>1.35</td><td>1.50</td><td>-</td><td>-</td><td>-</td><td>-</td></tr>
<tr><td>2</td><td rowspan="8">ULS-Seismic</td><td rowspan="8">1.00</td><td rowspan="8">1.00</td><td rowspan="8">0.30</td><td>0.30</td><td>1.00</td><td>0.30</td><td>1.00</td></tr>
<tr><td>3</td><td>−0.30</td><td>1.00</td><td>−0.30</td><td>1.00</td></tr>
<tr><td>4</td><td>0.30</td><td>−1.00</td><td>0.30</td><td>−1.00</td></tr>
<tr><td>5</td><td>−0.30</td><td>−1.00</td><td>−0.30</td><td>−1.00</td></tr>
<tr><td>6</td><td>1.00</td><td>0.30</td><td>1.00</td><td>0.30</td></tr>
<tr><td>7</td><td>1.00</td><td>−0.30</td><td>1.00</td><td>−0.30</td></tr>
<tr><td>8</td><td>−1.00</td><td>0.30</td><td>−1.00</td><td>0.30</td></tr>
<tr><td>9</td><td>−1.00</td><td>−0.30</td><td>−1.00</td><td>−0.30</td></tr>
</table>

3.5 SEISMIC RESPONSE ASSESSMENT OF DESIGNED STRUCTURES

The seismic assessment of the structures was carried out in the SeismoStruct [30] framework. This software has been extensively quality checked and validated in the scientific environment. It is a Finite Element package capable of predicting the large displacement behaviour of space frames under static or dynamic loadings, considering both geometric nonlinearities and material inelasticity [31].

SeismoStruct accommodates both static (forces and displacements) and dynamic (accelerations) loads and offers a range of analytical capabilities including eigenvalue analysis, nonlinear static pushover, nonlinear static time-history analysis, and response spectrum analysis, among others [31].

We used a static pushover analysis as the modelling approach. Pushover analysis is a method employed in assessing buildings against seismic loads, aiming to ascertain their structural performances under various irregularity scenarios [32]. Several studies have also used this type of analysis in studying irregular buildings [9,32–34]. Inelastic force-based frame elements were subdivided into seven integration sections, with each section further discretized into 204 fibbers to capture the nonlinear material behaviour accurately.

Eurocode 8, in its first [20] and second [22] generations, presents differences in the vertical irregularity criteria. The first generation is more qualitative in the criteria assessment. However, the new draft presents a more quantitative approach. Those criteria are presented in Table 3-5.

Table 3-5: Comparison of vertical irregularity criteria between the generations of Eurocode 8

Criteria	EN 1998-1 [20]	prEN 1998-1-2:2024 [22]
Conti-nuity	Continuity of all resistant systems in lateral directions	Continuity of all resistant systems in lateral directions
Lateral stiffness	Gradual or no reduction in lateral stiffness and mass between storeys.	Not exceed 30% of variation between storeys
Mass	Gradual or no reduction in mass between storeys	Not exceed 150% of variation between storeys
Resis-tance	Proportional variation in the ratio between real and required strength between storeys	The ratio between real and required strength of the floor does not exceed 30% between storeys
Setback	Not surpassing 20% of the plan dimension of the lower storey in the setback direction. Single setback situated within the lower 15% of the overall elevation of the primary structural system must not exceed 50% of the planned dimensions of the inferior floor. Non-symmetrical setbacks: Setbacks of all levels must not be superior to 30% of the planned dimensions at floor level above the foundation or the upper level of a rigid basement, with each individual setback being limited to 10% of the planned dimensions of the lower storey.	Not mentioned

3.6 RESULTS AND DISCUSSION

Table 3-6 presents the first three natural frequencies (F_1, F_2, and F_3) of each type of structure obtained from SeismoStruct. The structures A1 to A5, B1 to B4, and C1 to C4 behave similarly regarding natural frequency, without much deviation despite the increase in height.

Figure 3-3 presents a comparison between the structural frequencies obtained from SeismoStruct and the structural frequency method proposed by Ditommaso et al. [35]. The frequency results for the B1–B4 structures, the C1–C2 structures with heights of 4.5 to 5.5 m, and the C3–C4 structures with heights of 4.0 to 5.5 m comply with the structural frequency prediction method even though that methodology includes the infill walls' contributions in the assessment. However, huge differences between the frequencies obtained by the models and the aforementioned prediction method are shown for the other structures, reaching up to 108% for the A3_4.0m model.

Table 3-6: Natural frequencies

Structure	Height (1st or 3rd Storey) (m)	Frequency (Hz)			Structure	Height (1st or 3rd Storey) (m)	Frequency (Hz)		
		F_1	F_2	F_3			F_1	F_2	F_3
A1	3.0	2.0	2.1	2.3	B3	3.0	2.9	3.2	3.3
	3.5	1.9	2.0	2.1		3.5	2.8	3.1	3.1
	4.0	1.8	1.9	2.0		4.0	2.6	2.9	2.9
	4.5	1.7	1.7	1.9		4.5	2.5	2.7	2.7
	5.0	1.5	1.6	1.7		5.0	2.3	2.5	2.6
	5.5	1.4	1.5	1.6		5.5	2.2	2.3	2.4
A2	3.0	2.0	2.1	2.3	B4	3.0	3.0	3.2	3.3
	3.5	1.9	2.0	2.1		3.5	2.8	3.1	3.1
	4.0	1.8	1.9	2.0		4.0	2.7	2.9	2.9
	4.5	1.7	1.8	1.9		4.5	2.5	2.7	2.7
	5.0	1.6	1.7	1.8		5.0	2.3	2.5	2.6
	5.5	1.5	1.5	1.7		5.5	2.2	2.3	2.4
A3	3.0	1.9	2.0	2.1	C1	3.0	3.0	3.2	3.3
	3.5	1.8	1.9	2.0		3.5	2.9	3.0	3.1
	4.0	1.7	1.8	1.9		4.0	2.7	2.8	2.9
	4.5	1.6	1.7	1.8		4.5	2.5	2.7	2.8
	5.0	1.5	1.5	1.7		5.0	2.4	2.5	2.6
	5.5	1.4	1.4	1.5		5.5	2.2	2.3	2.4
A4	3.0	1.9	2.0	2.2	C2	3.0	3.0	3.2	3.3
	3.5	1.8	1.9	2.1		3.5	2.9	3.0	3.1
	4.0	1.7	1.8	1.9		4.0	2.7	2.8	2.9
	4.5	1.6	1.7	1.8		4.5	2.5	2.7	2.8
	5.0	1.5	1.5	1.7		5.0	2.4	2.5	2.6
	5.5	1.4	1.4	1.6		5.5	2.3	2.4	2.4
A5	3.0	2.0	2.0	2.2	C3	3.0	3.0	3.2	3.3
	3.5	1.9	1.9	2.1		3.5	2.9	3.0	3.1
	4.0	1.7	1.8	2.0		4.0	2.7	2.8	2.9
	4.5	1.6	1.7	1.8		4.5	2.5	2.6	2.7
	5.0	1.5	1.6	1.7		5.0	2.4	2.5	2.6
	5.5	1.4	1.4	1.6		5.5	2.2	2.3	2.4

Table 3-6: Cont.

Structure	Height (1st or 3rd Storey) (m)	Frequency (Hz)			Structure	Height (1st or 3rd Storey) (m)	Frequency (Hz)		
		F_1	F_2	F_3			F_1	F_2	F_3
B1	3.0	3.0	3.3	3.3	C4	3.0	3.1	3.3	3.4
	3.5	2.8	3.1	3.1		3.5	2.9	3.1	3.2
	4.0	2.7	2.9	2.9		4.0	2.8	2.9	3.0
	4.5	2.5	2.7	2.7		4.5	2.6	2.7	2.8
	5.0	2.3	2.5	2.6		5.0	2.4	2.5	2.6
	5.5	2.2	2.3	2.4		5.5	2.3	2.3	2.4
B2	3.0	3.0	3.2	3.3					
	3.5	2.8	3.1	3.1					
	4.0	2.7	2.9	2.9					
	4.5	2.5	2.7	2.8					
	5.0	2.4	2.6	2.6					
	5.5	2.2	2.4	2.4					

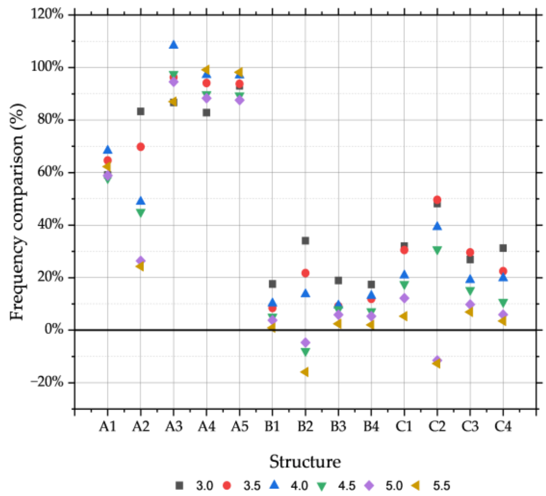


Figure 3-3: Comparison of structural frequencies: the calculated frequencies versus frequencies obtained from the prediction method proposed by Ditommaso et al. [35]

It should be noted that, for all of the parameters discussed in Figures 3-4 – 3-9, the A1 structure in which all storey heights are 3.0 m is the reference model that the others are compared to. Additionally, the moved mass in first vibration mode was always higher than 80%, starting by translation in X direction, than in Y direction in the second vibration mode, and finally by rotation in Z axis.

Figures 3-4 and 3-5 present the evolutions of the lateral stiffness on individual floors in all of the structures, along both the X and Y axes. The lateral stiffnesses of the structures were obtained by applying unitary displacements to the peripheral column nodes of the storey under assessment and fixing the column nodes of the storeys below. The slope obtained for the force–displacement relationship represents the lateral stiffness of the storey. This procedure was performed on all storeys, with lateral stiffness being the sum of all the calculated forces that generate a unitary displacement.

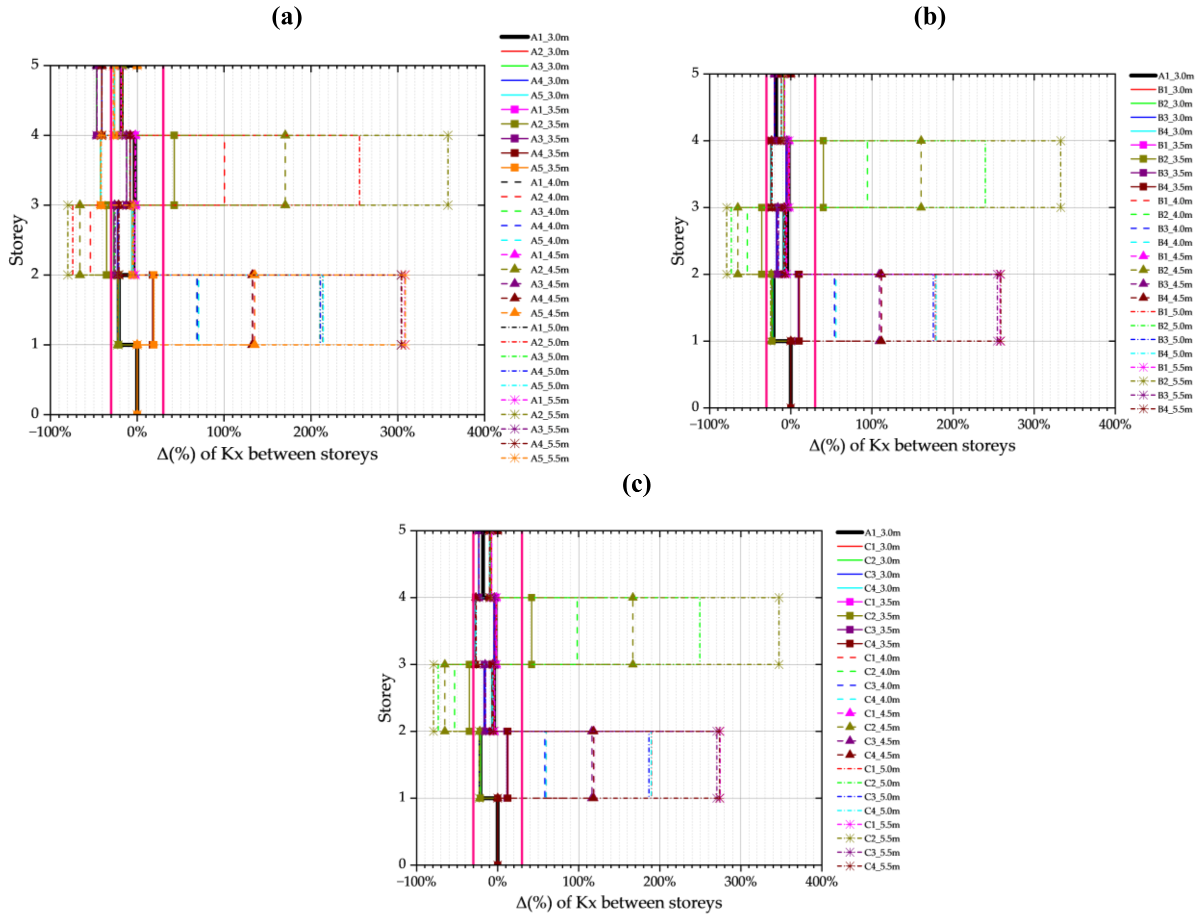


Figure 3-4: Lateral stiffnesses in the X-axis: (a) Type A structures; (b) Type B structures; (c) Type C structures

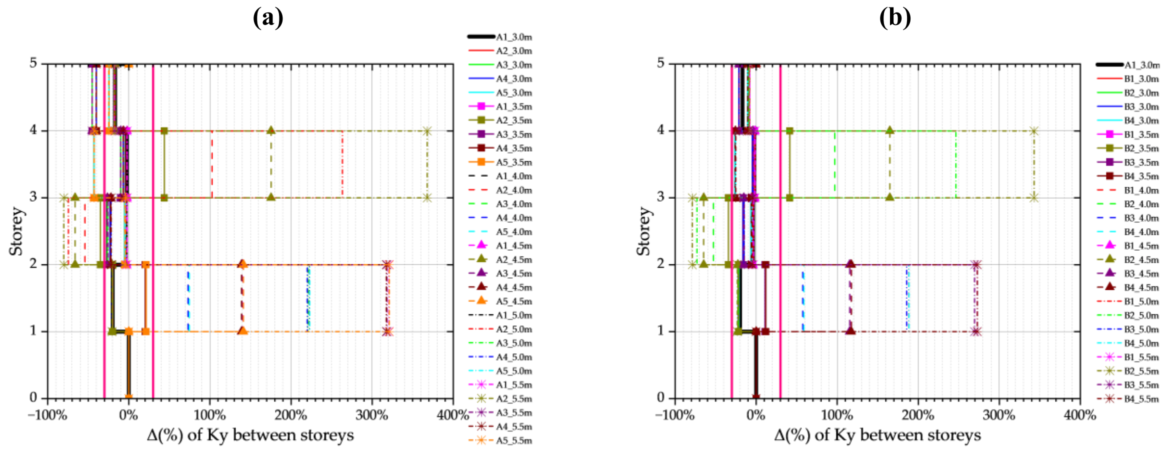


Figure 3-5: Lateral stiffnesses in the Y-axis: (a) Type A structures; (b) Type B structures; (c) Type C structures

(c)

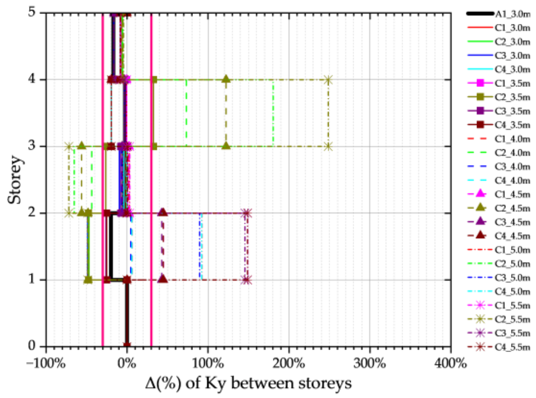


Figure 3-5: Cont.

Figures 3-4 and 3-5 and Table 3-6 support the following findings:

1. All structures, except for A1, B1, B3, and B4 with 3.0 m of height in the first storey and A2 and B2 on the third floor, as well as A1, B1, B3, B4, C1, C3, and C4 with 3.5 m of height on the first floor, present irregularities in elevation due to a reduction in the lateral stiffness of more than 30%, according to prEN 1998-1-2 [22];
2. For the models analysed, the increase in height on the first or an intermediate floor produces a stronger effect on lateral stiffness, and therefore, on the irregularity in elevation, than the reduction in the cross-sections of the columns. Also, in this regard, the impact of increasing height on the third floor is worse than on the first floor. These assessments all occurred on all considered plan geometries, which means that, for the assessed structures, an irregularity in the plan does not have a considerable effect on the lateral stiffness.

It is important to highlight that only the A5 structure presents a resistance variation higher than 30% in the fourth storey. This parameter was determined through pushover analyses of the structure, considering the forces applied on the peripheral columns in each storey and the axis separately with the nodes of the columns in the storeys below being fixed. The maximum base shear observed in each storey was taken and compared to each other. According to prEN 1998-1-2 [22], this represents an irregularity in elevation. However, that fact did not demonstrate a huge difference in the lateral stiffness behaviour.

Figures 3-6 and 3-7 represent the displacement responses and inter-storey drifts of the studied structures in the maximum baseshear of each capacitive curves, and Table 3-7 shows the maximum top displacement and storey drift at each observed height studied for the structures with these characteristics. Figure 3-8 illustrates the % variation in the maximum inter-storey drift observed in each structure in comparison with the structures with 3.0 m of elevation and the base shear of each structure for each studied height. Figure 3-9 shows the capacity curves of the studied structures, complementing the information provided in Figure 3-6 and Table 3-7.

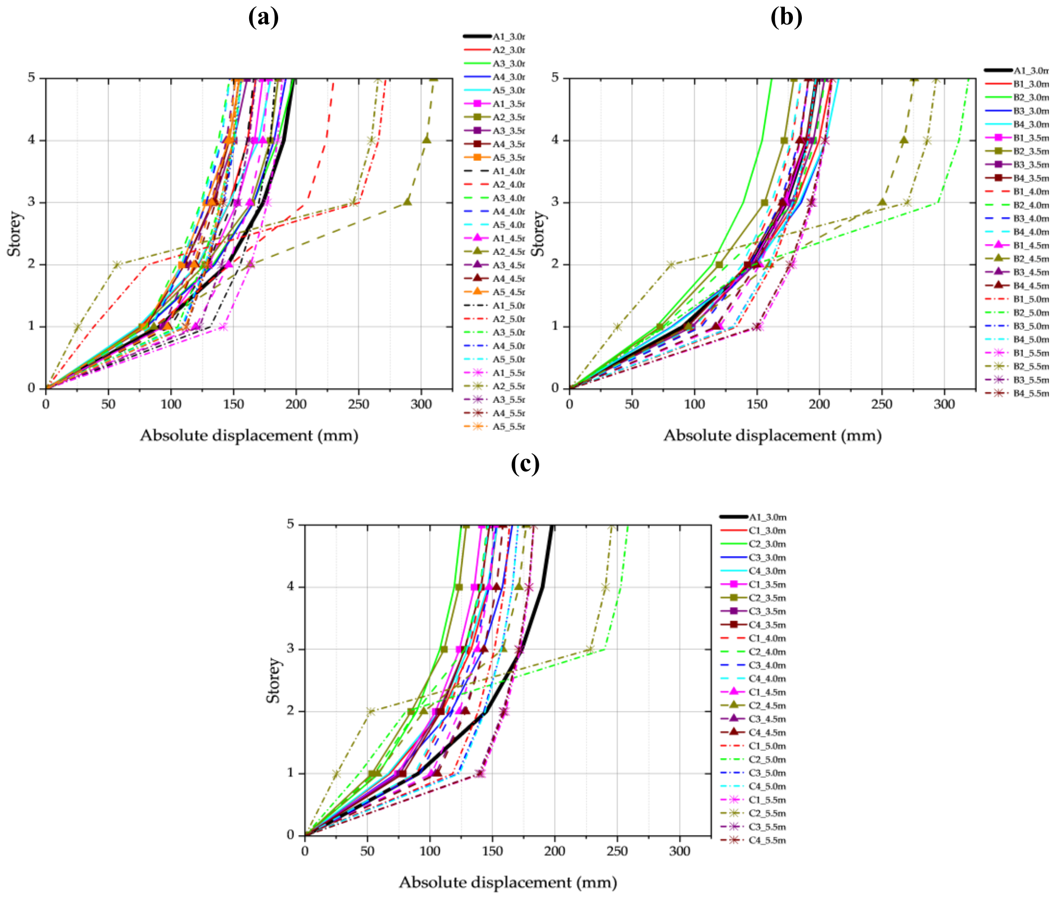


Figure 3-6: Absolute displacement response: (a) Type A structures; (b) Type B structures; (c) Type C structures

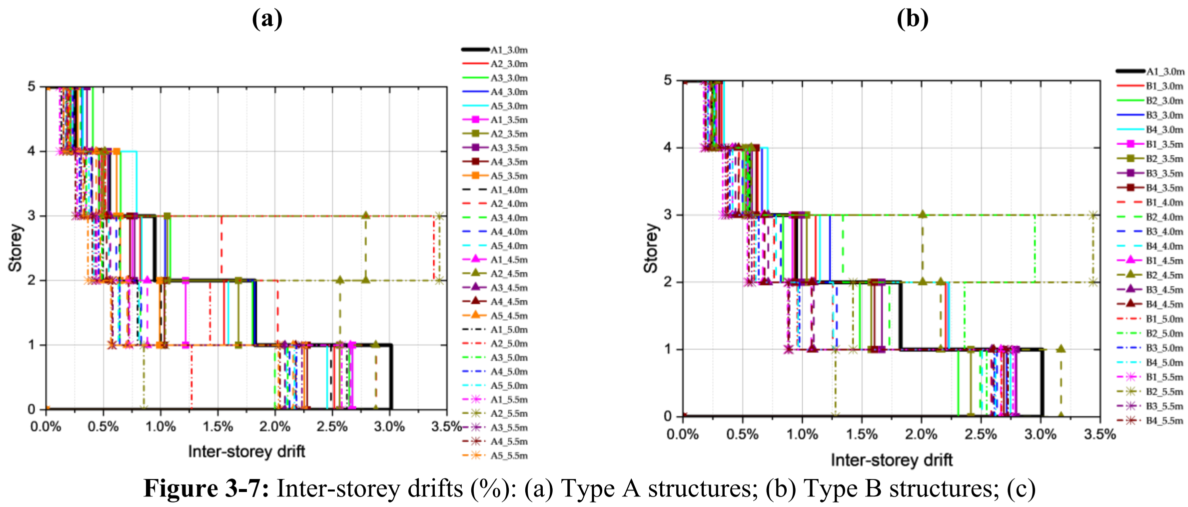


Figure 3-7: Inter-storey drifts (%): (a) Type A structures; (b) Type B structures; (c) Type C structures

(c)

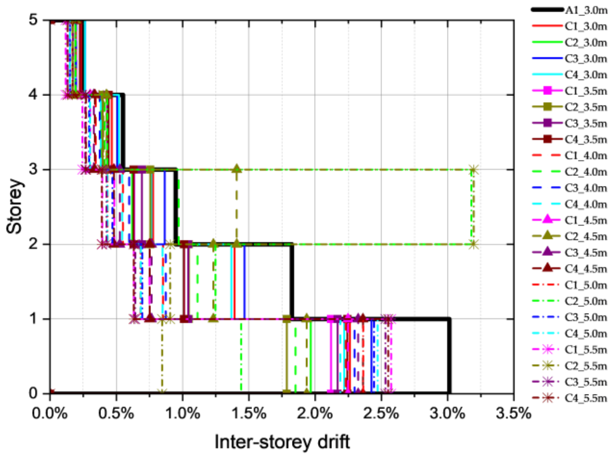


Figure 3-7: Cont.

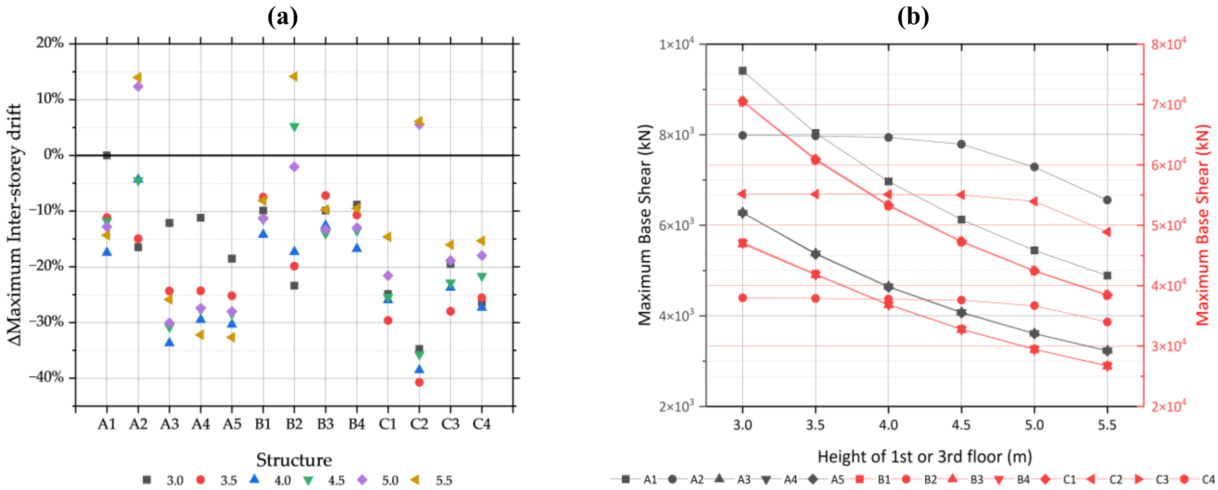


Figure 3-8: (a) Variations in the maximum storey-drift (%) in comparison to structures with 3.0 m of height in all storeys; (b) maximum base shear of the studied structures

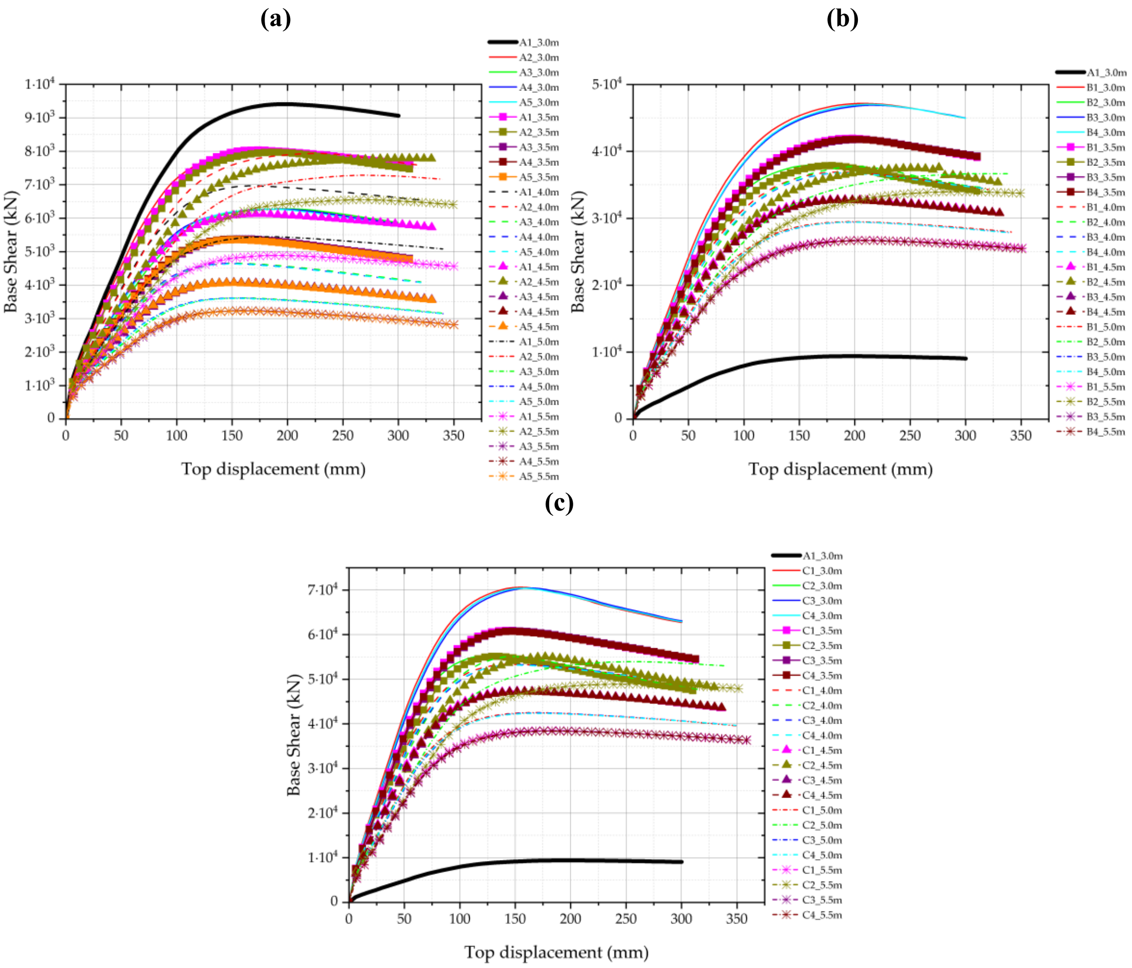


Figure 3-9: (a) Capacity curves of Type A structure; (b) capacity curves of Type A structure; (c) capacity curves of Type C structure

Table 3-7: Maximum top displacements and inter-storey drifts

Height (m)	Structure	Storey	Maximum Top Displacement (mm)	Height (m)	Structure	Storey	Inter-Storey Drift (%)
3.0	B4_3.0	5	215.4	3.0	A1_3.0	1	3.0
3.5	B3_3.5	5	204.1	3.5	B3_3.5	1	2.8
4.0	A2_4.0	5	230.1	4.0	A2_4.0	1	2.9
4.5	A2_4.5	5	309.9	4.5	B2_4.5	1	3.2
5.0	B2_5.0	5	319.2	5.0	A2_5.0	3	3.4
5.5	B2_5.0	5	293.4	5.5	A2_5.5/B2_5.5	3	3.4

The following findings can be extracted from Figures 3-6 – 3-9 and Table 3-7:

1. From Figure 3-6, it can be seen that decreases in the top displacements occur when the height of the ground soft storeys in the models increases from 3.0 m to 4.5 m, as demonstrated in the studies by Abd-Alghany et al. [36] and Ibrahim and Mohamad [37]. However, when the first-floor elevation rises to 5.0 m and 5.5 m, the top displacements increase;
2. From Table 3-7 and Figures 3-6 and 3-7, it can be seen that the maximum displacements and storey drifts are observed, respectively, in the B2 structure

(height = 5.0 m) and in the A2 and B2 structures (height = 5.5 m). Those structures present the height increase in the intermediate floor. Comparing these results with the information provided in Figure 3-6, it can be concluded that, for structural displacement, the increase in the elevation of the intermediate floor produces a worse displacement behaviour than the same increase on the ground floor. It is also important to highlight that, except for A2, B2, and C2 structures, all other buildings present similar results regarding structural displacement;

3. From Figure 3-8a, it can be seen that, compared to the regular structure (A1 with h = 3.0 m), a maximum storey-drift increase of 14.2% occurred in structure B2 with 5.5 m of elevation on the intermediate floor. The A2 structure, in the same conditions, produced a maximum storey-drift increase of 14.0%, similar to the previous structure. However, for the C2 structure, the maximum increase observed was 6.1% with a height of 5.5 m. It can be concluded, from these results, that the planned geometry can impact the storey-drift behaviour of structures. Additionally, considering the variations in the cross-sections of the columns in the assessed structures, it can be concluded that this parameter produces a stronger impact in this regard. Except for the A2, B2, and C2 structures, which present height increases on the third floor, analogous results can be seen in Refs. [13,17,36,38];
4. From Figure 3-8a, compared to the regular structure (A1 with h = 3.0 m a maximum storey-drift decrease of 40.7% was observed for the C2 structure with a height of 3.5 m. An important point of discussion on that matter is, for the C2 structure, the storey-drift starts to decrease from a height of 4.0 m to 5.5 m. It can be inferred that the planned geometry and cross-section of columns play an important role in this behaviour;
5. From Figures 3-8b and 3-9, it can be seen that the A1 structure exhibits a considerably higher base shear value in comparison with the A3, A4, and A5 structures. However, the B1, B3, and B4 structures have similar base shear behaviours. The same can be observed for the C1, C3, and C4 structures. This probably occurs due to the reductions in the cross-sections of the columns observed in the A3, A4, and A5 models. The same reduction occurs in the B and C structures. However, the planned geometry compensates for that effect. Except for the A2, B2, and C2 structures which present height increases on the third floor, similar results can be found in Refs. [13,17,36,38];
6. Still concerning Figure 3-8b, the structures A2, B2, and C2 (with increasing heights of the third storey) present, respectively, 15.2%, 19.5%, and 21.9% lower base shears at a height of 3.0 m in comparison with, respectively, the A1, B1, and C1 structures in the same conditions. However, for heights of 4.0 m to 5.5 m, the A2, B2, and C2 structures exhibit higher base shears than other structures of the same type. Despite those structures presenting a worse structural behaviour in that regard, the deleterious effect of the increasing height on the first floor on structural demands is higher than those demands on A2, B2, and C2 structures. It can be concluded that, for the assessed structures, the soft-storey effect on the intermediate floor is more detrimental to the seismic response of the structure than the ground soft-storey, but only for heights of 3.0 m and 3.5 m.

To evaluate the relationship between mass variations in consecutive storeys and irregularities in the elevation criterium in seismic response, as prescribed in prEN 1998-1-2 [22], increases in the masses of the external beams on the third storey of each model of 20%, 30%, 40%, and 50% were simulated for the A2 and B2 structures. The results of this analysis are presented in Table 3-8. This parameter has no discernible impact on the behaviour of the structure in terms of displacement or maximum base shear during seismic occurrences, which corroborates the mass variation criteria in the aforementioned code, which presents a limit of 150% of variation for a structure be considered irregular, and also agrees with the results of Benaied et al. [13].

Table 3-8: Absolute displacement and base shear data, considering mass variations

Structure	Mass Variation between Storeys	Absolute Displacement (mm)					Maximum Base Shear (kn)
		Storey					
		1	2	3	4	5	
A2	0.0%	75.7	122.3	146.9	161.5	168.0	7983.4
A2_20%	19.7%	75.1	122.2	147.0	161.6	168.0	7995.3
A2_30%	21.3%	74.9	122.1	147.1	161.6	168.0	8001.2
A2_40%	23.0%	74.6	122.1	147.2	161.7	168.0	8006.8
A2_50%	24.6%	79.4	128.0	153.2	167.7	174.0	8013.1
B2	0.0%	79.8	136.0	165.6	182.7	191.5	43072.4
B2_20%	15.0%	83.3	141.5	171.5	188.7	197.6	43164.2
B2_30%	16.2%	83.0	141.4	171.6	188.8	197.6	43172.1
B2_40%	17.5%	82.7	141.4	171.7	188.8	197.6	43178.9
B2_50%	18.7%	82.5	141.3	171.7	188.8	197.6	43184.1
C2	0.0%	61.0	96.6	117.4	130.2	136.6	63095.1
C2_20%	13.9%	60.7	96.5	117.5	130.2	136.5	63125.9
C2_30%	15.0%	60.5	96.4	117.5	130.2	136.5	63150.3
C2_40%	16.2%	60.4	96.4	117.5	130.2	136.5	63174.4
C2_50%	17.3%	60.2	96.4	117.6	130.2	136.5	63197.8

3.7 SUMMARY AND CONCLUSIONS

This study evaluated a broad range of irregularities in elevation scenarios, providing a thorough examination of their impacts on seismic responses. Robot Structural Analysis Professional 2024 software was used to design several structures according to Eurocode guidelines and SeismoStruct v2024 software was used to conduct a push-over analysis of these structures to analyse the seismic response of those models.

It is important to highlight that our conclusions were focused on RC structures with specific planned configurations and heights, with C30/37 concrete and B500 reinforcements. This work contributes to bettering our understanding of structural behaviour, regarding several imposed irregularities in elevation.

Hence, the following conclusions can be made.

1. Despite the increases in height and the reductions in the cross-section of columns, the investigated buildings exhibit similar behaviours in terms of natural frequency;
2. Increasing the height of the ground or intermediate floor has deleterious effects on lateral stiffness, top displacement, inter-storey drift, and base shear.

Additionally, the soft-storey effect on the intermediate floor is more damaging to the seismic response of the structure than the ground soft-storey, but only for heights of 3.0 m and 3.5 m. On higher levels in the buildings, the behaviour is the opposite;

3. For the assessed structures, the irregularity in elevation due to the difference in resistance or mass between floors did not significantly affect the seismic response. It is important to highlight that the maximum mass variation imposed was 50%. However, further studies need to be carried out to draw better conclusions;
4. The plan geometry and the cross-section of columns play an important role in the inter-storey drift. Additionally, the column's cross-section substantially impacts the base shear of structures. However, that effect could be compensated for in the plan geometry;
5. Further studies with different structural geometries, considering irregularities in plan and elevation, more cross-section options for columns and beams, and mass and structural resistance variations, are necessary to draw further generalized conclusions.

3.8 References

[1] Braga, F., V. Manfredi, A. Masi, A. Salvatori, and M. Vona. 2011. "Performance of Non-Structural Elements in RC Buildings during the L'Aquila, 2009 Earthquake." *Bulletin of Earthquake Engineering* 9 (1): 307–24. https://doi.org/10.1007/s10518-010-9205-7.

[2] Verderame, G. M., F. De Luca, P. Ricci, and G. Manfredi. 2011. "Preliminary Analysis of a Soft-Storey Mechanism after the 2009 L'Aquila Earthquake." *Earthquake Engineering and Structural Dynamics* 40 (8): 925–44. https://doi.org/10.1002/eqe.1069.

[3] Palermo, M., R. R. Hernandez, S. Mazzoni, and T. Trombetti. 2014. "On the Seismic Behavior of a Reinforced Concrete Building with Masonry Infills Collapsed during the 2009 L'aquila Earthquake." *Earthquake and Structures* 6 (1): 45–69. https://doi.org/10.12989/eas.2014.6.1.045.

[4] Das, P. K., S. C. Dutta, and T. K. Datta. 2021. "Seismic Behavior of Plan and Vertically Irregular Structures: State of Art and Future Challenges." *Natural Hazards Review*. https://doi.org/10.1061/(ASCE)NH.1527.

[5] Dutta, S. C., L. Halder, and R. P. Sharma. 2021. "Seismic Vulnerability Assessment of Low to Mid-Rise RC Buildings Addressing Prevailing Design and Construction Practices in the Northeastern Region of the Indian Subcontinent: A Case Study Based Approach." *Structures* 33: 1561–77. https://doi.org/10.1016/j.istruc.2021.05.032.

[6] Vuran, E., C. Serhatoğlu, M. Ö. Timurağaoğlu, E. Smyrou, İ. E. Bal, and R. Livaoğlu. 2024. "Damage Observations of RC Buildings from 2023 Kahramanmaraş Earthquake Sequence and Discussion on the Seismic Code Regulations." *Bulletin of Earthquake Engineering*, 1–30. https://doi.org/10.1007/s10518-023-01843-3.

[7] Varum, H., J. Melo, A. Furtado, and A. Lima. 2022. "Irregularities in RC Buildings: Perspectives in Current Seismic Design Codes, Difficulties in Their Application and Further

Research Needs.” In *Seismic Behaviour and Design of Irregular and Complex Civil Structures IV*, edited by Rita Bento, Mario De Stefano, Dietlinde Köber, and Zbigniew Zembaty. Vol. 50. Geotechnical, Geological and Earthquake Engineering. Cham: Springer International Publishing. https://doi.org/10.1007/978-3-030-83221-6.

[8] Jara, J. M., E. J. Hernández, B. A. Olmos, and G. Martínez. 2020. “Building Damages during the September 19, 2017 Earthquake in Mexico City and Seismic Retrofitting of Existing First Soft-Story Buildings.” *Engineering Structures* 209. https://doi.org/10.1016/j.engstruct.2019.109977.

[9] Athanassiadou, C. J. 2008. “Seismic Performance of R/C Plane Frames Irregular in Elevation.” *Engineering Structures* 30 (5): 1250–61. https://doi.org/10.1016/j.engstruct.2007.07.015.

[10] Tena-Colunga, A. 2010. “Review of the Soft First Story Irregularity Condition of Buildings for Seismic Design.” *The Open Civil Engineering Journal* 4 (1): 1–15. https://doi.org/10.2174/1874149501004010001.

[11] Özbayrak, A., and F. Altun. 2021. “Numerical Investigation of the Effect of Beam Slab Openings in RC Structures on Seismic Behavior.” *Revista de La Construccion* 20 (3): 512–26. https://doi.org/10.7764/RDLC.20.3.512.

[12] Mouhine, M., and E. Hilali. 2022. “Seismic Vulnerability Assessment of RC Buildings with Setback Irregularity.” *Ain Shams Engineering Journal* 13 (1). https://doi.org/10.1016/j.asej.2021.05.001.

[13] Benaied, B., M. Hemsas, A. Benanane, and M. Hentri. 2023. “Seismic Analysis of RC Building Frames with Vertical Mass and Stiffness Irregularities Using Adaptive Pushover Analysis.” *Revista de La Construcción* 22 (3): 597–612. https://doi.org/10.7764/rdlc.22.3.597.

[14] Ulutaş, H. 2024. “Investigation of the Causes of Soft-Storey and Weak-Storey Formations in Low- and Mid-Rise RC Buildings in Türkiye.” *Buildings* 14 (5): 1308. https://doi.org/10.3390/buildings14051308.

[15] Cansız, S. 2023. “Analytical Investigation of the Measures to Be Taken against Weak Storey Irregularities.” *Revista de La Construccion* 22 (2): 407–18. https://doi.org/10.7764/RDLC.22.2.407.

[16] Pesaralanka, V., S. P. Challagulla, F. Vicencio, P. S. Chandra Babu, I. Hossain, M. Jameel, and U. Ramakrishna. 2023. “Influence of a Soft Story on the Seismic Response of Non-Structural Components.” *Sustainability* 15 (4). https://doi.org/10.3390/su15042860.

[17] Hareen, C. H. B. V., and S. C. Mohan. 2021. “Evaluation of Seismic Torsional Response of Ductile RC Buildings with Soft First Story.” *Structures* 29: 1640–54. https://doi.org/10.1016/j.istruc.2020.12.031.

[18] Ruiz-García, J., and Y. Cárdenas. 2021. “Seismic Performance Assessment of Weak First-Storey RC Buildings De-signed with Old and New Seismic Provisions for Mexico City.” *Engineering Structures* 232. https://doi.org/10.1016/j.engstruct.2020.111803.

[19] Zakian, P., and A. Kaveh. 2023. “Seismic Design Optimization of Engineering Structures: A Comprehensive Review.” *Acta Mechanica* 234 (4): 1305–30. https://doi.org/10.1007/s00707-022-03470-6.

[20] EN 1998-1. *"Eurocode 8: Design of Structures for Earthquake Resistance – Part 1: General Rules, Seismic Actions and Rules for Buildings*." European Comittee for Standardization: Brussels, Belgium, 2004.

[21] EN 1992-1-1. *"Eurocode 2 - Design of Concrete Structures - Part 1–1: General Rules and Rules for Buildings."* European Comittee for Standardization: Brussels, Belgium, 2004.

[22] prEN1998-1-2:2024. *"Eurocode 8: Design of Structures for Earthquake Resistance - Part 1-2: Rules for New Buildings, Draft Post-ENQ 48th Meeting CEN/TC250/SC8."* European Comittee for Standardization: Brussels, Belgium, 2024.

[23] *"Censos 2021 Resultados Definitivos - Portugal." 2022. Instituto Nacional de Estatística, I.P.*

[24] Maranhão, H., H. Varum, J. Melo, and A. A. Correia. 2024. "Comparative Analysis of the Impact of Design and De-tailing Provisions for RC Moment Resisting Frames under the First- and Second-Generation of Eurocode 8." *Engineering Structures* 306: 117809. https://doi.org/10.1016/j.engstruct.2024.117809.

[25] EN 206. *"Concrete - Part 1: Specification, Performance, Production and Conformity."* European Comittee for Standardization: Brussels, Belgium. 2007.

[26] *EN 10080. "Steel for the Reinforcement of Concrete: Weldable Reinforcing Steel: General."* European Comittee for Standardization: Brussels, Belgium. 2005.

[27] Autodesk. 2024. "Autodesk Robot Structural Analysis Professional: BIM structural analysis software for engineers." 2024. Accessed January 20, 2024. https://help.autodesk.com/view/RSAPRO/2024/ENU/.

[28] EN 1990. *"Eurocode-Basis of Structural Design."* European Committee for Standardization: Brussels, Belgium, 2002.

[29] EN 1991-1-1. *"Eurocode 1: Actions on Structures - Part 1-1: General Actions - Densities, Self-Weight, Imposed Loads for Buildings."* European Comittee for Standardization: Brussels, Belgium, 2002.

[30] Seismosoft. 2021. "SeismoStruct 2021 – A Computer Program for Static and Dynamic Nonlinear Analysis of Framed Structures." www.seismosoft.com.

[31] Seismosoft. 2022. "SeismoStruct Verification Report." www.seismosoft.com.

[32] Dya, A. F. C., and A. W. C. Oretaa. 2015. "Seismic Vulnerability Assessment of Soft Story Irregular Buildings Using Pushover Analysis." *Procedia Engineering* 125: 925–32. https://doi.org/10.1016/j.proeng.2015.11.103.

[33] Chintanapakdee, C., and A. K. Chopra. 2004. "Seismic Response of Vertically Irregular Frames: Response History and Modal Pushover Analyses." *Journal of Structural Engineering*, 1177–85. https://doi.org/10.1061/ASCE0733-94452004130:81177.

[34] Nezhad, M. E., and M. Poursha. 2015. "Seismic Evaluation of Vertically Irregular Building Frames with Stiffness, Strength, Combined-Stiffness-and-Strength and Mass Irregularities." *Earthquake and Structures* 9 (2): 353–73. https://doi.org/10.12989/eas.2015.9.2.353.

[35] Ditommaso, R., N. Lamarucciola, and F. C. Ponzo. 2024. "Prediction of the Fundamental Period of Infilled RC Framed Structures Considering the Maximum Inter-Story Drift at Different Design Limit States." *Structures* 63. https://doi.org/10.1016/j.istruc.2024.106422.

[36] Abd-Alghany, M. M., K. F. El-Kashif, and H. A. Abdalla. 2021. "Seismic Response of Multi-Storey Reinforced Con-crete Buildings with Soft Floor." *HBRC Journal* 17 (1): 407–28. https://doi.org/10.1080/16874048.2021.1949690.

[37] Ibrahim, T. S., and H. J. Mohamad. 2024. "Effects of Soft Story Locations on the Nonlinear Time History Analysis of Multistory Reinforced Concrete Buildings." *Civil and Environmental Engineering* 20 (1): 544–54. https://doi.org/10.2478/cee-2024-0041.

[38] Cabrera, J. L., S. E. Ruiz, and A. Teran-Gilmore. 2024. "Open Ground Story Mid-Rise Buildings Represented by Simplified Systems." *Buildings* 14 (5). https://doi.org/10.3390/buildings14051269.

3.9 NOMENCLATURE

$\varepsilon_{cu,1}$	Ultimate compressive strain of concrete
ε_{sy}	Yielding strain of reinforcement
$\gamma_{G,j}$	Partial factor for permanent action j
$\gamma_{Q,1}$	Partial factor for leading variable action
$\psi_{2,i}$	Combination factor for variable actions
$A_{cce,i,x}$	Seismic accidental torsional effect generated by $E_{Ed,x}$
$A_{cce,i,y}$	Seismic accidental torsional effect generated by $E_{Ed,y}$
A_{Ed}	Design value of seismic action
A_g	Design ground acceleration
DCM	Ductility class medium
E_{cm}	Elastic modulus of concrete
$E_{Ed,x}$	Action effect caused by the imposition of the seismic action along the selected horizontal axis x
$E_{Ed,y}$	Action effect caused by the imposition of the seismic action along the selected horizontal axis y
E_s	Elastic modulus of reinforcement
f_{ck}	Characteristic compressive strength of concrete
f_{cm}	Mean compressive strength of concrete
f_{cd}	Design compressive strength of concrete
$f_{ctk,0.95}$	Characteristic tensile strength of concrete
f_{ctm}	Mean tensile strength of concrete
f_{ctd}	Design tensile strength of concrete
f_{yk}	Characteristic yield strength of reinforcement
f_{ym}	Mean strength of reinforcement
f_{yd}	Design yield strength of reinforcement
$G_{k,j}$	Characteristic permanent action j
MRF	Moment-resisting frame
q	Behaviour factor
Q_k	Characteristic variable action i
S_α	Soil factor
ULS	Ultimate limit state

4 Numerical study on the influence of corrosion and vertical irregularities on seismic behaviour of RC frame structures

Santos, D.; Melo, J.; Furtado, A.; Varum, V. (submitted in press) Numerical study on the influence of corrosion and vertical irregularities on seismic behaviour of RC frame structures.

4.1 ABSTRACT

The structural vulnerability of RC structures during major seismic events raised several concerns regarding structural design and behaviour. Additionally, corrosion´s impact on steel and concrete, including reduction in ductility, confinement and strength, can compromise structural performance, especially for reversal loading. This work investigates the combined effect of corrosion and seismic actions on the structural performance of RC structures. Numerical models of a RC structure with 0%, 5%, 10%, 15% and 20% of corrosion were proposed. The effect of corrosion in the numerical models were calibrated based on experimental studies carry out on corroded RC elements. Afterwards, it was considered the scenario of corrosion in all peripheral structural elements of 5 and 10-storey MRF structures. To enforce vertical irregularity, it has been imposed variation of ground floor height in each structure. An adaptative pushover analysis was performed to assess the effect of corrosion and vertical irregularity in seismic response. The results demonstrate that, from the level of 10% to 20% of corrosion, the uniform corrosion produces deleterious impact in structural response, regardless the level of irregularity in elevation. However, the irregularity generates a higher impact in seismic response than the uniformly distributed corrosion in height. Additionally, the combined effect of those parameters is even worst in seismic vulnerability.

4.2 INTRODUCTION

The structural vulnerability of reinforced concrete (RC) buildings during major seismic events raised several concerns regarding structural design and behaviour, construction methods and existing building maintenance guidelines. Modern RC buildings are designed to establish a global ductile failure mechanism, allowing for anticipated plastic rotations in beams. This design approach mitigates shear collapse by implementing a capacity design procedure and adhering to specific requirements regarding construction details and material properties [1].

Corrosion´s impact on steel and concrete, including reduction in ductility, confinement and strength, can compromise structural performance. It is marked by a decline in strength and, more critically, a decrease in structural displacement capacity, often leading to the change of collapse modality and unforeseen failures. This issue was highlighted in some numerical studies aimed at capturing the effect of corrosion on the seismic performance of the entire building [2–8]. Such a problem can be even higher in existing RC buildings characterised many times by the strong beam/weak column hierarchy typical of design performed only towards vertical loads, frequently exposed to brittle soft-storey collapses [9]. The corrosion on RC structures may change not only their seismic performance but also the behaviour under other lateral loads such as tsunamis [10].

To assess the residual performance of corroded structures, comprehensive research has been conducted over the past few decades, leading to the development of numerous empirical and theoretical relationships between corrosion levels and various performance indicators, including corrosion-induced crack width, bond-slip behaviour, mechanical properties of corroded reinforcement bars, and the load-carrying capacity of corroded elements. However, for these correlations to be applied in structural assessments, the corrosion level must first be accurately quantified, which presents significant challenges for in-service structures compared to laboratory specimens [11].

Recent experimental studies have demonstrated the significance of assessing the impact of corrosion on the strength and, particularly, the displacement capacity of RC columns. This is especially relevant for columns affected by pitting corrosion and those well-confined and designed to exhibit ductile behaviour [7]. Meda et al. [4] indicate that corroded elements experience a reduction in ultimate strength and overall ductility by as much as 50% when the corrosion level reaches around 20% mass loss, even when the confinement loss from stirrup deterioration is disregarded.

This work aims to contribute to that field by performing a numerical analysis of 5-10 storeys MRF (moment-resisting frame) structures. 0%, 5%, 10%, 15% and 20% corrosion have been considered in the adaptative push-over analysis of those structures. To calibrate those models, it was performed a study of models proposed by the experimental work of Meda et al. [4], Goksu et al. [5], Yang et al. [6], Rajput et al. [7] and Rinaldi et al. [8]. Considering those studies, the structural configuration of each percentage of corrosion was proposed, and the existing buildings were assessed based on those characteristics.

This study considered the worst scenario proposed by Caixinhas et al. [12], namely: Extreme conditions in which all façade elements are subjected to corrosion, facilitating a comprehensive assessment of the overall impact of deterioration. Adaptative pushover analyses were conducted in SeismoStruct v2025 software [13] to assess the impact of corrosion on seismic response. The capacity curves, performance points, initial stiffness, yield point, maximum resistance, and inter-story drift profile were discussed and compared.

4.3 CORROSION IMPACT ON SEISMIC BEHAVIOUR

Besides the well-known cross-section reduction, the decrease in the deformation capacity of rebars due to corrosion is evident in reinforcements. Experimental results highlighted residual values of the elongation to maximum load relatively equal to the 50% of the sound performance [14–17], a lower but anyway relevant decrease of ultimate elongation and a negligible deterioration of strength, both at yielding and ultimate level [18,19]. The reduction of ultimate elongation of stirrups acts on RC sections' confinement: together with the progressive reduction of stirrups' diameter, the deterioration of the deformation capacity due to corrosion leads to the loss of confinement and the following decrease of strength and ductility properties at section and element level [9].

Based on the corrosion effects, the decrease in the structural seismic performance of RC structures is often related to the shift of the collapse modality. Modern RC buildings are designed to develop a global ductile failure mechanism with plastic hinges expected in beams, preventing shear collapse by adopting the capacity design procedure and respecting specific requirements for executive details and material characteristics [1]. The effects of corrosion on steel and concrete (reduction of ductility, confinement, strength, etc.) can impair structural response with a decrease in strength and, more importantly, with a reduction of structural displacement capacity, often leading to the change of collapse modality and unexpected failures [2].

Such issue can be even higher in the presence of constructions where no specific attention was devoted to these aspects: this is the case, for example, of existing RC buildings characterised by the intense beam/weak column hierarchy typical of design performed only towards vertical loads, frequently exposed to brittle soft-storey collapses [9].

Experimental studies to assess the cyclic behaviour of corroded RC columns have been carried out by several authors, namely: Meda et al. [4], Goksu et al. [5], Yang et al. [6], Rajput et al. [7] and Rinaldi et al. [8]. Table 4-1 presents the experimental parameters of those works. Figure 4-1 presents the last drift ratio and lateral load regarding the percentage of corrosion. The last drift ratio is defined as the comparison between the last observed drift in the assessed corroded element and the last drift shown in the uncorroded column in each experimental study. The moment of the drift is defined by the significant damage level of the column. From then, it can be seen:

1. The shift of all parameters between studies, from geometrical parameters to concrete and rebars properties, reduced axial load of the columns and also the applied lateral load developed in the assessed experiments;
2. Figure 4-1a presents a clear decreasing tendency of drift ratio when the corrosion increases. Combining the same trend of lateral load illustrated in Figure 1b, it can be inferred that the deleterious impact of corrosion from 10% upwards.

Table 4-1: Experimental parameters of corrosion studies

Experimental parameters	**Meda et al.** [4]	**Goksu et al.** [5]	**Yang et al.** [6]	**Rajput et al.** [7]	**Rinaldi et al.** [8]
Columns dimensions (cm)	30x30	20x30	21x21	30x30	30x30
Columns height (cm)	180	139	100	180	180
Longitudinal rebar rate (%)	0.89%	1.03%	2.31%	1.79%	0.89%
Transversal bars	ø8/30cm	ø8/10cm	ø6/9cm	ø10/8cm	ø8/25cm
Average concrete compressive strength (MPa)	20.0	25.0	46.4	33.0	25.0
Concrete modulus of elasticity (MPa)	30.0	25.0	36.0	27.0	23.5
Reduced Axial Load (%)	22.0%	19.0%	14.0%	12.0%	13.0%
Applied Lateral Load (kN)	63.0	60.7	60.0	121.8	60.0

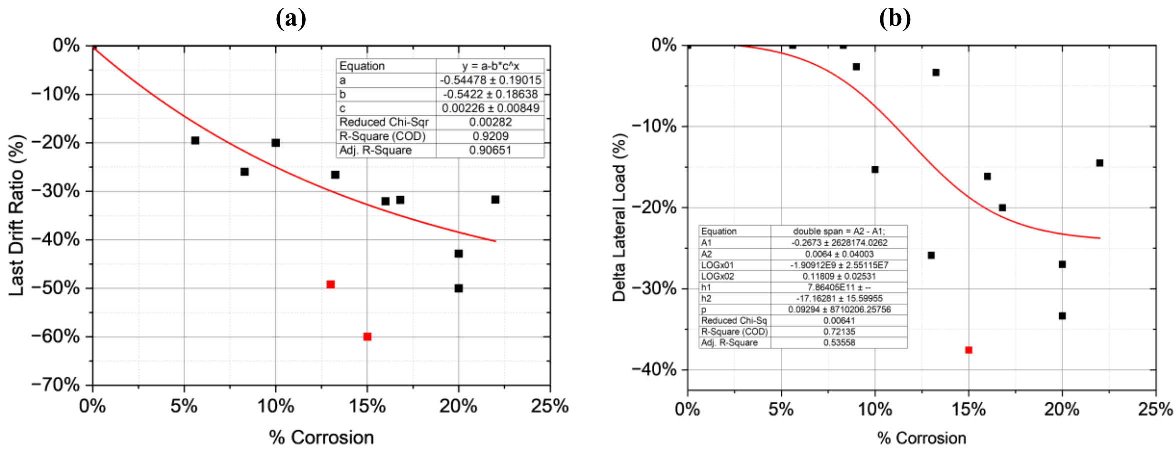


Figure 4-1: Comparison of experimental studies regarding the shift of: (a) last drift ratio and (b) lateral load

4.4 NUMERICAL MODELLING RC MRF STRUCTURES

4.4.1 MODELLING APPROACH

The modelling approach employed in this study for simulating RC elements utilized, similarly to Caixinhas et al. [12], inelastic force-based frame elements subdivided into seven integration sections, with each section further discretized into 204 fibbers to capture the nonlinear material behaviour accurately.

To characterize the behaviour of rebars and concrete under corroded and uncorroded conditions, the uniaxial constitutive model developed by Monti and Nuti [20] was adopted for steel, while the model proposed by Mander et al. [21] was employed for concrete.

To simulate the effects of corrosion on columns and beams, two strategies were employed:

1. Reduction of the effective cross-sectional area of the longitudinal and transverse reinforcement to simulate the material loss associated with the corrosion process in RC elements. The reduction (%) of corrosion represents directly the percentage of cross-section rebars loss due to this process. SeismoStruct v2025 [13], nonetheless, has a limitation in stirrups modelling stirrups: it restricts the bars selection to standard diameters, thereby precluding the use of intermediate sizes. To overcome this limitation, the stirrup spacing was adjusted to obtain the desired equivalent cross-sectional area per meter;
2. Modifying the mechanical properties of the steel, considering the parameters observed in experimental studies of Meda et al. [4], Goksu et al. [5], Yang et al. [6], Rajput et al. [7] and Rinaldi et al. [8]. The calibration process is discussed in next section.

4.4.2 MODEL´S CALIBRATION

Several models have already been developed to simulate the impact of corrosion on structural behaviour. Alipour et al. [22] and Rao et al. [23] are examples of those studies. However, considering the challenges in modelling this problem, their works present some deficiencies, such as considering the impact of corrosion in inelastic bucking and low-cycle fatigue of rebars and its deleterious influence on damage states. Kashani et al. [24] addressed those issues and incorporated the corrosion influence in seismic damage thresholds, as verified later by Dizaj et al. [25].

Figure 4-2 demonstrates the numerical calibration of a column based on Meda et al. [4]. The detailing of the lateral displacement history is shown in Figure 4-1a and the load-drift relationship of uncorroded column and column with 20% corrosion is presented in Figure 4-1b. This approach has been performed by Goksu et al. [5], Yang et al. [6], Rajput et al. [7] and Rinaldi et al. [8], resulting in the models´ materials settings illustrated in Figure 4-3 and Table 4-2 and 4-3.

From those figures and tables the following conclusions can be drawn:

1. Table 4-2 shows the constant material properties settings, notwithstanding the shift of corrosion level. This means that, for those properties, the corrosion level has no influence;
2. Almost all studies demonstrated similar results in steel elasticity modulus (E_s) and transition curve shape calibrating coefficient A1 (a_1) and fracture/bucking strain (P), regardless of the increase in corrosion (200-210, 18,5-19.0 and 0.1372-0.2, respectively). Additionally, for 20% of corrosion, the kinematic/isotropic (k) weighting coefficient and spurious unloading corrective parameter (r (%)) also present similar settings in Meda et al. [4] and Rinaldi et al. [8] (0.9 and 5.0, respectively)
3. Figures 4-3a and 4-3b, on the other hand, demonstrate considerable variation of yield strength and transition curve initial shape parameter when the level of corrosion increases;
4. Considering the discussion performed above, the proposed model settings are demonstrated in Table 4-3. Despite the variation of E_s in the Rajput et al. [7] study, 210 GPa in all corrosion levels was considered a control variable approach. Additionally, f_y (Yield strength) was 520 MPa in 0% and 5% corroded structures. However, this value decreases to 350 MPa and 312 MPa for larger levels of corrosion. Furthermore, R_0 varies between 19.4 to 19,7. Finally, P remained constant (0.137) for 0%-15% of corrosion, changing its value to 0.200 at the level of 20% of corrosion;
5. It is important to highlight the limitation of the discussion performed above due to the restriction of data.

Table 4-2: Constant material properties settings throughout increasing corrosion level

Strain hardening parameter (μ)	**Transition curve shape calibrating coeff. A2 (a_2)**	**Isotropic hardening calibrating coeff. A3 (a_3)**	**Isotropic hardening calibrating coeff. A4 (a_4)**	**Specific weight (kN/m^3) (γ)**
0.015	0.150	0.150	0.100	78

Table 4-3: Proposed material properties settings

Material properties	Corrosion level				
	0%	**5%**	**10%**	**15%**	**20%**
E_s (GPa)	210	210	210	210	210
f_y (MPa)	520	520	350	350	312
μ	0.015	0.015	0.015	0.015	0.015
R_0	19.500	19.500	19.400	19.450	19.700
a_1	19.000	19.000	19.000	19.000	18.500
a_2	0.150	0.150	0.150	0.150	0.150
a_3	0.150	0.150	0.150	0.150	-
a_4	0.100	0.100	0.100	0.100	-
P	0.137	0.137	0.137	0.137	0.200
γ (kN/m^3)	78	78	78	78	78
k	-	-	-	-	0.900
r (%)	-	-	-	-	2.500

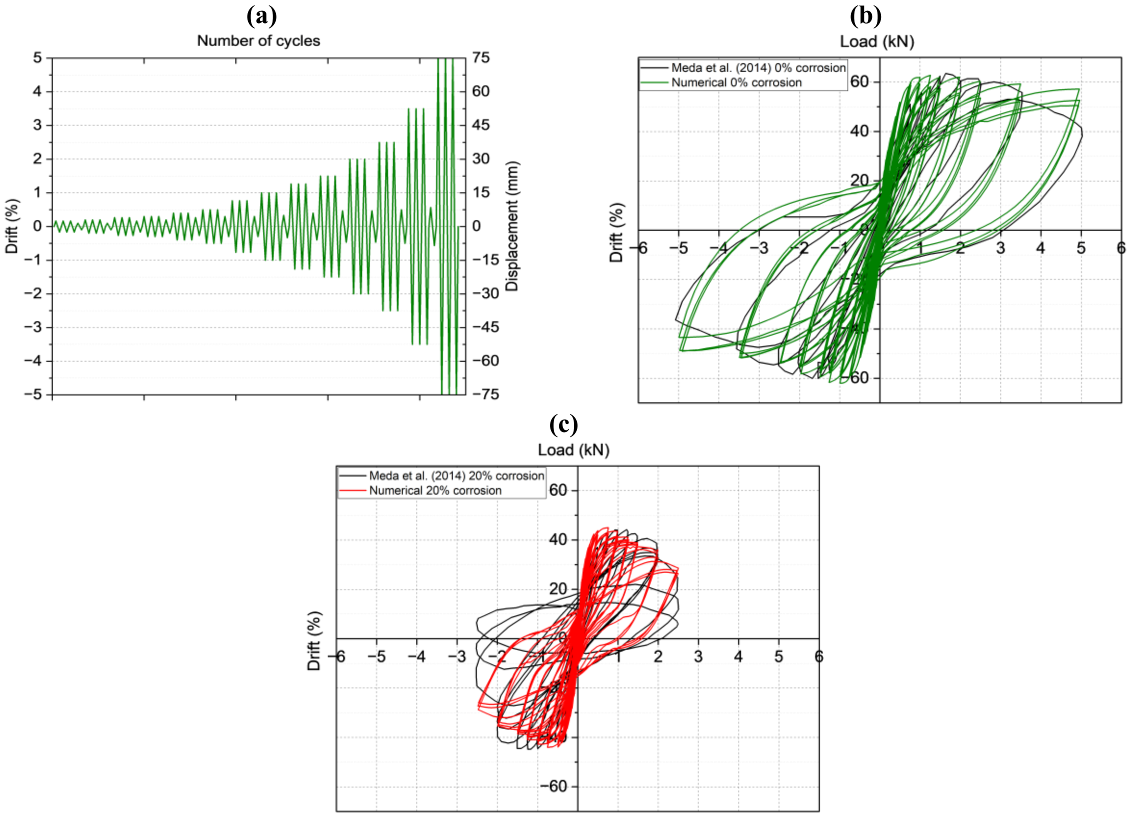


Figure 4-2: Comparison between Meda et al. [4] and proposed numerical modelling: (a) Horizontal displacement history; (b) Horizontal load-drift of uncorroded columns, (c) Horizontal load-drift of 20% corroded columns

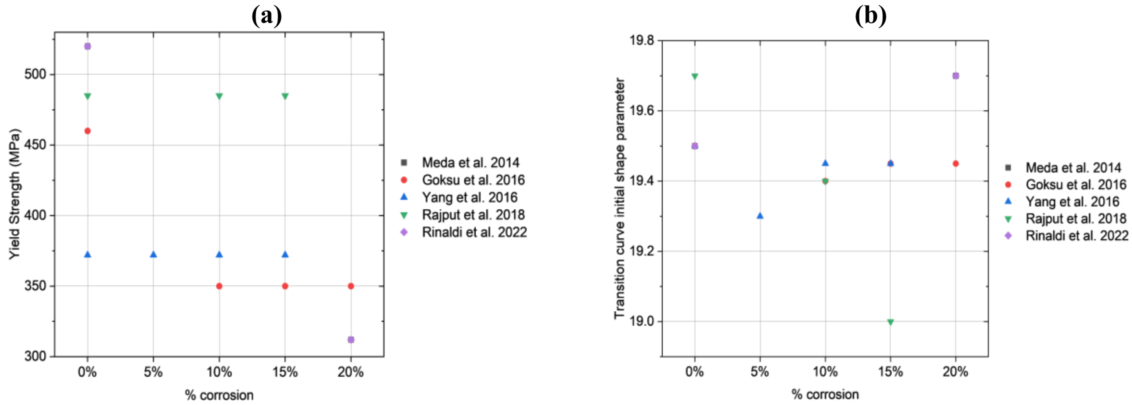


Figure 4-3: Materials properties settings: a) Yield strength (f_y); b) Transition curve initial shape parameter (R_0)

4.5 CASE STUDY

The RC MRF buildings considered in this work were obtained from Santos et al. [26]. The structural system for the buildings consists of 5/10-storey frame RC structures with 3 spans in both directions, spacing of 6.0 m on the X-axis and 7.0 m on the Y-axis. The height between floors is 3.0 m for all storeys, except on the ground floor where the height considered is 3.0 m or 4.0 m or 5.0 m to produce irregular in elevation by changing the lateral stiffness. The slabs had 15 cm of thickness and were treated as rigid diaphragms. Those geometrical properties are demonstrated in Figure 4-4.

The DCM (Ductility class medium) structures were designed according to EN 1990 [27], EN 1998-1 [1] and prEN1998-1-2 [28]. Despite the structures designed with those provisions being considered new structures, these ones will pass through corrosion process during their life cycle. It is crucial to understand the impact of corrosion on seismic performance to better predict their future behaviour.

The models have been named as shown in Figure 4-5. The first part indicates that there are frame structures. Then, the next ones demonstrated the number of storeys of the building. The third part specifies the ground floor height, and finally, the last part indicates the behaviour factor considered in the design regarding the regularity classification of the structure.

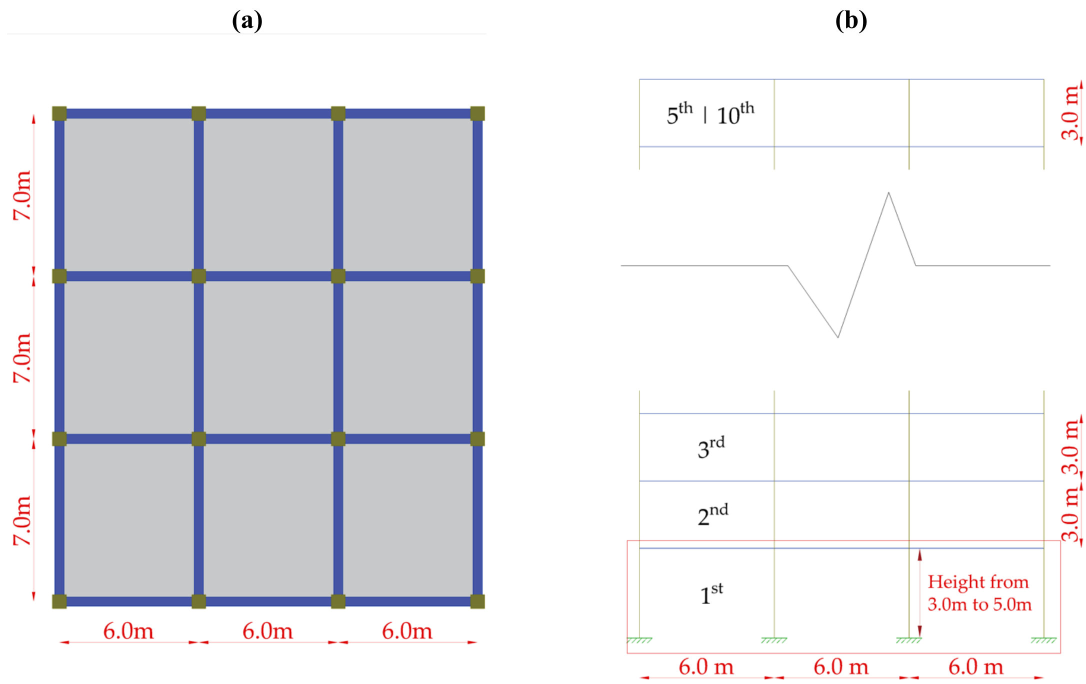


Figure 4-4: Frame building system configurations: a) Plan view; b) 5|10-storey building cutaway

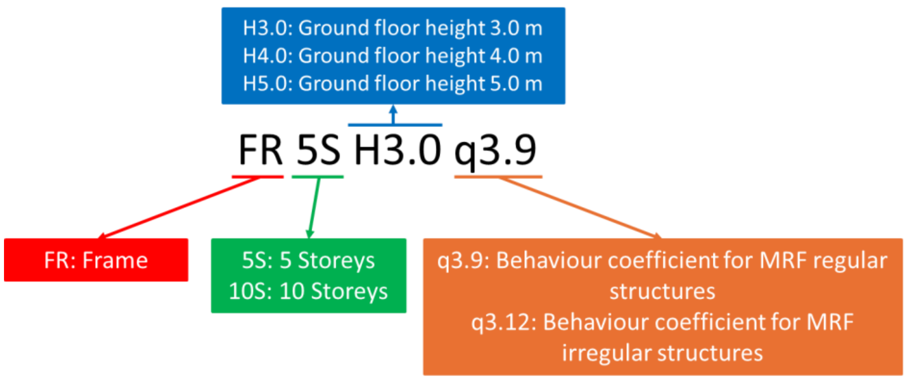


Figure 4-5: Nomenclature of models

The material's characteristics and actions considered in the design were all fully discussed by Santos et al. [26]. The structures were DCM made of C30/37 concrete ($f_c = 38\ MPa$), with properties defined by Eurocode 2 [29] and EN 206-1:2007 [30]. The reinforcement steel is classified as B500 ($f_y = 520\ MPa$), with mechanical properties by EN 10080:2005 [31]. Regarding the loads considered in the design, it was considered, for vertical loads, the office buildings characteristics. Additionally, the aforementioned work specified the approach for seismic load consideration. Those data were considered for the corrosion impact in the seismic response.

Figure 4-6 demonstrates the maximum lateral stiffness shift (Max inter-storey Δk) between storeys observed in each model. Considering that this was the considered control variable regarding the irregularity in elevation imposition, those results were used to classify the structures as regular or irregular in elevation.

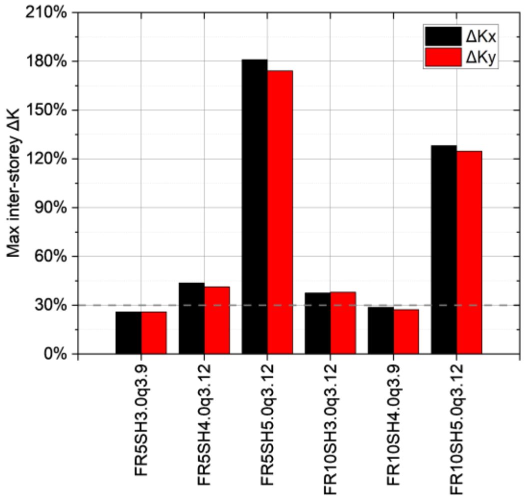


Figure 4-6: Maximum inter-storey lateral stiffness shift in X and Y axis

The corrosion propagation has been considered in all external RC elements, representing the worst environmental condition, according to the worst scenario proposed by Caixinhas et al. [12]. This scenario analysed five corrosion levels: 0%, 5%, 10%, 15% and 20%.

4.6 RESULTS AND DISCUSSION

Figures 4-7, 4-8, 4-9 and 4-10 presents, respectively, the capacity curves, the critical points of those curves, the inter-storey drifts and the limit states demonstrated in the assessed models, considering each level of corrosion. Table 4-4 demonstrates the percentual variation of critical points observed in capacity curves in comparison with each uncorroded structures.

From those figures, the following conclusions can be drawn:

1. From the capacity curves demonstrated in Figure 4-7, the level of corrosion that starts to impact significantly the structural capacity is 10%. Figure 4-8 illustrates the maximum base shear shift compared to each corroded model. It shows this tendency, with exception to FR10SH4.0q3,9 that presents a considerable impact to the level of corrosion of 5%. For the 10%, 15% and 20% of corrosion, a decrease in maximum base shear was observed at around 13.8%-32.0%. When it compared of irregularity in elevation, the lateral stiffness shift and level of corrosion, the maximum base shear behaviour are similar, regardless of the level of irregularity;
2. In Figure 4-8, the corrosion impacts substantially for each structural critical performance criteria:
   a) First yield: All structures behave similarly to each other, presenting a substantial decrease of the first yield appearance from 10% of corrosion. However, FR10SH3,0q3,12 have presented a considerably higher

decrease in that parameter in 10%, 15% and 20% of corrosion. The same occurs for the FR10SH4,0q3,9 for 20% of corrosion;

b) First shear capacity: all structures performed similarly, regardless of irregularity differences. The considerable decrease in shear capacity in comparison to the observed in the respective uncorroded structures starts to occur in 15% of corrosion (31.8%-53.5%) and 20% of corrosion (44.3%-62.3%);

c) First crush uncover: As observed in the aforementioned parameters, the level of 10% of corrosion is the crucial point of impact in this criterium. For 5-storey buildings, the shifts in base shear for 5%, 10%, 15% and 20% of corrosion in comparison to the respective uncorroded structures, were, respectively, 5.0%-6.3%, 19.9%-23.9%, 23.2%-27.7% and 27.4%-32.1%. For 10-storey models, the shifts were, respectively, 2.8%-3.7%, 10.7%-14.8%, 10.6%-14.6% and 16.3%-22.5%. Doing the same assessment for the displacement observed in Figure 7f, the variation, the maximum variation observed was 5.9%;

d) First chord rotation capacity: this criterium behave analogously to the previous one. For 5-storey buildings, the shifts in base shear for 5%, 10%, 15% and 20% of corrosion in comparison to the respective uncorroded structures, were, respectively, 5.8%-6.6%, 21.0%-24.4%, 24.1%-28.3% and 28.4%-32.1%. For 10-storey models, the shifts were, respectively, 3.6%-6.3%, 18.0%-18.6%, 17.7%-18.5% and 25.2%-27.0%. Doing the same analysis for the displacement observed in Figure 7h, the variation, for 5-storey were, respectively, 5.1%-6.1%, 7.7%-9.1%, 9.1%-12.0% and 9.1%-10.3%. For 10-storey buildings, they were, 2.6%-3.3%, 6.7%-7.2%, 2.9%-6.7% and 10.0%-82.1%;

e) First crush confinement: This parameter was observed only in 10 storey structures, but not in all corrosion conditions.

3. Figure 4-9 demonstrated that the corrosion level does not considerably impact the inter-storey drift. However, the irregularity, as expected and demonstrated in Santos et al. [26], impacts substantially that parameter;

4. From Figure 4-10, it can be observed that, for the level of 5% of corrosion, the base shear shift of limit states in each level of corrosion in comparison to respective uncorroded models is no more than 7.0%. However, this variation is considerably higher from 10% to 20% of corrosion (13.3%- 33.6%). It can be also observed that, for 5% of corrosion, the difference of base shear between the limit states level is slight. For the higher level of corrosion, the behaviour is the opposite, except for FR10SH4.0q3,9 structure that presented the slight difference between limit states base shear in all levels of corrosion.

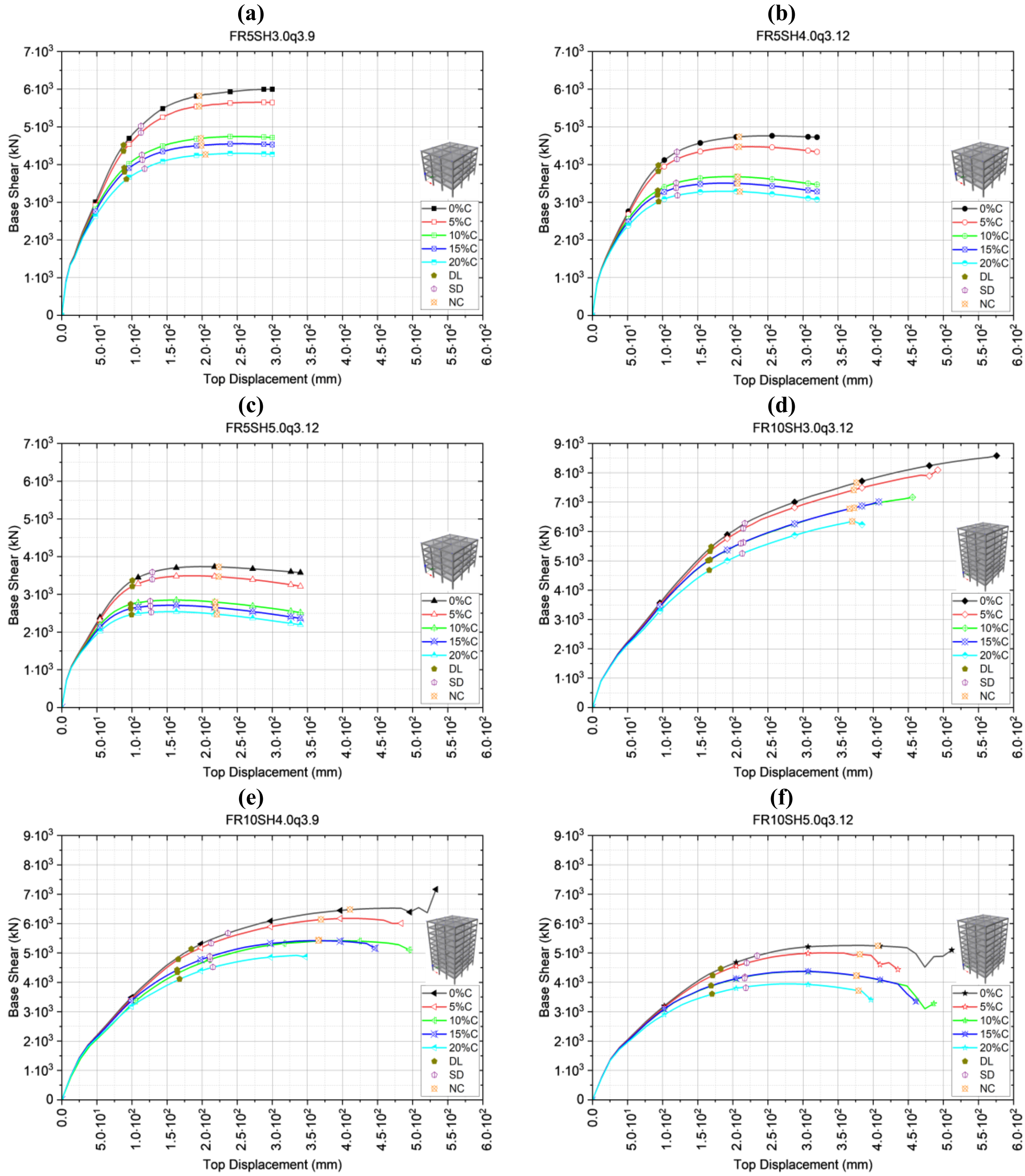


Figure 4-7: 5|10-storey frame structures capacity curves with different levels of corrosion

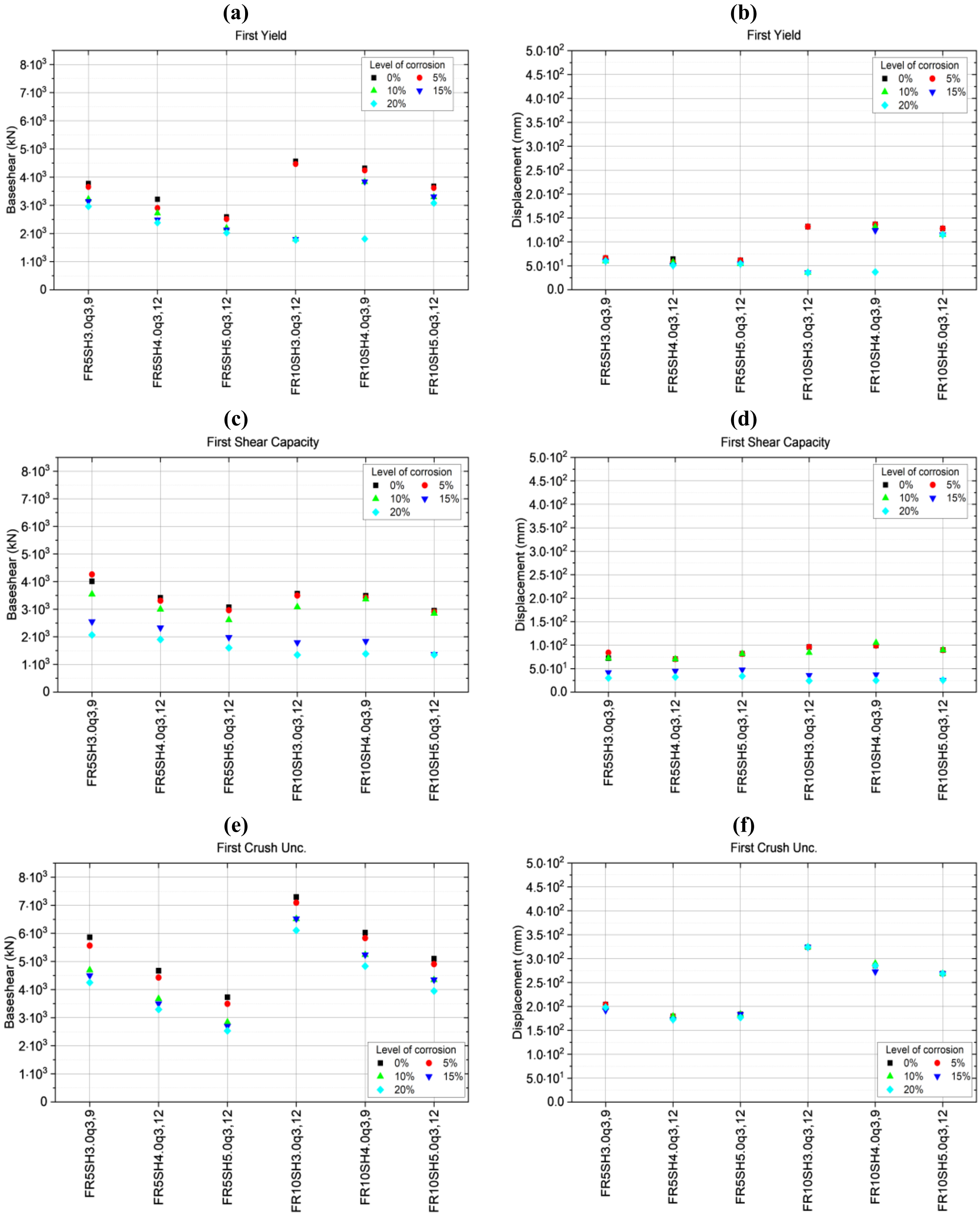


Figure 4-8: Critical points of capacity curves in 5|10-storey frame buildings considering different levels of corrosion

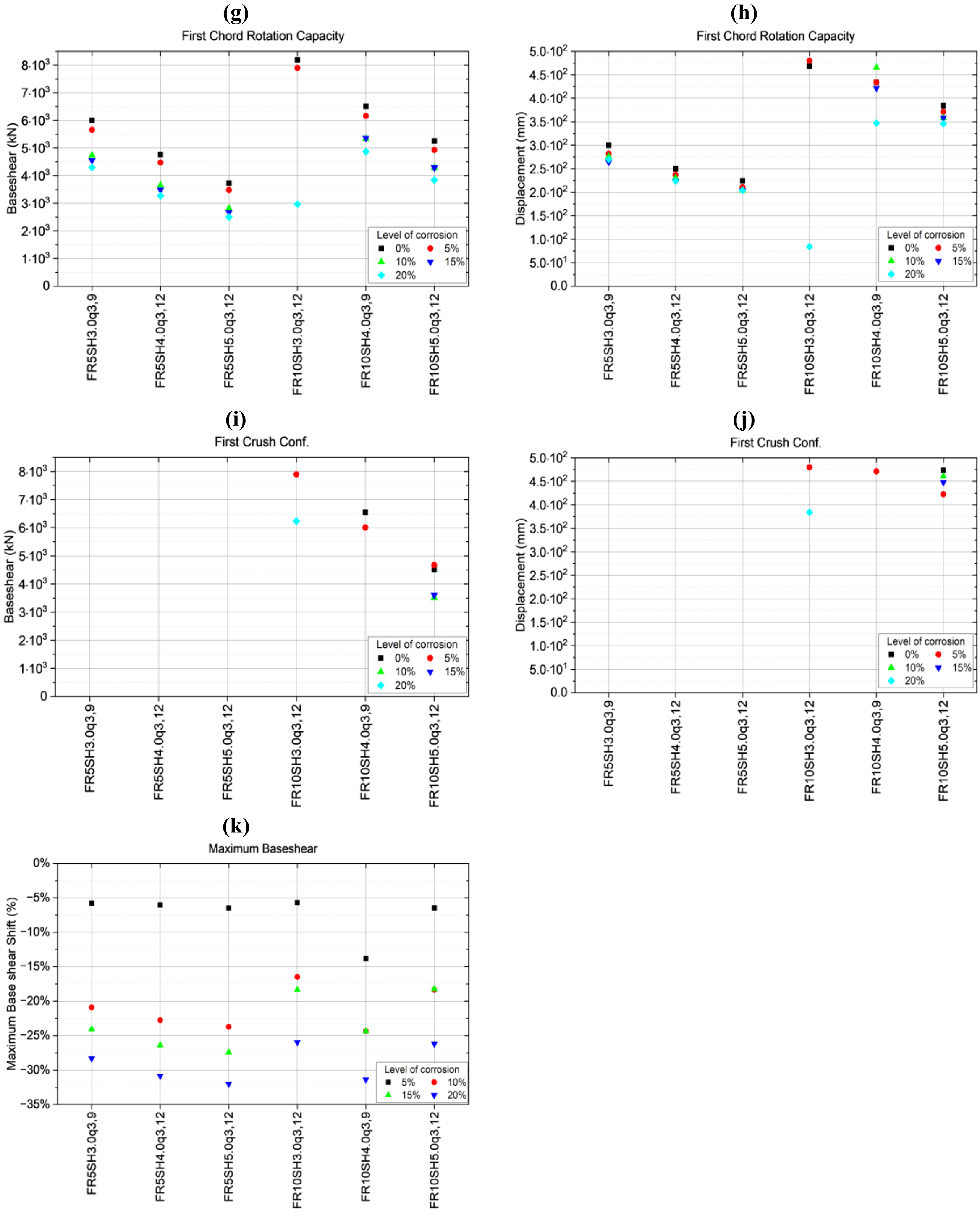


Figure 4-8: Cont.

Table 4-4: Critical points of capacity curves comparison with uncorroded structures on base shear

Critical point of capacity curve	Structure	Level of corrosion			
		5%	10%	15%	20%
First Yield	FR5SH3.0q3,9	-3.2%	-14.7%	-16.8%	-21.6%
	FR5SH4.0q3,12	-9.6%	-15.3%	-22.5%	-25.9%
	FR5SH5.0q3,12	-3.0%	-15.3%	-17.9%	-21.9%
	FR10SH3.0q3,12	-2.2%	-60.7%	-60.6%	-61.3%
	FR10SH4.0q3,9	-1.8%	-11.2%	-11.0%	-58.1%
	FR10SH5.0q3,12	-1.7%	-10.4%	-10.2%	-16.3%
First Shear Capacity	FR5SH3.0q3,9	6.2%	-11.6%	-36.4%	-48.5%
	FR5SH4.0q3,12	-3.2%	-12.2%	-31.8%	-44.3%
	FR5SH5.0q3,12	-3.6%	-15.1%	-35.5%	-47.9%
	FR10SH3.0q3,12	-1.8%	-13.5%	-49.5%	-62.3%
	FR10SH4.0q3,9	-1.8%	-3.5%	-47.2%	-60.3%
	FR10SH5.0q3,12	-1.5%	-3.2%	-53.5%	-54.2%
First Crush Unc.	FR5SH3.0q3,9	-5.0%	-19.9%	-23.2%	-27.4%
	FR5SH4.0q3,12	-5.2%	-21.3%	-25.0%	-29.6%
	FR5SH5.0q3,12	-6.3%	-23.9%	-27.7%	-32.1%
	FR10SH3.0q3,12	-2.8%	-10.7%	-10.6%	-16.3%
	FR10SH4.0q3,9	-3.2%	-13.0%	-13.0%	-19.8%
	FR10SH5.0q3,12	-3.7%	-14.8%	-14.6%	-22.5%
First Chord Rotation Capacity	FR5SH3.0q3,9	-3.5%	-19.1%	-22.4%	-26.7%
	FR5SH4.0q3,12	-4.3%	-21.7%	-25.4%	-30.0%
	FR5SH5.0q3,12	-6.7%	-24.5%	-28.3%	-32.9%
	FR10SH3.0q3,12	8.2%	-	-	-59.4%
	FR10SH4.0q3,9	2.3%	-11.5%	-11.1%	-19.2%
	FR10SH5.0q3,12	-3.3%	-16.0%	-15.9%	-24.7%
First Crush Conf.	FR5SH3.0q3,9	-	-	-	-
	FR5SH4.0q3,12	-	-	-	-
	FR5SH5.0q3,12	-	-	-	-
	FR10SH3.0q3,12	-	-	-	-
	FR10SH4.0q3,9	-8.3%	-	-	-
	FR10SH5.0q3,12	3.4%	-22.2%	-20.0%	-

(a) (b)

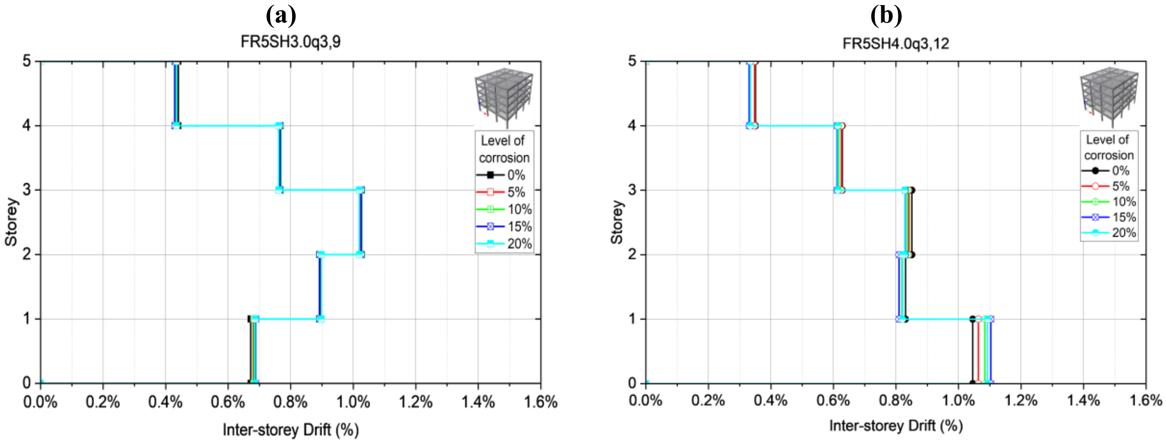
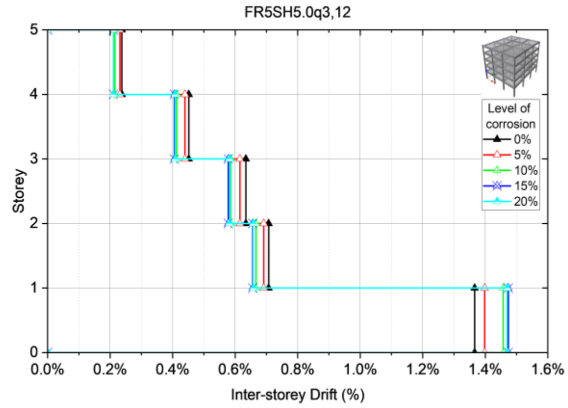
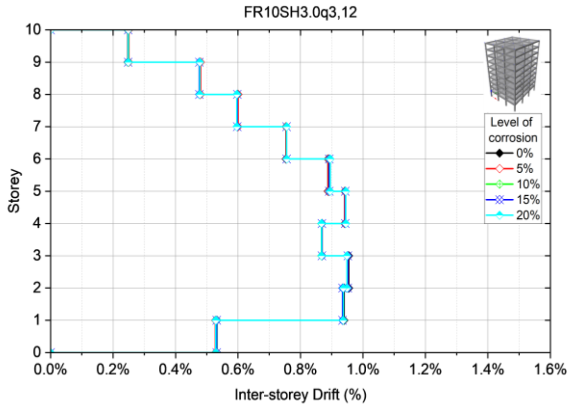


Figure 4-9: Inter-storey drifts in 5|10-storey frame buildings considering different levels of corrosion

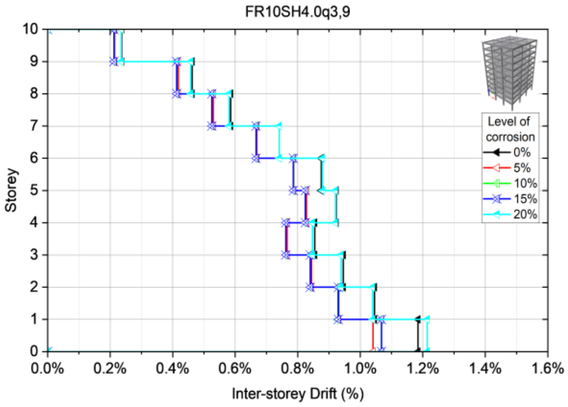
(c)



(d)



(e)



(f)

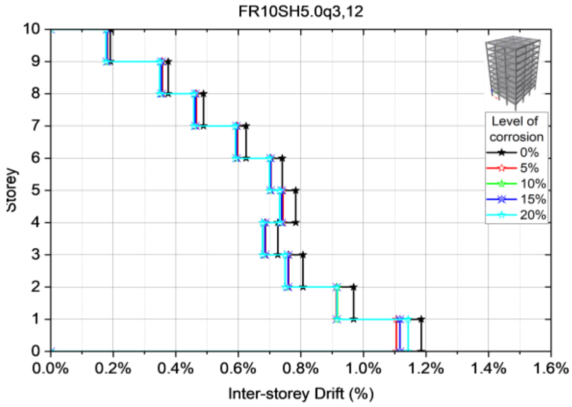
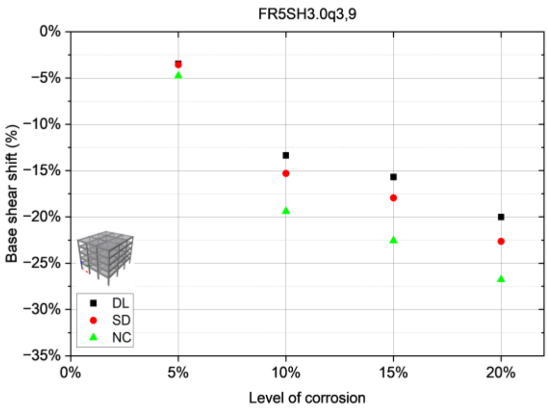


Figure 4-9: Cont.

(a)



(b)

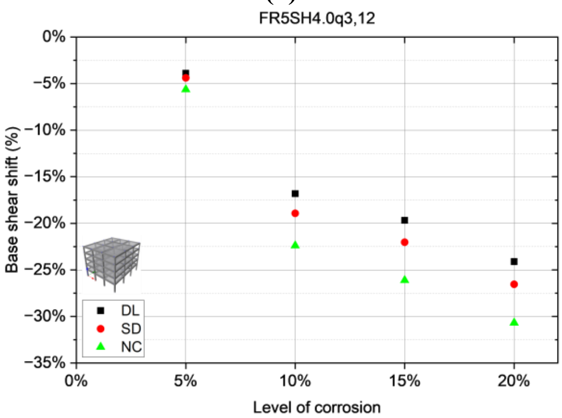


Figure 4-10: 5|10-storey frame structures limit state comparison with corrosion level

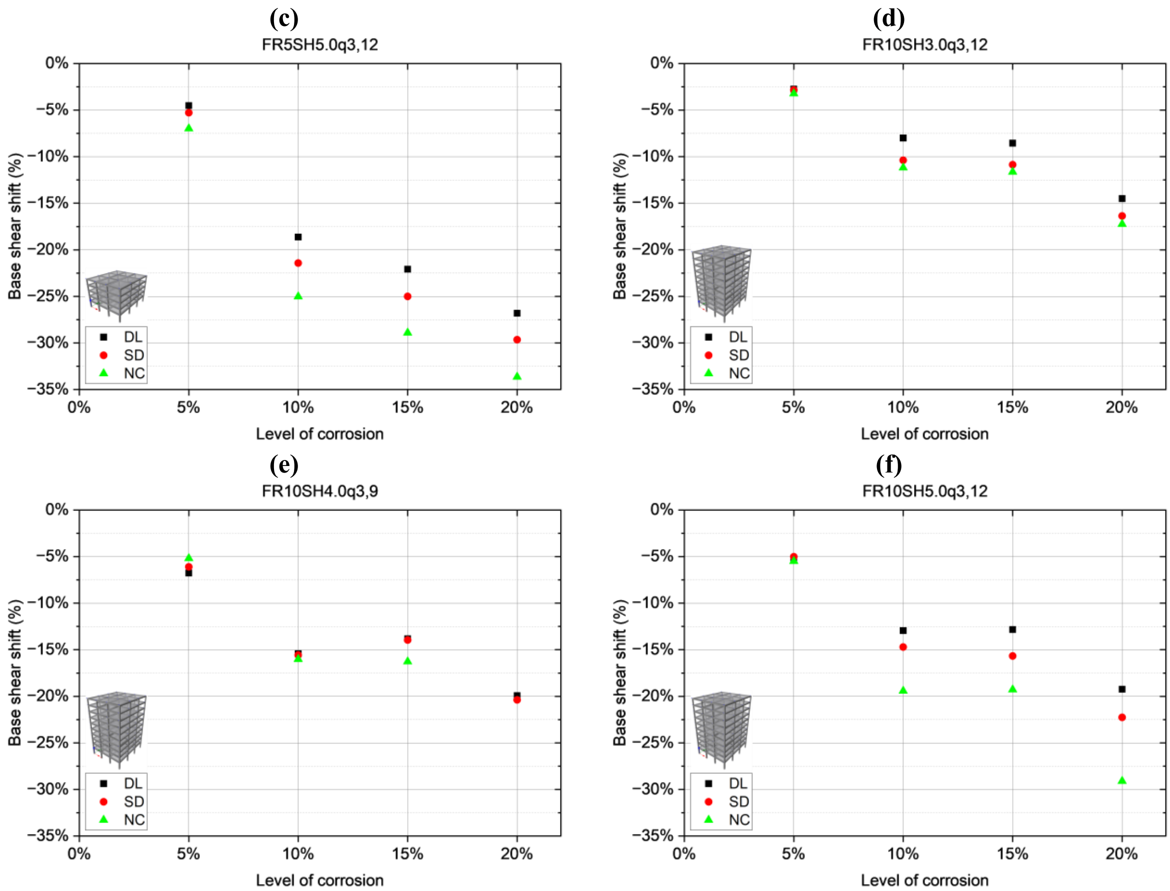


Figure 4-10: Cont.

4.7 SUMMARY AND CONCLUSIONS

RC structures are extensively utilized in contemporary construction. Nonetheless, they may undergo degradation processes caused by corrosion and seismic events throughout their service life. Accurately predicting structural behaviour, considering corrosion and seismic response, remains a considerable challenge in numerical modelling. The existing literature lacks extensive research on modelling strategies that integrate the effects of corrosion, seismic response in reinforced concrete structures and their impacts in vertical irregular structures, highlighting the complexity and underexplored nature of this field.

This work intended to contribute to that field by performing a numerical analysis of 5 and 10 storeys MRF structures, considering 0%, 5%, 10%, 15% and 20% of corrosion in peripheral structural elements. Several experimental studies were considered to calibrate the models in each level of corrosion.

From this work, it can be concluded that:

1. 10% of corrosion was the level that started to present a substantial impact in seismic response in almost all the assessed parameters;
2. Corrosion does not significantly impact the inter-storey drifts once it was considered in all floors, regardless the different levels of vertical irregularities;
3. Comparing the corrosion level and the vertical irregularity imposed in the assessed structures, despite corrosion impacts decreasing structural capacity, the

irregularity in elevation produces worst impacts in seismic response. However, the combined effect substantially increased the seismic vulnerability.

It is important to highlight the limitation of this study. It was just considered RC frame structures. For future studies, it is important to assess also RC shear wall structures, different concrete strengths, structural geometries and, especially the combined impact of irregularity in plan and corrosion in seismic response.

4.8 REFERENCES

[1] EN 1998-1. *“Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings.”* European Comittee for Standardization: Brussels, Belgium: 2004.

[2] Berto, L., R. Vitaliani, A. Saetta, and P. Simioni. 2009. “Seismic Assessment of Existing RC Structures Affected by Degradation Phenomena.” *Structural Safety* 31 (4): 284–97. https://doi.org/10.1016/j.strusafe.2008.09.006.

[3] Afsar Dizaj, E., and M. M. Kashani. 2022. “Nonlinear Structural Performance and Seismic Fragility of Corroded Reinforced Concrete Structures: Modelling Guidelines.” European *Journal of Environmental and Civil Engineering* 26 (11): 5374–5403. https://doi.org/10.1080/19648189.2021.1896582.

[4] Meda, A., S. Mostosi, Z. Rinaldi, and P. Riva. 2014. “Experimental Evaluation of the Corrosion Influence on the Cyclic Behaviour of RC Columns.” *Engineering Structures* 76 (October): 112–23. https://doi.org/10.1016/j.engstruct.2014.06.043.

[5] Goksu, C., and A. Ilki. 2016. “Seismic Behavior of Reinforced Concrete Columns with Corroded Deformed Reinforcing Bars.” *ACI Structural Journal* 113 (5): 1053–64. https://doi.org/10.14359/51689030.

[6] Yang, S. Y., X. B. Song, H. X. Jia, X. Chen, and X. La Liu. 2016. “Experimental Research on Hysteretic Behaviors of Corroded Reinforced Concrete Columns with Different Maximum Amounts of Corrosion of Rebar.” *Construction and Building Materials* 121 (September): 319–27. https://doi.org/10.1016/j.conbuildmat.2016.06.002.

[7] Rajput, A. S., and U. K. Sharma. 2018. “Corroded Reinforced Concrete Columns under Simulated Seismic Loading.” *Engineering Structures* 171: 453–63. https://doi.org/10.1016/j.engstruct.2018.05.097.

[8] Rinaldi, Z., F. Di Carlo, S. Spagnuolo, and A. Meda. 2022. “Influence of Localised Corrosion on the Cyclic Response of Reinforced Concrete Columns.” *Engineering Structures* 256. https://doi.org/10.1016/j.engstruct.2022.114037.

[9] Berto, L., S. Caprili, A. Saetta, W. Salvatore, and D. Talledo. 2020. “Corrosion Effects on the Seismic Response of Existing RC Frames Designed According to Different Building Codes.” *Engineering Structures* 216. https://doi.org/10.1016/j.engstruct.2020.110397.

[10] Zoppo, M. Del, M. Di Ludovico, G. P. Lignola, and A. Prota. 2025. “Time-Dependent Tsunami Fragility Analysis for Reinforced Concrete Building Stock.” *Engineering Structures* 330 (May). https://doi.org/10.1016/j.engstruct.2025.119931.

[11] Hu, J. Y., S. S. Zhang, E. Chen, and W. G. Li. 2022. "A Review on Corrosion Detection and Protection of Existing Reinforced Concrete (RC) Structures." *Construction and Building Materials*. https://doi.org/10.1016/j.conbuildmat.2022.126718.

[12] Caixinhas, A., J. Tomé, J. Melo, G. Marreneca, and A. Furtado. 2025. "Impact of Corrosion on the Behaviour of Reinforced Concrete Buildings." *Buildings* 15 (8): 1267. https://doi.org/10.3390/buildings15081267.

[13] Seismosoft. 2021. "SeismoStruct 2021 – A Computer Program for Static and Dynamic Nonlinear Analysis of Framed Structures." www.seismosoft.com.

[14] Caprili, S., and W. Salvatore. 2015. "Cyclic Behaviour of Uncorroded and Corroded Steel Reinforcing Bars." *Construction and Building Materials* 76: 168–86. https://doi.org/10.1016/j.conbuildmat.2014.11.025.

[15] Zhang, W., X. Song, X. Gu, and S. Li. 2012. "Tensile and Fatigue Behavior of Corroded Rebars." *Construction and Building Materials* 34: 409–17. https://doi.org/10.1016/j.conbuildmat.2012.02.071.

[16] Imperatore, S., Z. Rinaldi, and C. Drago. 2017. "Degradation Relationships for the Mechanical Properties of Corroded Steel Rebars." *Construction and Building Materials* 148 (September): 219–30. https://doi.org/10.1016/j.conbuildmat.2017.04.209.

[17] Finozzi, I., A. Saetta, and H. Budelmann. 2018. "Structural Response of Reinforcing Bars Affected by Pitting Corrosion: Experimental Evaluation." *Construction and Building Materials* 192: 478–88. https://doi.org/10.1016/j.conbuildmat.2018.10.088.

[18] Salvatore, W., S. Caprili, A. Braconi, M. Finetto, L. Bianco, C. Ascanio, J. Moersch, C. Apostolopoulos, and G. Ferreira Pimenra. 2014. "Effects of Corrosion on Low-Cycle Fatigue (Seismic) Behaviour of High-Strength Steel Reinforcing Bars (Rustell).".

[19] Caprili, S., J. Moersch, and W. Salvatore. 2015. "Mechanical Performance versus Corrosion Damage Indicators for Corroded Steel Reinforcing Bars." *Advances in Materials Science and Engineering* 2015. https://doi.org/10.1155/2015/739625.

[20] Monti, G., C. Nuti, and S. Santini. 1996. "CYRUS-Cyclic Response of Upgraded Sections." *University of Chieti, Italy*.

[21] Mander, J. B., M. J. N. Priestley, and R. Park. 1988. "Theoretical Stress-Strain Model for Confined Concrete." *Journal of Structural Engineering* 114 (8): 1804–26. https://doi.org/10.1061/(ASCE)0733-9445(1988)114:8(1804).

[22] Alipour, A., B. Shafei, and M. Shinozuka. 2011. "Performance Evaluation of Deteriorating Highway Bridges Located in High Seismic Areas." *Journal of Bridge Engineering* 16 (5): 597–611.

[23] Rao, A. S., M. D. Lepech, A. S. Kiremidjian, and X. Y. Sun. 2017. "Simplified Structural Deterioration Model for Reinforced Concrete Bridge Piers under Cyclic Loading." *Structure and Infrastructure Engineering* 13 (1): 55–66. https://doi.org/10.1080/15732479.2016.1198402.

[24] Kashani, M. M., L. N. Lowes, A. J. Crewe, and N. A. Alexander. 2016. "Nonlinear Fibre Element Modelling of RC Bridge Piers Considering Inelastic Buckling of Reinforcement." *Engineering Structures* 116: 163–77. https://doi.org/10.1016/j.engstruct.2016.02.051.

[25] Afsar Dizaj, E., R. Madandoust, and M. M. Kashani. 2018. "Exploring the Impact of Chloride-Induced Corrosion on Seismic Damage Limit States and Residual Capacity of Reinforced Concrete Structures." *Structure and Infrastructure Engineering* 14 (6): 714–29.

[26] Santos, D., J. Melo, and H. Varum. 2024. "Comparative Analysis of the Impact of Vertical Irregularities on Reinforced Concrete Moment-Resisting Frame Structures According to Eurocode 8." *Buildings* 14 (9). https://doi.org/10.3390/buildings14092982.

[27] EN 1990. *"Eurocode - Basis of structural design."* European Comittee for Standardization: Brussels, Belgium, 2002.

[28] prEN 1998-1-2. *"Eurocode 8: Design of Structures for Earthquake Resistance - Part 1-2: Rules for New Buildings, Draft Post-ENQ 48th Meeting CEN/TC250/SC8."* European Comittee for Standardization: Brussels, Belgium, 2024.

[29] EN 1992-1-1. "*Eurocode 2 - Design of concrete structures - Part 1–1: General rules and rules for buildings.*" European Comittee for Standardization: Brussels, Belgium, 2004.

[30] EN 206. "*Concrete - Part 1: Specification, performance, production and conformity.*" European Comittee for Standardization: Brussels, Belgium, 2007.

[31] EN 10080. "*Steel for the reinforcement of concrete: weldable reinforcing steel: general.*" European Comittee for Standardization: Brussels, Belgium, 2005.

4.9 NOMENCLATURE

a_1, a_2, a_3, a_4	Transition curve shape calibrating coefficients A1, A2, A3 and A4, respectively
E_s	Steel elasticity modulus
f_c	
f_y	Steel yield strength
k	Kinematic/isotropic weighting coefficient
P	Fracture/bucking strain
r (%)	Spurious unloading corrective parameter
R_0	Transition curve initial shape parameter
μ	Strain hardening parameter
γ	Specific weight (kN/m^3)
DCM	Ductility class medium
DL, SD, NC	Limit states: Damage limitation, significant damage and near collapse, respectively
MRF	Moment-resisting frame
RC	Reinforced concrete

5 Vertical irregularity in RC MRF and wall structures: Evaluation and proposal of irregularity levels and impacts on behaviour factor

Santos, D.; Melo, J.; Maranhão, H.; Varum, V. (submitted in press) Vertical irregularity in RC MRF and wall structures: Evaluation and proposal of irregularity levels and impacts on behaviour factor.

5.1 Abstract

Eurocode 8 is currently undergoing revision on several aspects, such as ductility classes, damage limitation thresholds, local ductility requirements and their corresponding detailing provisions, and criteria for structural irregularities. This work assessed the impact of irregularity in elevation in seismic response by analyzing the plan's regular DCM frame and wall structures. The increase in ground floor height implemented the irregularity to produce shifts in the inter-storey lateral stiffness. The design of those models was performed following the current generation Eurocode 8 (EC8) guidelines, using ETABS22® and the nonlinear analysis was assessed by an adaptative pushover analysis in SeismoStruct v2025. Additionally, this study examines the changes in behaviour factor calculus methodology in the first and second generations of EC8 in those models. The main purpose of this work is to propose an intermediate behaviour factor, considering the hypothesis that different levels of seismic response of irregular structures exist. The outcomes of this research revealed that, for DCM structures, irregular elevation impacts the seismic response substantially. Moreover, it is proposed to the second generation of Eurocode 8 a classification of regular, irregular and strongly irregular structures, with a correspondent reduction of 20% of (q) for irregular structures and 30% for strongly irregular structures.

5.2 Introduction

The first generation of Eurocode 8 (EN 1998-1:2004) [1], together with Eurocode 2 (EN 1992-1-1:2004) [2], established the foundational framework for structural analysis and design, including the modelling requirements for stiffness, strength, and ductility. These standards provide methodologies aimed at ensuring adequate structural performance under seismic loading. The second generation of Eurocode 8 (prEN 1998-1-2:2024) [3], currently under development, introduces several significant updates. Among them, the revised definition of the behaviour factor (q) is particularly significant, as it explicitly incorporates the influence of vertical irregularities in structural configurations, enhancing the link between seismic design criteria and structural geometry in elevation.

Several studies have assessed the impact of behaviour and ductility factors on structural seismic performance, highlighting their crucial role. Maranhão et al. [4] assessed, respectively, the impact of the second generation of prEN 1998-1-2:2024 [3] provisions in design and detailing provisions for moment-resisting frames (MRF) structures, including the study of 5 and 10-storey MRF regular in plan and in elevation buildings design in all prescribed ductility classes from that code. In a later work, Maranhão et al. [5] also examined, including those previously studied, the repercussions of the changes in the analysis methodology in the behaviour factor and chord rotation verification according to the prEN1998-1-2:2024 [3].

Fardis et al. [6] conducted a comparative study between the first and second generations of Eurocode 8, using three design variants of a six-storey frame building with two basement levels.

The current version of Eurocode 8, EN 1998-1:2004 [1], does not provide a quantitative criterion for evaluating vertical irregularity in structures. In contrast, the second generation of Eurocode, prEN 1998-1-2:2024 [3], introduces a quantitative approach to assess each criteria of vertical irregularity. In this context, MRF and wall structures were designed for DCM (Ductility Class Medium), and an intermediate level of vertical irregularity is proposed. This allows an evaluation of the corresponding impact of vertical irregularity on the behaviour factor, providing insights into how stiffness irregularities in elevation influence seismic performance regarding both generations of Eurocode 8.

The main purpose of this work is to propose an intermediate level of irregularity and its corresponding impact on the behaviour factor, considering the hypothesis that different levels of seismic response of irregular structures exist.

This proposal is based on the hypothesis that different degrees of vertical irregularity can lead to distinct seismic responses in structures. The innovation of this work lies in the detailed assessment of the effects of vertical irregularity—caused by shifts in lateral stiffness—on the seismic performance of MRF and wall structures. The study considers the vertical irregularity provisions of prEN 1998-1-2:2024 [3] and introduces a classification of varying levels of vertical irregularity in elevation and their respective influence on the behaviour factor.

In the present study, 12 MRF and 24 wall structures, all designed for Ductility Class Medium (DCM) and regular in plan, were developed with varying ground floor heights ranging from 3.0 m to 5.5 m. All structures were non-torsionally flexible and designed per the provisions of EN 1992-1-1:2004 [2] and EN 1998-1:2004 [1]. The criteria established by prEN 1998-1-2:2024 [3] were adopted to evaluate vertical irregularity in elevation. Adaptive pushover analyses were performed on the designed structures to assess the behaviour factor. Vertical irregularity was introduced by varying the lateral stiffness in elevation, which was used as the control parameter. Other potential sources of irregularity—such as setbacks, mass distribution, and resistance variation—were kept within the limits defined in prEN 1998-1-2:2024 [3], ensuring that they did not contribute to irregularity.

5.3 BUILDINGS DESIGN

In this section, the structural design of Buildings is presented to enable a subsequent nonlinear analysis. The design process followed established seismic design principles to ensure realistic and code-compliant structural configurations. This serves as a basis for assessing the building's nonlinear behaviour under seismic loading in the following stages of the study.

5.3.1 BUILDING GEOMETRY AND STRUCTURAL SYSTEM

This study examines two types of building configurations, differing in plan geometry as follows and illustrated in Figure 5-1:

- Moment Resisting Frame (MRF) structures with 5 and 10 storeys and
- Wall structures with 5, 10, 15, and 20 storeys.

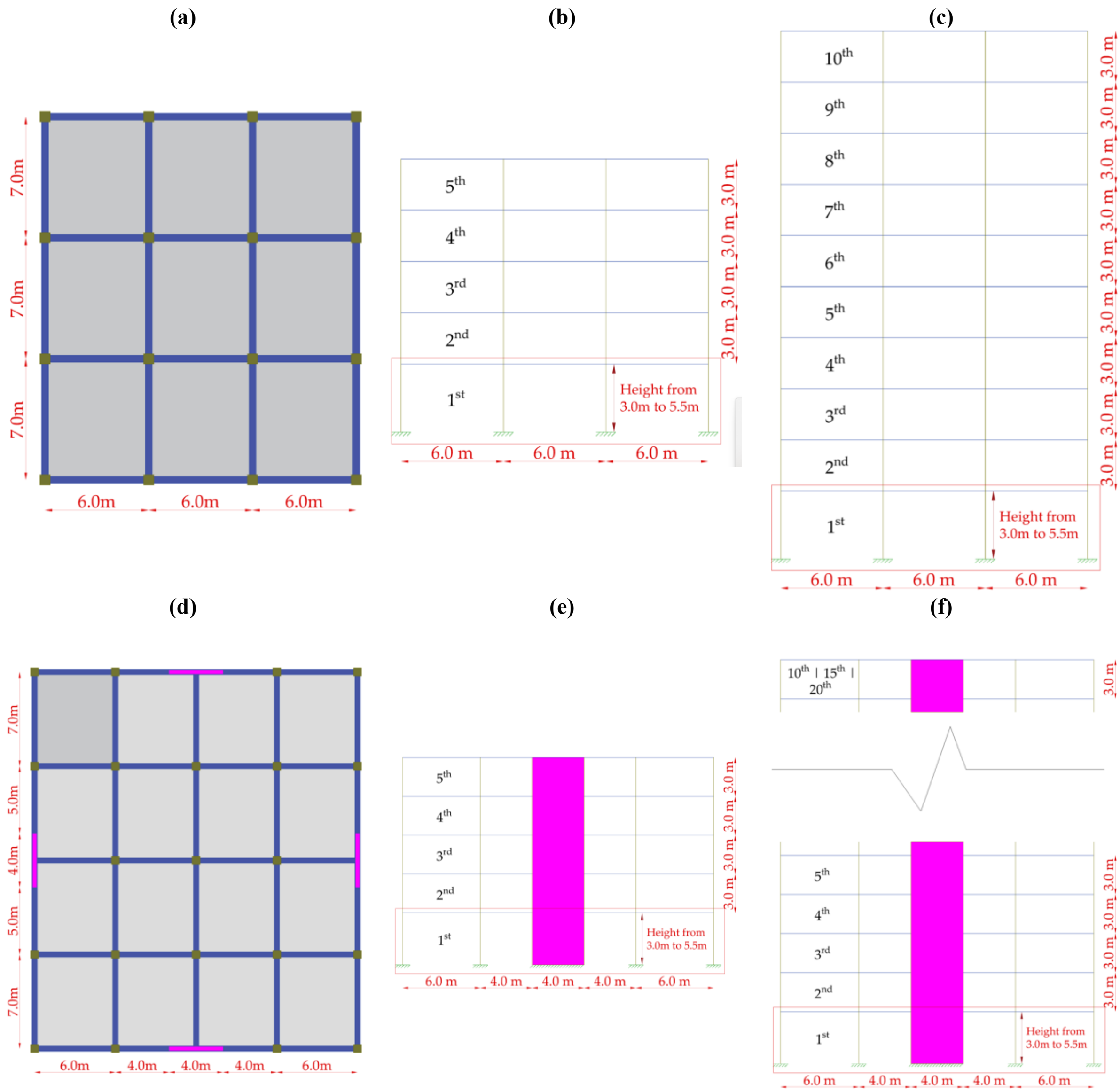


Figure 5-1: (a) Planview of MRF structures; (b) cutaway of 5-storey MRF structures; (c) cutaway of 10-storey MRF structures; (d) planview of wall structures; (e) cutaway of 5-storey wall structures and (f) cutaway of 10, 15 and 20-storey wall structures

The structural system for the buildings consists of a series of MRF and wall structures spaced 6.0 m on the X-axis and 7.0 m on the Y-axis, typical of residential buildings, as illustrated in Figure 5-1 (a and d, respectively). MRF and wall structures are present, respectively, with 3 and 4 bays in both directions, respectively. The height between floors is 3.0 m for all storeys. The models had their ground floor heights increased by 0.5 m to a maximum of 5.5 m elevation to produce irregularity elevation by changing the lateral stiffness. Table 5-1 presents the main geometrical characteristics of each analyzed structure.

The structures have been named as disposed in Table 5-1. The first two letters define the structural type (MRF or wall structure). Then, the third and fourth elements specify the structure's number of storeys. The third part of the nomenclature indicates the height of the ground floor, while the final part specifies the behaviour factor considered in the structural design.

Table 5-1: Adopted nomenclature for the structures

Number of storeys	Structural type	S_a m/s^2	Number of bays		Vertical regularity	Structure label
			X	Y		
5	MRF structure	7.5	3	3	Regular	FR5SH3.0q3.9
					Regular	FR5SH3.5q3.9
					Irregular	FR5SH4.0q3.12
					Irregular	FR5SH4.5q3.12
					Irregular	FR5SH5.0q3.12
					Irregular	FR5SH5.5q3.12
	Wall structure		4	4	Regular	WA5SH3.0q3.0
					Regular	WA5SH3.5q3.0
					Regular	WA5SH4.0q3.0
					Irregular	WA5SH4.5q2.4
					Irregular	WA5SH5.0q2.4
					Irregular	WA5SH5.5q2.4
10	MRF structure	7.5	3	3	Irregular	FR10SH3.0q3.12
					Regular	FR10SH3.5q3.9
					Regular	FR10SH4.0q3.9
					Irregular	FR10SH4.5q3.12
					Irregular	FR10SH5.0q3.12
					Irregular	FR10SH5.5q3.12
	Wall structure		4	4	Regular	WA10SH3.0q3.0
					Regular	WA10SH3.5q3.0
					Irregular	WA10SH4.0q2.4
					Irregular	WA10SH4.5q2.4
					Irregular	WA10SH5.0q2.4
					Irregular	WA10SH5.5q2.4
15	Wall structure	7.5	4	4	Regular	WA15SH3.0q3.0
					Regular	WA15SH3.5q3.0
					Irregular	WA15SH4.0q2.4
					Irregular	WA15SH4.5q2.4
					Irregular	WA15SH5.0q2.4
					Irregular	WA15SH5.5q2.4
20	Wall structure	7.5	4	4	Regular	WA20SH3.0q3.0
					Irregular	WA20SH3.5q2.4
					Irregular	WA20SH4.0q2.4
					Irregular	WA20SH4.5q2.4
					Irregular	WA20SH5.0q2.4
					Irregular	WA20SH5.5q2.4

5.3.2 DESIGN CRITERIA AND MODELLING

5.3.2.1 SIZING AND MODELLING

The specifications in EN 1998-1:2004 [1] and prEN 1998-1-2:2024 [3] were meticulously considered in the structural design of all structures. It was carried out by Finite Element (FE) modelling in ETABS22® [7].

Santos et al. [8] defined the models' beams, columns, and shear walls under analysis using concrete grade C30/37 and steel reinforcement grade B500. The concrete´s properties were defined according to EN 1992-1-1:2004 [2] and EN 206-1:2007 [9] standards. The assumed nominal concrete cover was 30 mm.

Similarly to Santos et al. [8], the characteristics of the models can be delineated as follows:

1. It was considered earthquake forces in X and Y directions;
2. Global torsional effects were considered in models;
3. Slabs were modelled as rigid diaphragms;
4. Columns, shear walls and beams were modelled as linear elastic bar elements. However, joint stiffness was not explicitly represented in this analysis;
5. No redistribution of bending moments was considered in this modelling approach;
6. It was assumed that the connections among beams, columns, and shear walls were concentric;
7. The design incorporated both accidental eccentricities and second-order effects;
8. The structural response excluded the influence of infill walls, although their weight was incorporated into the overall loads;
9. A fully restrained condition was assumed for the foundation support.

5.3.2.2 LOAD DEFINITION

The vertical loads of office buildings were considered in all evaluated structures following EN 1990:2002 [10] and EN 1991-1:2002 [11]. These considerations are summarized as follows:

1. Permanent loads:
    a. Structure self-weight: 25 kN/m^3;
    b. External walls and partitions: 1.2 kN/m^2, except for the roof;
    c. Storeys finishing: 1.3 kN/m^2, except for the roof (1.0kN/m^2).
2. Live load: 2.0 kN/m^2 is prescribed for buildings categorized as civil dwellings in usage category A, except for the roof that is considered accessible only for maintenance and repair, accounting for 1.0 kN/m^2.

Regarding horizontal loads, they were only considered seismic demands, dismissing wind, snow, and thermal actions. That was established to make seismic demands paramount in influencing the design.

Concerning the parameters that define the design spectra, all structures were considered as DCM, ground type B, and importance class II. Finally, the combinations of actions were assumed as defined by Santos et al. [8], considering meticulously the guidelines of EN 1998-1:2004 [1] and EN 1990:2002 [10].

5.3.3 VERTICAL IRREGULARITY ASSESSMENT

To assess vertical irregularity, the control variable adopted was the lateral stiffness ratio between consecutive storeys, a parameter highly indicative of irregularity in elevation, as demonstrated in several studies [8,12–14]. The design procedure is defined as follows:

1. A preliminary design of the structures was carried out, assuming an arbitrary classification of regularity in elevation.
2. The lateral stiffness to each storey of all structures was calculated on ETABS22®. Following the procedure by Santos et al. [8], unit lateral forces were applied at each storey's centre of mass position, while nodes of columns and shear walls at the lower storeys were fixed. The slope obtained for the force-displacement relationship represents the lateral stiffness of the storey. This procedure was applied to all storeys, with the lateral stiffness determined as the sum of all calculated forces required to produce a unit displacement. Regarding the vertical irregularity classification, lateral stiffness shift was considered the control variable. EN1998-1:2004 [1] does not provide a specific numerical limit to that variation. Nevertheless, prEN 1998-1-2:2024 [3] establishes a maximum shift in lateral stiffness to regular structures of 30%. That limit was considered in the vertical irregularity classification;
3. Finally, the structures were designed, considering the classification of regular or irregular structures regarding the abovementioned criteria.

The maximum lateral stiffness variation between consecutive storeys for each structure is presented in Figure 5-2. The structures with a stiffness variation below 30% (classified as regular in elevation) are shown in green bars, while those exceeding this threshold (classified as irregular) are shown in red bars.

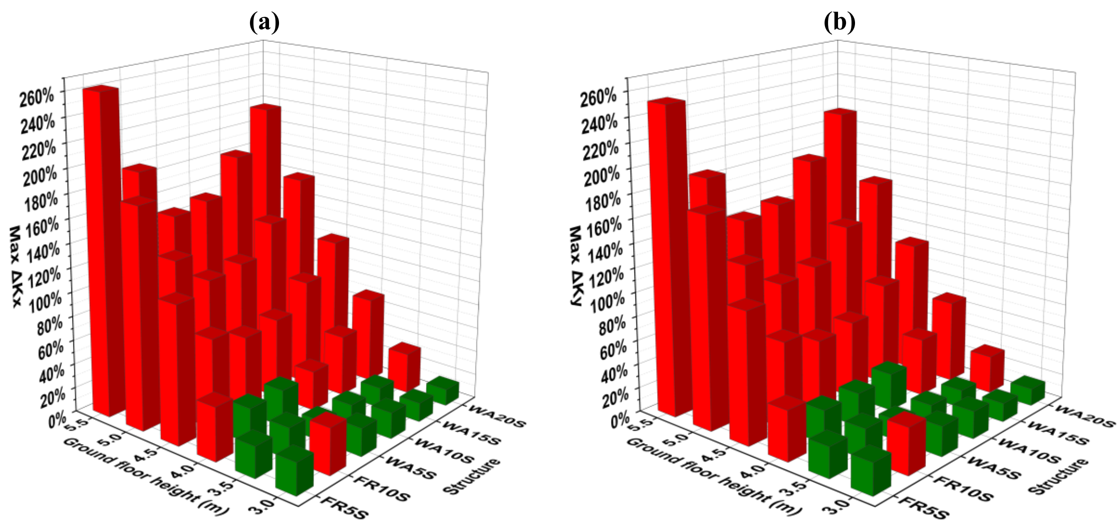


Figure 5-2: Maximum lateral stiffness variation between storeys in: (a) X-axis, (b) Y-axis

5.3.4 BUILDING DESIGN RESULTS

Table 5-2 presents the geometric and reinforcement characteristics of the designed MRF and RC wall structures considered in this study. The structures range from 5 to 20 storeys and were designed with varying ground floor heights (from 3.0 m to 5.5 m) to introduce different levels of vertical irregularity. All structures are regular in plan and follow the provisions of the DCM (Ductility Class Medium). The table details the dimensions and longitudinal reinforcement ratios of columns, beams, and shear walls (when applicable), along with the corresponding slab thicknesses for each storey range. The lateral stiffness variation across storeys—particularly at lower levels—was intentionally introduced to evaluate its effect on seismic response and behaviour factors. The abbreviation N/A indicates that shear walls are not present in the corresponding MRF structures.

Table 5-2: Geometric and reinforcement properties of the elements for the designed MRF and RC wall structures

Structure	Storey	Columns		Shear Walls		Beams		Slabs thickness (cm)
		Section (cmxcm)	Long. rebar Ratio (*r*)	Section (cmxcm)	Long. rebar Ratio (*r*)	Section (cmxcm)	Long. rebar ratio (*r*)	
FR5SH3.0q3.9 FR5SH3.5q3.9 FR5SH4.0q3.12 FR5SH4.5q3.12 FR5SH5.0q3.12 FR5SH5.5q3.12	Up to storey 2	55x55	1.06%	N/A	N/A	40x55	1.57%-1.29%	15
	Storey 2 to 4	50x50	0.97%			40x50	1.41%-1.26%	
	Storey 4 to 5	45x45	1.19%			35x45	1.60%	
FR10SH3.0q3.12 FR10SH3.5q3.9 FR10SH4.0q3.9 FR10SH4.5q3.12 FR10SH5.0q3.12 FR10SH5.5q3.12	Storey 1	60x60	1.05%	N/A	N/A	40x55	1.14%	15
	Storey 2 to 4	55x55	1.25%					
	Storey 5 to 8	50x50	1.51%			40x50	1.41%-1.26%	
	Storey 9 to 10	45x45	1.86%			35x45	1.80%-1.60%	
WA5SH3.0q3.0 WA5SH3.5q3.0 WA5SH4.0q3.0 WA5SH4.5q2.4 WA5SH5.0q2.4 WA5SH5.5q2.4	Up to storey 2	55x55	1.06%	400x30	0.45%	40x55	2.14%-1.14%	15
	Storey 2 to 4	50x50	0.97%			40x50	2.20%-1.41%	
	Storey 4 to 5	45x45	1.19%			35x45	1.60%	
WA10SH3.0q3.0 WA10SH3.5q3.0 WA10SH4.0q2.4 WA10SH4.5q2.4 WA10SH5.0q2.4 WA10SH5.5q2.4	Storey 1	60x60	1.05%	400x30	0.45%	40x55	1.14%	15
	Storey 2 to 4	55x55	1.25%					
	Storey 5 to 8	50x50	1.51%			40x50	1.41%-1.26%	
	Storey 9 to 10	45x45	1.86%			35x45	1.80%-1.60%	

N/A: Not applicable

Table 5-2: Cont.

Structure	Storey	Columns		Shear Walls		Beams		Slabs thickness (cm)
		Section (cmxcm)	Long. rebar Ratio (r)	Section (cmxcm)	Long. rebar Ratio (r)	Section (cmxcm)	Long. rebar ratio (r)	
WA15SH3.0q3.0 WA15SH3.5q3.0 WA15SH4.0q2.4 WA15SH4.5q2.4 WA15SH5.0q2.4 WA15SH5.5q2.4	Storey 1	75x75	0.67%	400x30	0.45%	40x55	2.28%-1.41%	15
	Storey 2 to 3	70x70	0.77%					
	Storey 4 to 5	65x65	0.89%					
	Storey 6 to 7	60x60	1.05%					
	Storey 8 to 9	55x55	1.25%					
	Storey 10 to 13	50x50	1.51%			40x50	1.73%-1.26%	
	Storey 14 to 15	45x45	1.86%			35x45	1.80%-1.60%	
WA20SH3.0q3.0 WA20SH3.5q2.4 WA20SH4.0q2.4 WA20SH4.5q2.4 WA20SH5.0q2.4 WA20SH5.5q2.4	Storey 1 to 2	85x85	0.52%	400x30	0.77%	40x55	3.43%-1.71%	15
	Storey 3 to 4	80x80	0.59%					
	Storey 5 to 6	75x75	0.67%					
	Storey 7 to 8	70x70	0.77%					
	Storey 9 to 10	65x65	0.89%					
	Storey 11 to 12	60x60	1.05%					
	Storey 13 to 14	55x55	1.25%					
	Storey 15 to 16	50x50	1.51%			40x50	2.51%-1.41%	
	Storey 17 to 20	45x45	1.86%			35x45	2.39%-1.60%	

N/A: Not applicable

5.3.5 NONLINEAR ANALYSIS

In order to investigate the effects of an intermediate level of vertical irregularity and its corresponding impact on the behaviour factor, a nonlinear analysis was performed. This allowed for a detailed evaluation of how variations in lateral stiffness along the height affect the seismic response of the designed structures.

The modelling approach employed in this study for simulating RC elements utilized inelastic force-based frame elements subdivided into seven integration sections, with each section further discretized into 150 fibbers to capture the nonlinear material behaviour accurately.

The seismic response of the assessed structures was analysed using the nonlinear analysis performed in SeismoStruct v2025 [15] through the adaptative pushover method, involving the estimation of the capacity curve of each structure and their strength, stiffness and second-order effects globally and locally. Pushover analysis is a recognized method for assessing the structural response of buildings to seismic loads. Numerous studies have also employed this approach to investigate the behaviour of regular and irregular buildings [16–19].

The concrete constitutive relationship proposed by Mander et al. [20], coupled with the overall stress-strains behaviour confined concrete described by Popovics [21], were considered in models' configurations. Additionally, for the C30/37 unconfined concrete grade, the mean compressive strength (f_{cm}) is equal to 38 MPa.

Analogously, Menegotto et al. [22] considered the steel constitutive relationship coupled with the hardening rules defined by Taucer et al. [23]. Furthermore, the B500 steel grade means yield characteristic strength (f_{yk}) equals to 500 MPa. It is essential to highlight that, in the nonlinear analysis, it can be assumed that the average value of the steel yield stress is equal to the characteristic.

5.4 BEHAVIOUR FACTOR ASSESSMENT

All behaviour factors considered in the design of the structures were considered as defined in EN 1998-1:2004 [1]. However, prEN 1998-1-2:2024 [3] defines behaviour factor as the product of the partial components q_R, q_S and q_D:

- q_R consider overstrength resulting from the redistribution of seismic action effects in redundant structural systems;
- q_S accounts for overstrength arising from all other sources;
- q_D acknowledges the deformation and energy dissipation capacity.

Following the approach of Maranhão et al. [5], the estimation of the behaviour factor and its components was carried out using the mathematical model adapted by Plumier [24], as incorporated in prEN 1998-1-2:2024 [3]. In line with Plummier's proposal, the components q_R, q_S and q_D, were evaluated through the analysis of the building's capacity curve at key performance points: V_e, d_e, V_1, d_1, V_u, d_y, d_r.

Those performance points are defined as follows:

- V_e: the horizontal shear force obtained from the elastic analysis, considering a design spectrum reduced by the behaviour factor;
- d_e: the displacement correspondent to the V_e measured at the buildings´ rooftop;
- V_1: the horizontal base shear force when the first plastic hinge is observed;
- d_1: the displacement correspondent to the V_1;
- V_u: the horizontal shear force at the global plastic mechanism formation;
- d_r: the displacement correspondent to the V_u
- d_y: the displacement at yield.

The q_R, q_S and q_D were considered as defined by Maranhão et al. [5] and depicted, respectively, in expressions (1), (2) and (3).

$$q_R = \frac{V_u}{V_1} \tag{1}$$

$$q_s = \frac{V_1}{V_e} \tag{2}$$

$$q_D = \frac{d_r}{d_y} \tag{3}$$

5.5 Results and discussion

5.5.1 Seismic response assessment

Figure 5-3 presents the 1st, 2nd and 3rd vibration modes and their respective frequencies and periods of each type of structure. The 10-storey MRF structures and 10-15-20-storey wall structures behave similarly in natural periods. However, the 5-storey MRF and wall structures exhibit higher fundamental frequencies than the other configurations, notably the 5-storey wall structures.

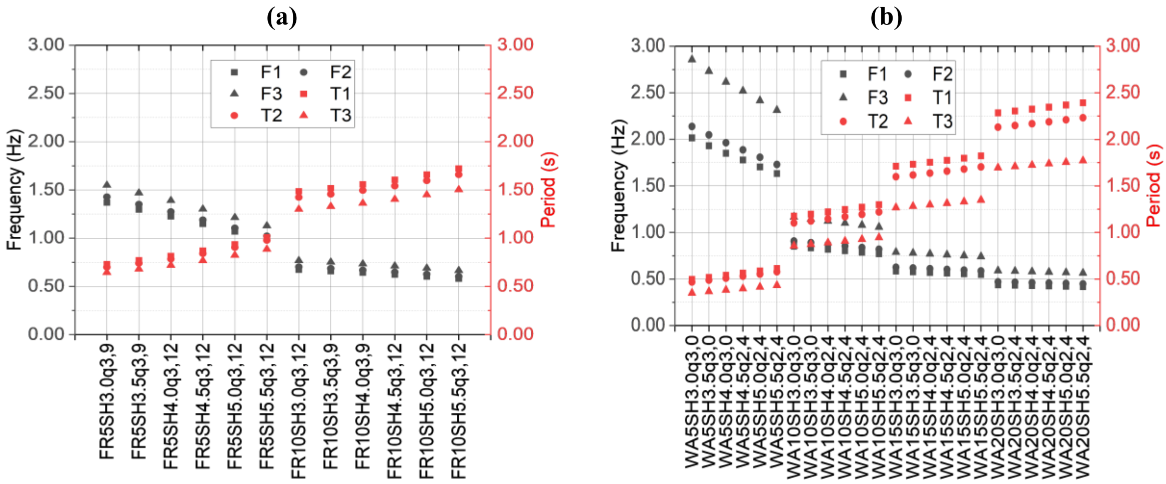


Figure 5-3: Natural frequencies: (a) MRF structures; (b) RC wall structures

Figure 5-4 presents the capacity curves of all studied structures with highlighted damage limitation, significant damage and near collapse limit states. Figure 5-5 shows the critical points of each capacity curve compared to the correspondent base shear (Figure 5-5a for MRF structures and Figure 5-5c for wall structures) and displacement (Figure 5-5b for MRF structures and Figure 5-5d for wall structures).

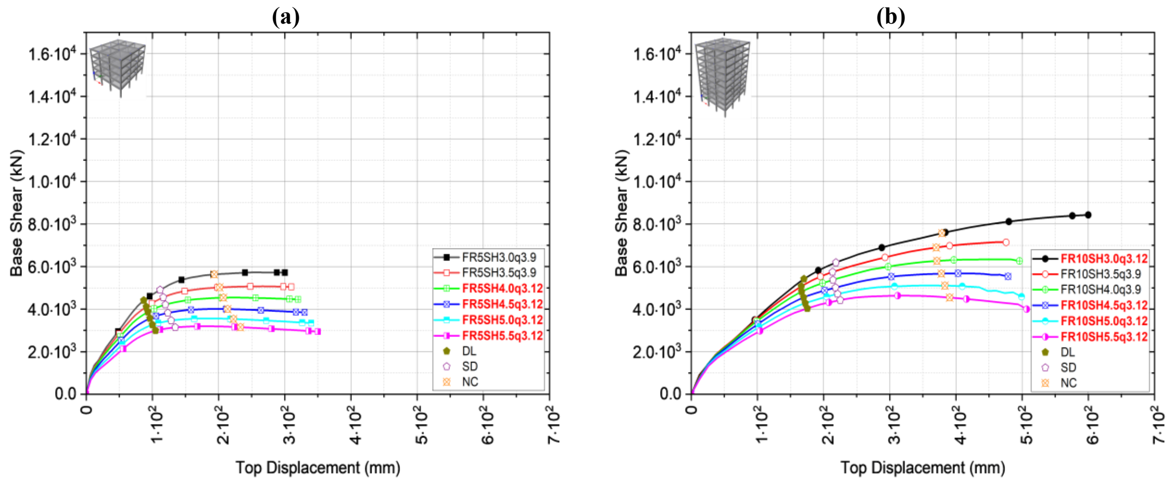


Figure 5-4: Capacity curves: (a) 5-storey MRF structures; (b) 10-storey MRF structures; (c) 5-storey wall structures; (d) 10-storey wall structures; (e) 15-storey wall structures and (f) 20-storey wall structures

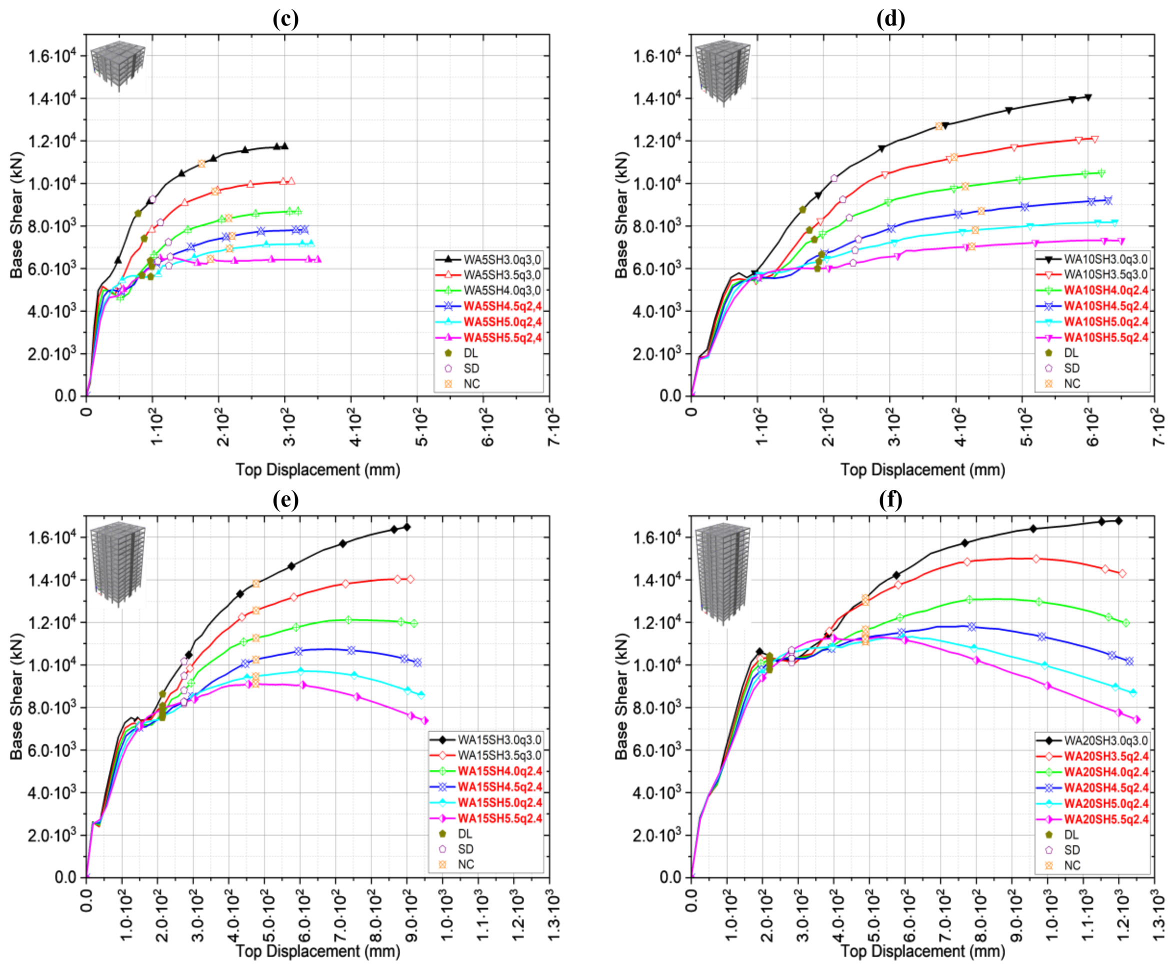


Figure 5-4: Cont.

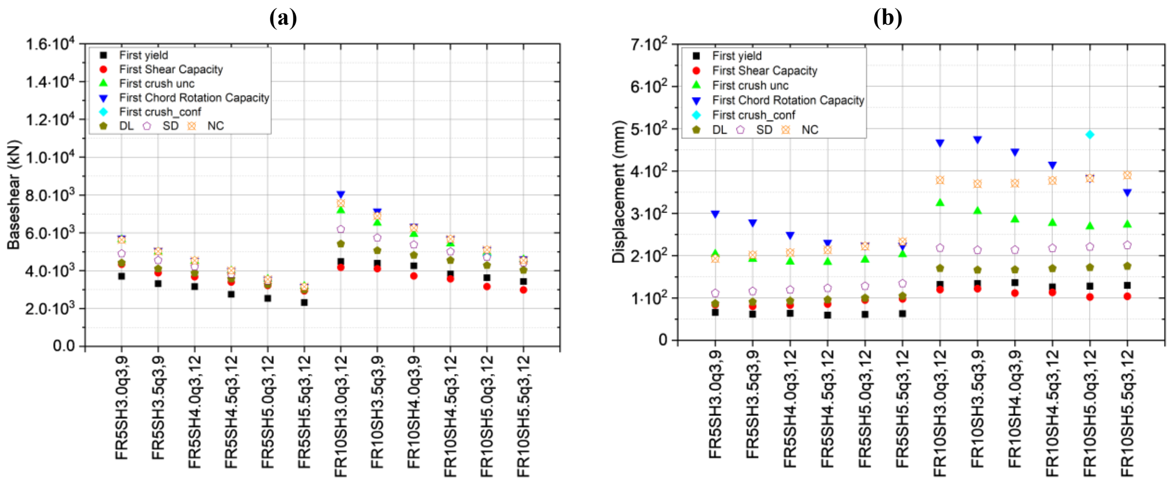


Figure 5-5: Critical points of capacity curves: (a) Base shear - MRF structures; (b) Displacement - MRF structures; (c) Base shear - Wall structures; (d) Displacement - Wall structures

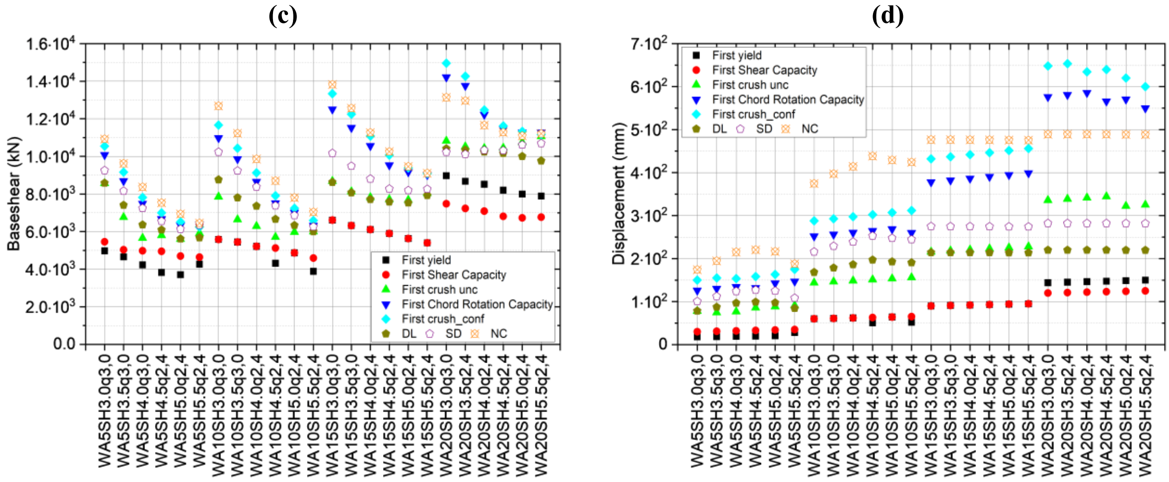


Figure 5-5: Cont.

From Figures 5-4 and 5-5, it can be concluded that:

1. For frame structures, comparing regular structures in each height, the maximum base shear shifts between 11.4%-17.8% for regular structures and between 20.7%-44.2% for irregular structures. For wall structures, the variation is, for regular structures, between 13.9%-25.8% and 10.5%-47.9% for the irregular ones. Similar behaviour occurs for the damage limitation, significant limit and near collapse base shear limits;
2. Regarding the displacement, the first yield and first shear capacity limits occur closely in all assessed structures. However, looking at the base shear parameter, this behaviour does not occur in 5-storey wall structures with 4.0 m, 4.5 m and 5.0 m of height on the ground floor and 10-storey wall structures with 4.5m and 5.5m on the first floor;
3. Regarding Figures 5-5a and 5-5c, it is clear the decreasing tendency of the occurrence of first crush uncover, first chord rotation capacity, first crush confinement and also the damage limitation, significant damage and near collapse limits in comparison to the increase of height of the ground floor, which means the increase of the irregularity level;
4. The near collapse limit occurs for FR5SH5.5q3,12, FR10SH5.5q3,12, and 5, 10 and 15-storey wall structures after the first chord rotation capacity occurrence. For the other models, this limit occurs before any structural element reaches its chord rotation capacity;
5. There is no observed crush confinement in frame structures, except FR10SH5.0q3,12, which presents this phenomenon in one beam and one column of the last floor. All wall structures, on the other hand, present at least one structural element, namely beams or/and shear walls, with crush confinement;
6. Comparing the discussion performed above with the lateral stiffness of each model demonstrated in Figure 5-2, there are different levels of irregularity regarding maximum base shear. A few models with maximum lateral stiffness

shifts slightly higher than 30% demonstrate structural behaviour in all assessed parameters similar to regular models.

Figures 5-6 and 5-7 present, respectively, the inter-storey drifts and global drifts of all studied structures in significant level limit. The reference point of displacement was the centre of mass in each storey.

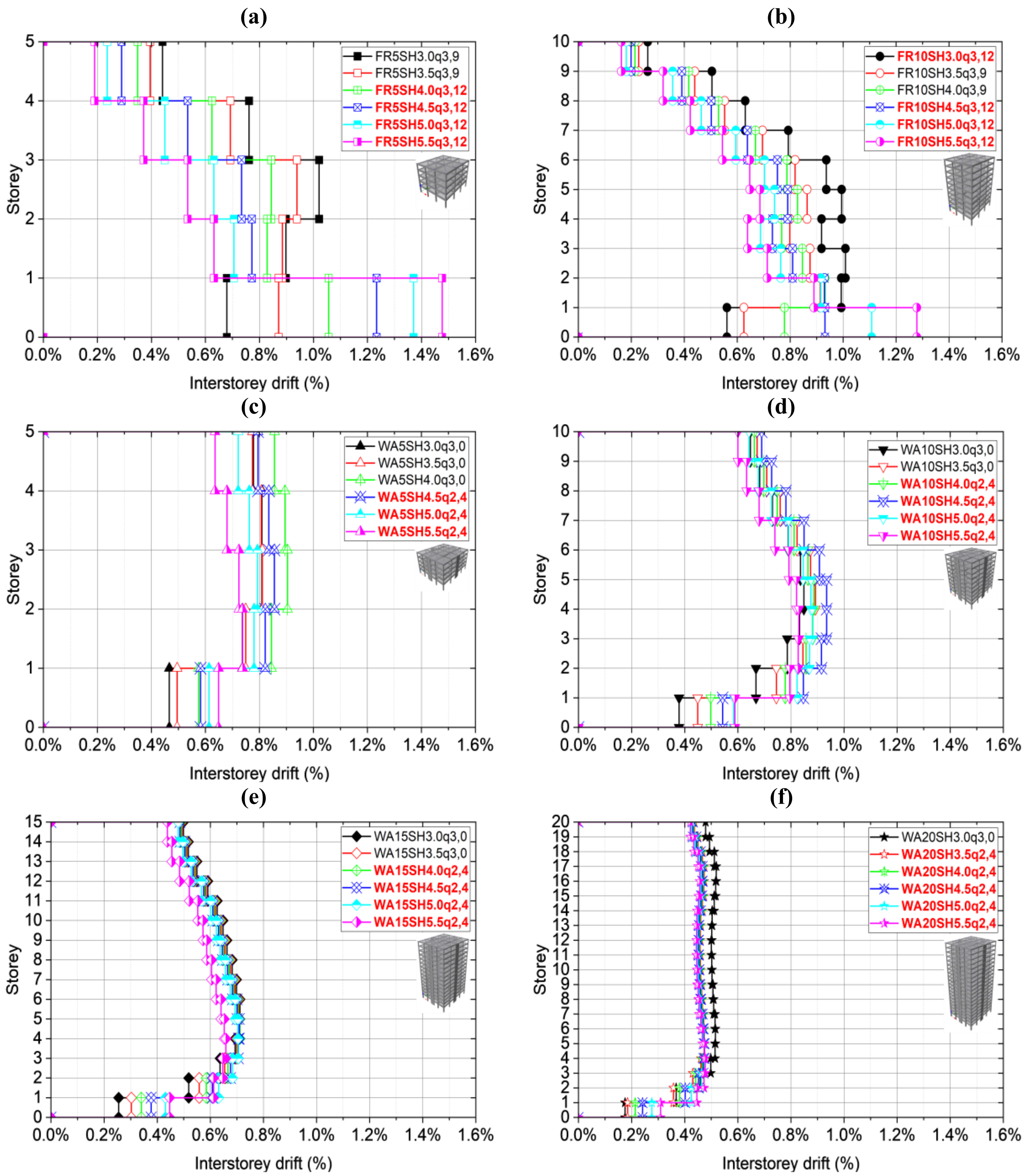


Figure 5-6: Inter-storey drifts: (a) 5-storey MRF structures; (b) 10-storey MRF structures; (c) 5-storey wall structures; (d) 10-storey wall structures; (e) 15-storey wall structures and (f) 20-storey wall structures

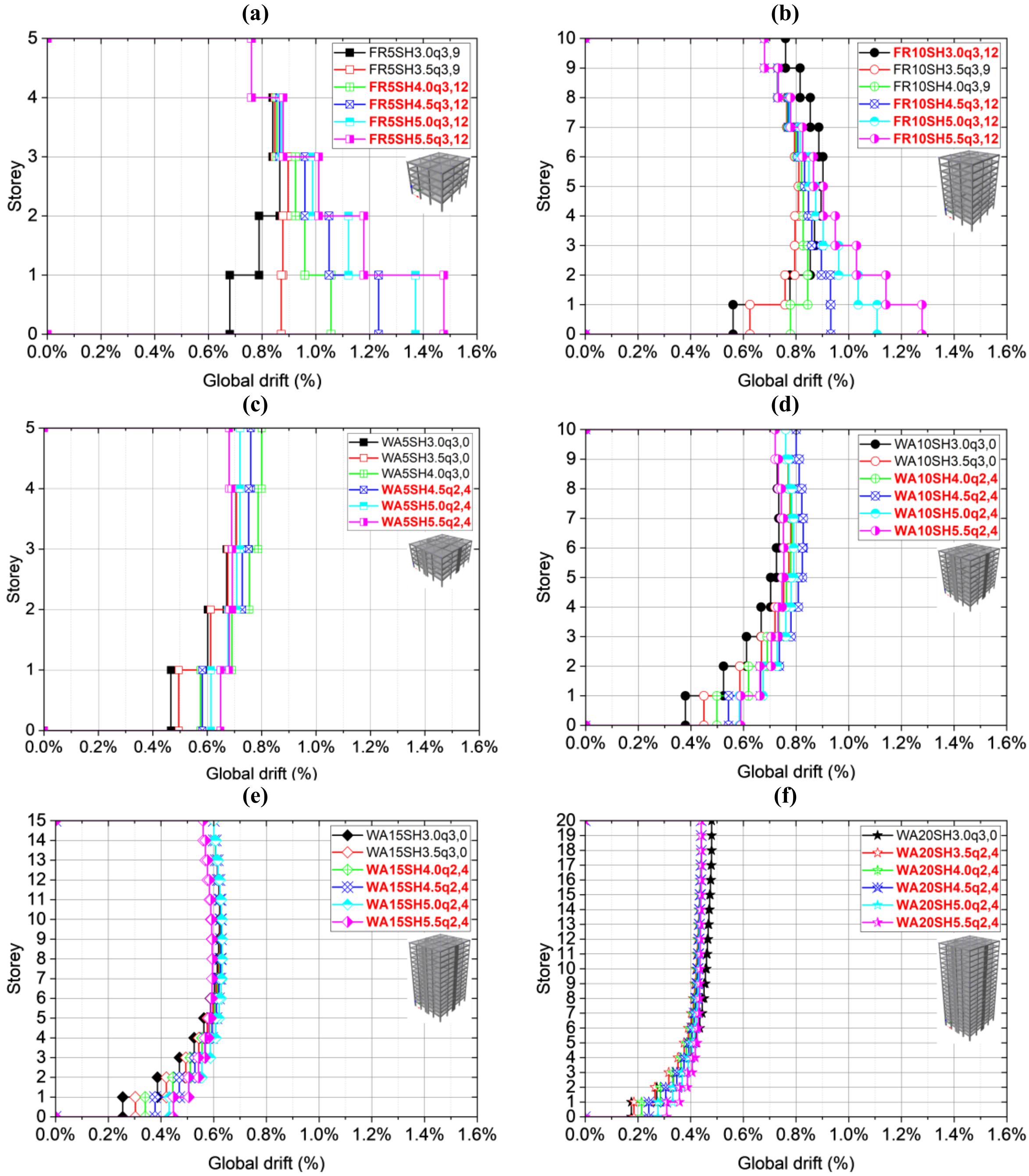


Figure 5-7: Global drifts: (a) 5-storey MRF structures; (b) 10-storey MRF structures; (c) 5-storey wall structures; (d) 10-storey wall structures; (e) 15-storey wall structures and (f) 20-storey wall structures

From those, it can draw several conclusions:

1. Considering the variation of ground floor height, the maximum inter-storey drifts observed for 5 and 10-storey MRF structures were 1.48% and 1.28%, respectively, in irregular structures. For wall structures 5-10-15-20 storeys, the following values were seen: 0.90%, 0.93%, 0.71% and 0.52%. The maximum inter-storey drift in wall structures does not shift significantly;

2. When compared to regular and irregular structures in each model type, it can be drawn that, for MRF structures, the 5-10 storeys shift of maximum inter-storey drift is 44.7% and 26.7%. For wall structures, the values are 8.8%, 10.1%, 3.9% and 8.8%, respectively. As expected, the wall structure has less variation in that behaviour in comparison to frame structures;

3. Analogously to the discussion in (1), the maximum global drift observed for MRF structures were 1.48% and 1.28%, both in irregular structures. For wall structures 5-10-15-20 storeys, the following values were seen: 0.80%, 0.83%, 0.63% and 0.48%. The maximum global drift in wall structures does not shift significantly, similar to the inter-storey drift behaviour discussed in (1). Furthermore, those maximum values were observed in irregular structures, except for 5-20 storey wall structures;

4. Observing regular and irregular structures, as done in (2), it can be drawn that, for MRF structures, the 5-10 storeys shift of maximum global drift is, respectively, 70.5% and 41.7%. The values for wall structures are 5.6%, 12.7%, 3.0% and 8.3%, respectively.

Figures 5-8 and 5-9 show, in significant damage and near collapse levels, chord rotation performance of columns, shear walls and beams of each structure, shear capacity of columns and shear walls and the percentage of collapsed beams due to shear capacity.

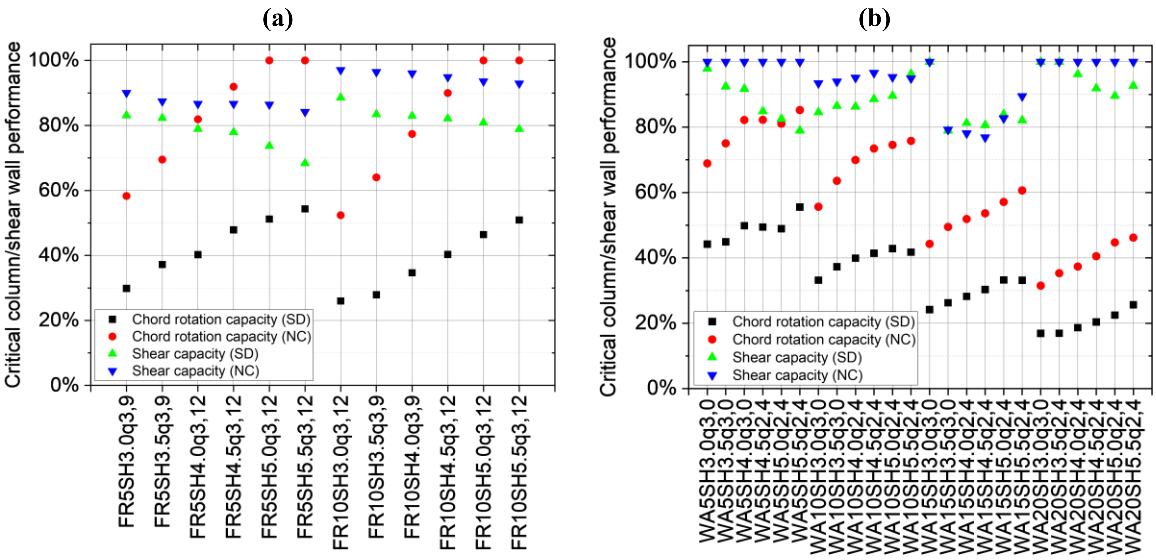


Figure 5-8: Columns/Shear walls chord rotation and shear performance (Significant damage and near collapse levels): (a) MRF structures and (b) wall structures

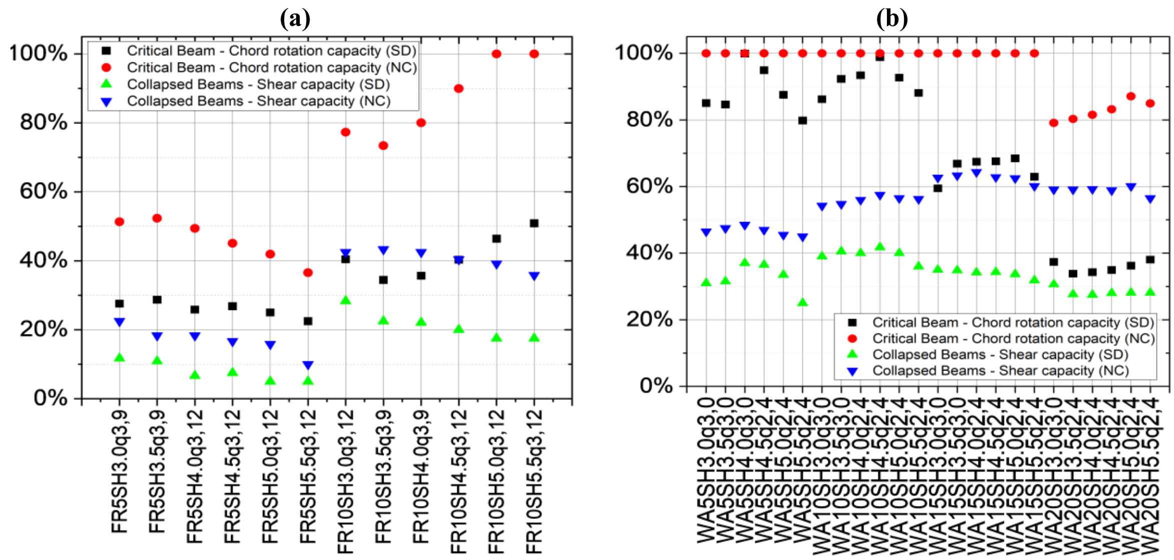


Figure 5-9: Beams chord rotation and shear performance (Significant damage and near collapse levels): (a) MRF structures and (b) wall structures

From Figures 5-8 and 5-9, several considerations can be made as follows:

1. As preconized in the design, all structures align with capacity design principles by allowing beams to yield before the columns and shear walls under seismic conditions. Nonlinear analysis demonstrates that beams are the first to develop plastic hinges before columns and the first to reach the rupture due to the plastic hinges;

2. At a significant damage level, all structures have no collapsed columns and shear walls due to chord rotation. However, there are failures in the second-floor shear walls due to shear force in SW20SH3.0q3,0 and SW20SH3.5q2,4 structures. For beams, considerable quantity of beams collapses due to shear forces;

3. At near collapse level, most of the beams collapse due to shear forces; lower, however significant percentage of the beams also presents failure because of chord rotation. For vertical elements, it can be observed that there are collapsed columns by chord rotation capacity in FR5SH(5.0-5.5)q3,12, FR10SH(5.0-5.5)q3,12 structures. There are 3, 5, 4 and 10 collapsed columns, all in first storey. Regarding shear capacity of columns and shear walls, there is collapse of the shear wall by shear capacity of third floor in SW5SH(3.0-3.5)q3,0 and SW5SH4.0q2,4 models; in second storey in SW5SH(4.5-5.5)q2,4 and SW20SH(4.0-5.5)q2,4 structures; in second and third floor in SW20SH3.0q3,0 and SW20SH3.5q2,4 models.

5.5.2 Behaviour factor analysis and proposal

Table 5-3 shows the nonlinear analysis parameters for computing the behaviour factor values and its components. Figure 5-10 shows, for each model (5-10 storey MRF building and 5-10-15-20 storey wall building), the results of q_R, q_S and q_D calculated with the data presented in Table 5-3. Figure 5-11 presents the comparison of the behaviour factor considered in the design of the structures, based on the EN 1998-1:2004 [1], the behaviour factor prescribed in the prEN 1998-1-2:2024 [3], and the lateral stiffness shift in X-axis and Y-axis of each designed structure. Figure 12 demonstrates the percentual shift of those behaviour factors compared to the regular structure of each building typology.

Table 5-3: Nonlinear analysis parameters for behaviour factor components

Structure	V_e (kN)	d_e (m)	V_1 (kN)	d_1 (m)	d_y (m)	V_u (kN)	d_r (m)
FR5SH3.0q3,9	2450.0	0.036	3705.1	0.066	0.087	5727.7	0.193
FR5SH3.5q3,9	2359.5	0.037	3315.5	0.062	0.091	5072.4	0.267
FR5SH4.0q3,12	2032.9	0.032	3157.4	0.064	0.093	4543.5	0.224
FR5SH4.5q3,12	1877.2	0.033	2750.8	0.059	0.096	4008.0	0.198
FR5SH5.0q3,12	1520.9	0.027	2539.7	0.061	0.100	3561.9	0.184
FR5SH5.5q3,12	1387.9	0.028	2321.2	0.062	0.105	3195.9	0.175
FR10SH3.0q3,12	3137.0	0.084	4484.8	0.132	0.170	8428.7	0.600
FR10SH3.5q3,9	3477.5	0.098	4392.6	0.134	0.166	7153.1	0.464
FR10SH4.0q3,9	3434.8	0.099	4257.6	0.136	0.167	6338.6	0.446
FR10SH4.5q3,12	3028.4	0.088	3830.1	0.126	0.170	5681.3	0.403
FR10SH5.0q3,12	2637.3	0.077	3627.0	0.128	0.172	5111.4	0.358
FR10SH5.5q3,12	2517.8	0.078	3430.2	0.130	0.176	4632.8	0.325
SW5SH3.0q3,0	5659.0	0.036	4976.1	0.018	0.078	11723.2	0.300
SW5SH3.5q3,0	4863.5	0.037	4654.3	0.019	0.088	10085.5	0.310
SW5SH4.0q3,0	4980.6	0.032	4227.1	0.019	0.097	8693.4	0.320
SW5SH4.5q2,4	5005.5	0.040	3825.1	0.020	0.099	7821.9	0.330
SW5SH5.0q2,4	4698.3	0.034	3705.1	0.020	0.097	7158.4	0.340
SW5SH5.5q2,4	4641.9	0.035	4267.1	0.028	0.084	6514.8	0.126
SW10SH3.0q3,0	6737.3	0.120	5581.3	0.060	0.168	14073.2	0.600
SW10SH3.5q3,0	5554.1	0.110	5442.4	0.061	0.179	12110.2	0.610
SW10SH4.0q2,4	5497.1	0.099	5213.6	0.062	0.186	10498.1	0.620
SW10SH4.5q2,4	5462.3	0.088	4316.6	0.050	0.197	9211.5	0.630
SW10SH5.0q2,4	5584.5	0.090	4867.1	0.064	0.193	8174.4	0.640
SW10SH5.5q2,4	5534.1	0.091	3883.7	0.052	0.191	7326.3	0.598
SW15SH3.0q3,0	8753.1	0.162	8243.2	0.108	0.214	16519.5	0.900
SW15SH3.5q3,0	7418.4	0.164	6319.9	0.091	0.214	14032.2	0.892
SW15SH4.0q2,4	7187.8	0.166	6110.5	0.092	0.214	12117.0	0.754
SW15SH4.5q2,4	7080.1	0.149	5897.9	0.093	0.214	10737.2	0.670
SW15SH5.0q2,4	7156.7	0.150	5631.8	0.094	0.214	9704.6	0.620
SW15SH5.5q2,4	7122.7	0.152	5402.6	0.095	0.214	9108.7	0.475
SW20SH3.0q3,0	10461.3	0.216	8972.4	0.144	0.220	16776.0	1.200
SW20SH3.5q2,4	10346.8	0.218	8678.8	0.145	0.220	15006.2	0.895
SW20SH4.0q2,4	10071.9	0.195	8521.3	0.146	0.220	13095.9	0.854
SW20SH4.5q2,4	9771.6	0.197	8201.0	0.189	0.220	11828.9	0.763
SW20SH5.0q2,4	9097.9	0.174	7999.2	0.149	0.220	11329.5	0.620
SW20SH5.5q2,4	8828.9	0.175	7892.8	0.150	0.220	11306.3	0.525

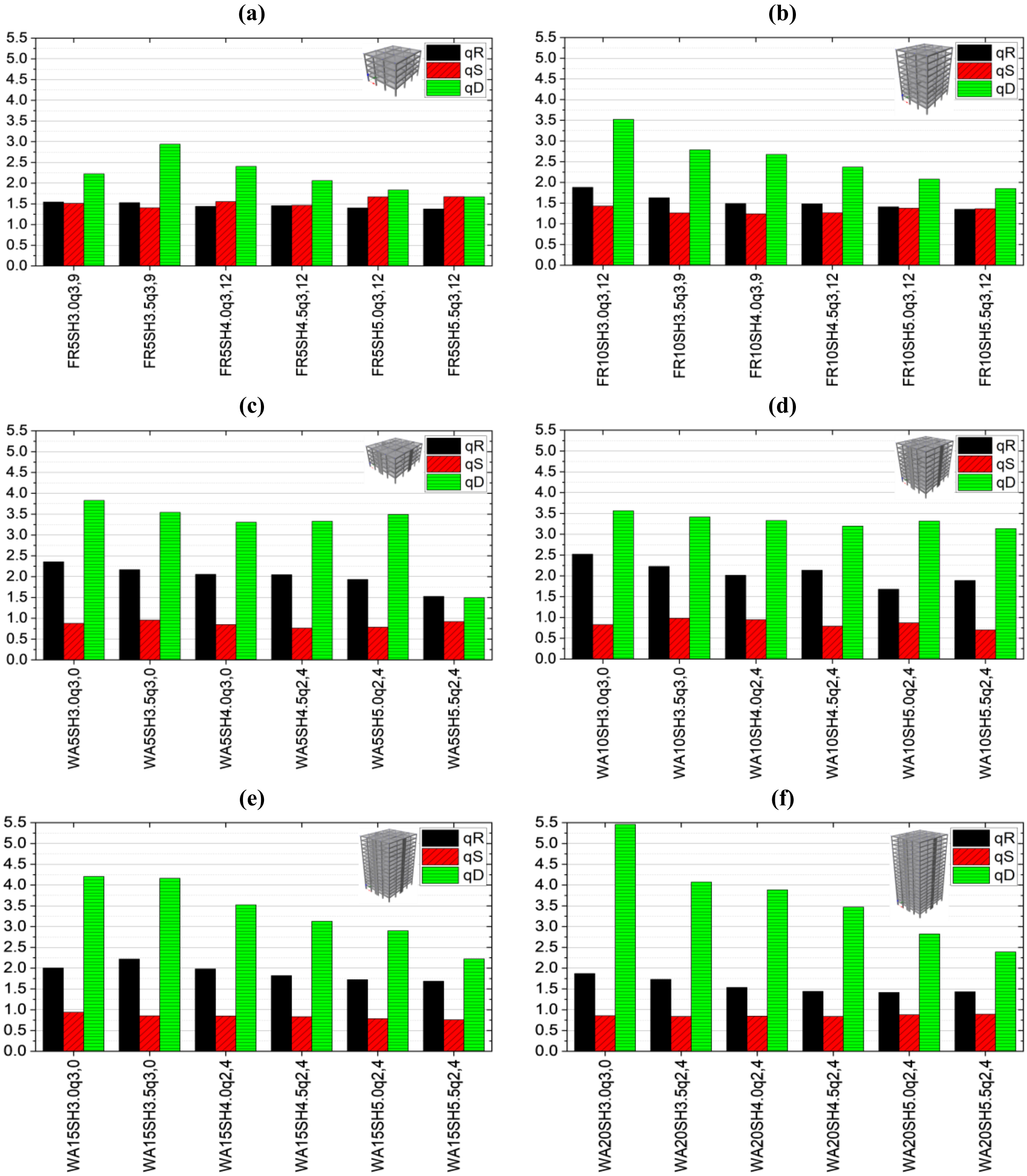


Figure 5-10: Behaviour factor components: (a) 5-storey MRF structures; (b) 10-storey MRF structures; (c) 5-storey wall structures; (d) 10-storey wall structures; (e) 15-storey wall structures and (f) 20-storey wall structures

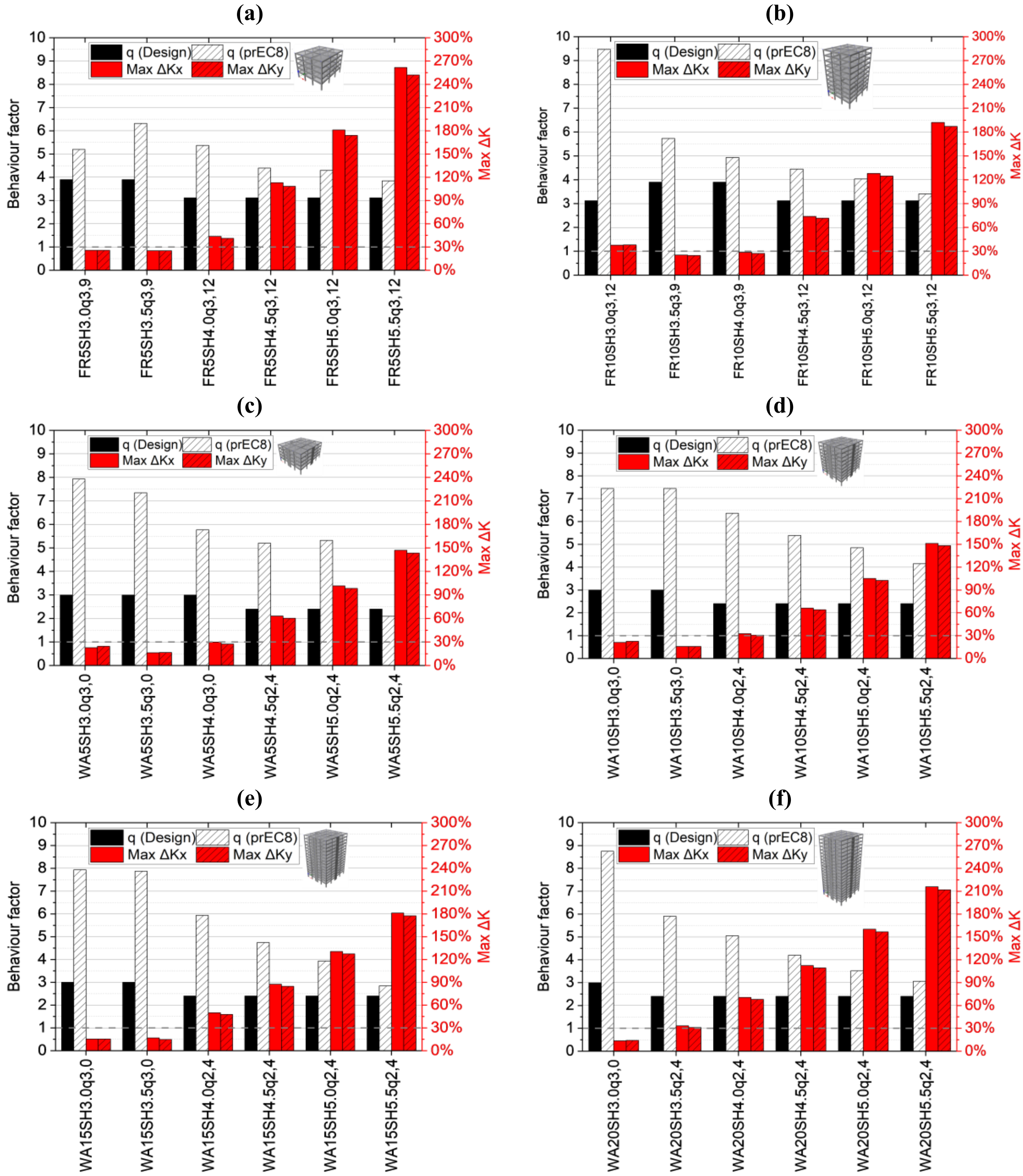


Figure 5-11: Behaviour factor (EC8 and prEC8) comparison with lateral stiffness: (a) 5-storey MRF structures; (b) 10-storey MRF structures; (c) 5-storey wall structures; (d) 10-storey wall structures; (e) 15-storey wall structures and (f) 20-storey wall structures

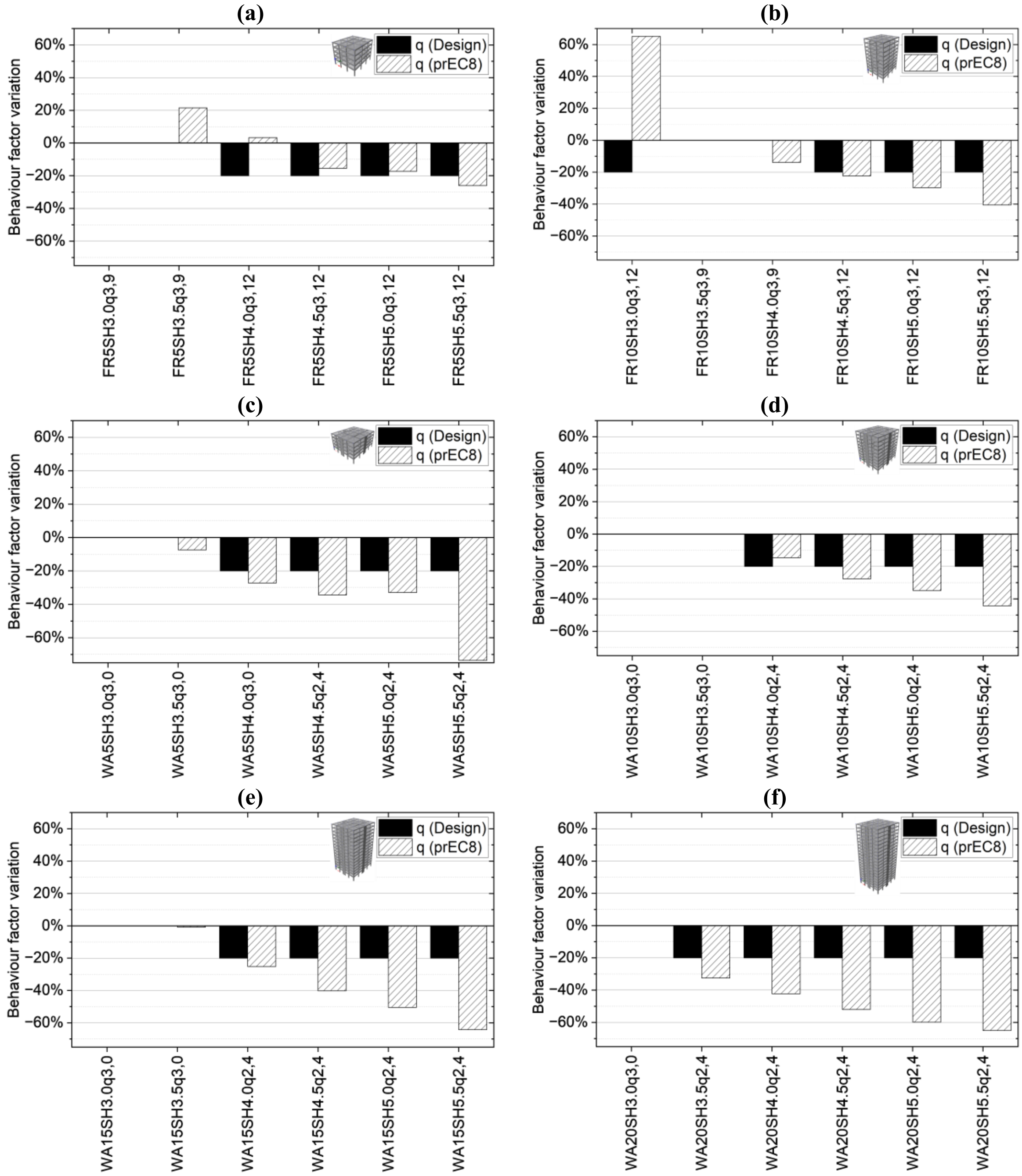


Figure 5-12: Behaviour factor (design and prEC8) comparison with lateral stiffness: (a) 5-storey MRF structures; (b) 10-storey MRF structures; (c) 5-storey wall structures; (d) 10-storey wall structures; (e) 15-storey wall structures and (f) 20-storey wall structures

From Table 5-3, Figures 5-10, 5-11 and 5-12, it can be inferred that::

1. The behaviour factor component accounting for the overstrength, q_R and q_S, exhibits specific ranges, respectively, 1.35-2.52 and 0.70-1.67. Despite the increase in irregularity, those parameters do not have a considerable shift when compared to each structural typology;

2. For deformation and energy dissipation capacity (q_D), on the other hand, presents a broader shift, especially in the SW20SH3.0q2,4. That structure is slightly irregular; however, because of that classification, the structure needed to be more reinforced, causing probably this effect of more deformation and energy dissipation;
3. The behaviour factor values determined according to the second generation of Eurocode 8 are substantially higher than what was considered in the design. A similar tendency can be observed in Maranhão et al. [5], Louzai et al. [25] and Ferraioli [26]. That fact and the aforementioned studies demonstrate that the prEN 1998-1-2:2024 is conservative;
4. Maranhão et al. [5] demonstrated that the structural height increase reduces the behaviour factor. However, the increment of maximum lateral stiffness shift between storeys, due to the imposition of the highest vertical irregularity, reduces significantly the decrease of behaviour factor;
5. Observing the structural typology separately (5 and 10-storey MRF and 5,10,15, and 20-storey wall structure), when irregularity in elevation increases, it can be observed the reduction tendency of the behaviour factor in all typologies;
6. Comparing the behaviour factor of irregular structures of first and second generations of Eurocode 8, the MRF structures present similarity in both approaches, except structures FR10SH3.0q3,12 and FR10SH5.5q3,12. For wall structures, on the other hand, the shift of behaviour factor from the prEN 1998-1-2:2024 [3] was higher than the observed in the EN 1998-1:2004 [1]. The FR10SH3.0q3,12 equivalent behaviour factor can be explained by the fact of that structure been slightly irregular. It was reinforced, considering this irregularity, however, it behaves similarly to regular structures. Considering that aspect, this behaviour was expected.

Figure 5-13 demonstrates the correlation between the prEN 1998-1-2:2024 behaviour factor and the X-axis's maximum shift of inter-storey lateral stiffness in frame and wall structures. It is essential to highlight that this comparison has no significant change, regardless of the orthogonal axis of assessed lateral stiffness variation.

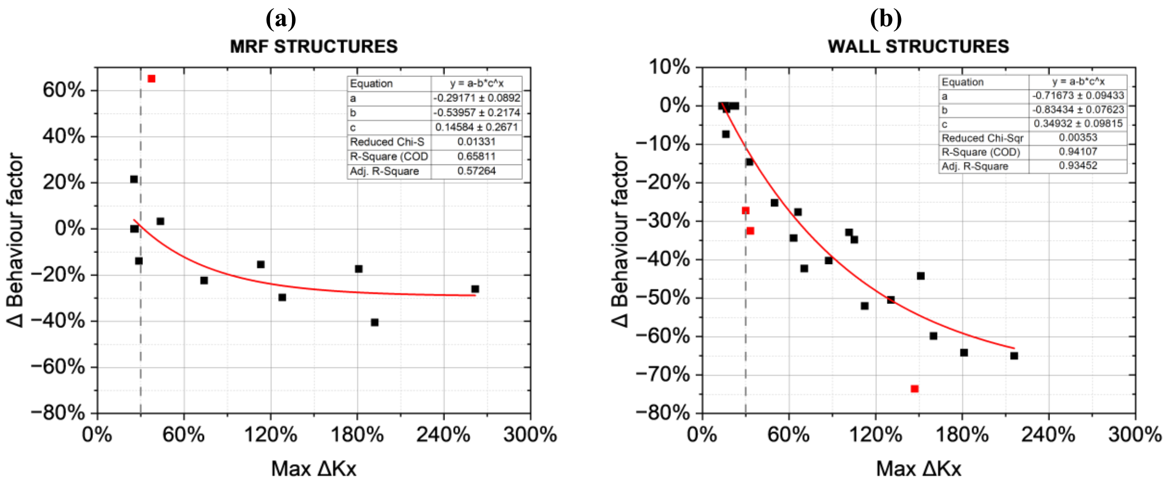


Figure 5-13: prEC8 behaviour factor comparison with maximum lateral stiffness shift: (a) MRF structures; (b) wall structures; (c) MRF and wall structures

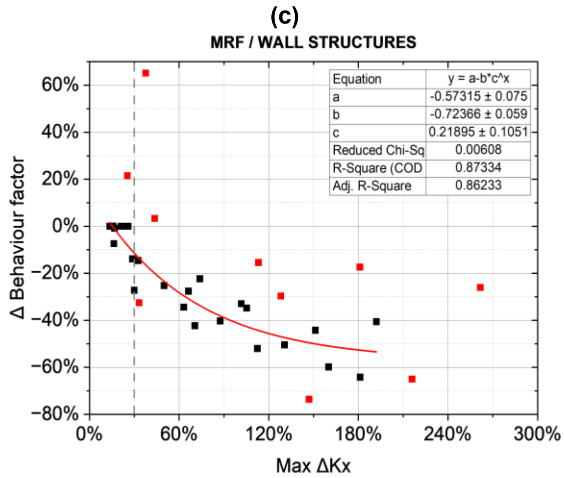


Figure 5-13: Cont.

From Figure 5-13, it can be concluded that:

1. Figures 5-13a and 5-13b demonstrate a clear tendency to decrease the behaviour factor when the maximum inter-storey lateral stiffness shift increases;
2. From Figure 5-13c, it is clear that different levels of imposed vertical irregularity generate distinct reductions in behaviour factor;
3. Considering that this work concentrates on irregularity in elevation imposed by shifts of lateral stiffnesses, therefore the limitation of this research, it can be proposed to Eurocode 8, as happens in other seismic codes, levels of irregularity: irregular and strongly irregular structures;
4. Observing the results in Figure 5-13c, considering also the limit of inter-storey lateral stiffness shift of 60% that occurs in buildings with 4.0 m of ground floor height, a reasonable elevation limit:
    a. The lateral stiffness shift to an irregular structure should be between 30% and 60%. For values above 60%, a structure shall be considered strongly irregular;
    b. The irregular structures should keep reduction factors at 20%, and highly irregular structures shall have 30% reduction factors, as observed in Mexican [27,28] and North American seismic codes [29].

5.6 SUMMARY AND CONCLUSIONS

This study assessed MRF and wall regular and irregular in elevation structures with inter-storey lateral stiffness shift as the control variable, conducting a comprehensive analysis of their effects on seismic responses. Additionally, it was analyzed the behaviour factor calculation methodology of the first and second generations of Eurocode 8 [1,3] by the comparison of the defined (q) in the structural design as preconized by the current EC8 [1] and the same variable verified in nonlinear analysis, considering the prEN 1998-1-2:2024 [3].

Therefore, the following main conclusions can be drawn:

1. As expected, the increment of vertical irregularity decreases the maximum base shear and the base shear of damage limitation, significant damage and near collapse levels;

2. The maximum inter-storey and global drift in wall structures does not shift significantly. However, the opposite behaviour is observed in frame structures. Comparing regular and irregular elevation structures, the irregular frame ones demonstrate the worst behaviour regarding maximum inter-storey drift. On the other hand, for wall structures, those differences are not discernible;
3. Models classified as irregular by the criterium of shift slightly above 30% of the maximum lateral stiffness shift present structural response parameters close to regular structures;
4. There is a trend of reduction, in different levels, in behaviour factor as the maximum inter-storey lateral stiffness shift increases, hence the elevation irregularity increment;
5. This study proposes a classification of regular, irregular and strongly irregular structures. It is suggested that the lateral stiffness shift to an irregular structure should be between 30% and 60%. For strongly irregular structures, this value shall be above 60%. Additionally, a reduction of 20% of (q) for irregular structures and 30% for strongly irregular ones is recommended, as can be seen in Mexican [27,28] and North American seismic codes [29].

It is essential to highlight that the conclusions addressed here were delimitated to RC structures with concrete grade C30/37, steel grade B500, and specific plan and height configurations. Hence, validation across different structures persists.

5.7 REFERENCES

[1] EN 1998-1. *"Eurocode 8 - Design of structures for earthquake resistance - Part 1–1: General rules, seismic actions and rules for buildings."* European Comittee for Standardization: Brussels, Belgium, 2004.

[2] EN 1992-1-1. *"Eurocode 2 - Design of concrete structures - Part 1–1: General rules and rules for buildings."* European Comittee for Standardization: Brussels, Belgium, Brussels, Belgium, 2004.

[3] prEN 1998-1-2: *"Eurocode 8: Design of Structures for Earthquake Resistance - Part 1-2: Rules for New Buildings, Draft Post-ENQ 48th Meeting CEN/TC250/SC8."* European Comittee for Standardization: Brussels, Belgium, 2024.

[4] Maranhão, H., H. Varum, J. Melo, and A. A. Correia. 2024. "Comparative Analysis of the Impact of Design and Detailing Provisions for RC Moment Resisting Frames under the First- and Second-Generation of Eurocode 8." *Engineering Structures* 306: 117809. https://doi.org/10.1016/j.engstruct.2024.117809.

[5] Maranhão, H., H. Varum, and J. Melo. 2025. "Exploring Behaviour Factors and Chord Rotation in RC MRF Structures: Nonlinear Analysis Based on Second Generation of Eurocode 8." *Engineering Structures* 323: 119252. https://doi.org/10.1016/j.engstruct.2024.119252.

[6] Fardis, M. N., and T. B. Panagiotakos. 2024. "Comparative Application to RC Buildings of the Two Generations of Eurocodes and Proposals for Seismic Design." *Bulletin of Earthquake Engineering* 22 (9): 4477–4513. https://doi.org/10.1007/s10518-024-01931-y.

[7] Computer and Structures Inc. 2020. “ETABS”. Berkeley, USA.

[8] Santos D, Melo J, Varum H. “Comparative Analysis of the Impact of Vertical Irregularities on Reinforced Concrete Moment-Resisting Frame Structures According to Eurocode 8.” *Buildings* 14 (9). https://doi.org/10.3390/buildings14092982.

[9] EN 206-1. *“Concrete - Part 1: Specification, performance, production and conformity.”* European Comittee for Standardization: Brussels, Belgium, 2007.

[10] EN 1990. “*Eurocode - Basis of structural design.*” European Comittee for Standardization: Brussels, Belgium, 2002.

[11] EN 1991-1-1. “*Eurocode 1: Actions on structures - Part 1-1: General actions - Densities, self-weight, imposed loads for buildings.*” European Comittee for Standardization: Brussels, Belgium, 2002.

[12] Benaied, B., M. Hemsas, A. Benanane, and M. Hentri. 2023. “Seismic Analysis of RC Building Frames with Vertical Mass and Stiffness Irregularities Using Adaptive Pushover Analysis.” *Revista de La Construcción* 22 (3): 597–612. https://doi.org/10.7764/rdlc.22.3.597.

[13] Santos, D., J. Melo, and H. Varum. 2024. “Code Requirements for the Seismic Design of Irregular Elevation RC Structures.” *Buildings* 14 (5): 1351. https://doi.org/10.3390/buildings14051351.

[14] Bento, R., and J. Azevedo. 2000. “Behaviour Coefficient Assessment for Soft Storey Structures.” In *Proceedings of the Twelfth World Conference on Earthquake Engineering*, 1–8. Auckland, New Zealand.

[15] Seismosoft. 2021. “SeismoStruct 2021 – A Computer Program for Static and Dynamic Nonlinear Analysis of Framed Structures.”

[16] Chintanapakdee, C., and A. K. Chopra. 2004. “Seismic Response of Vertically Irregular Frames: Response History and Modal Pushover Analyses.” *Journal of Structural Engineering*, 1177–85. https://doi.org/10.1061/ASCE0733-94452004130:81177.

[17] Athanassiadou, C. J. 2008. “Seismic Performance of R/C Plane Frames Irregular in Elevation.” *Engineering Structures* 30 (5): 1250–61. https://doi.org/10.1016/j.engstruct.2007.07.015.

[18] Nezhad, M. E., and M. Poursha. 2015. “Seismic Evaluation of Vertically Irregular Building Frames with Stiffness, Strength, Combined-Stiffness-and-Strength and Mass Irregularities.” *Earthquake and Structures* 9 (2): 353–73. https://doi.org/10.12989/eas.2015.9.2.353.

[19] Dya, A. F. C., and A. W. C. Oretaa. 2015. “Seismic Vulnerability Assessment of Soft Story Irregular Buildings Using Pushover Analysis.” *Procedia Engineering* 125: 925–32. https://doi.org/10.1016/j.proeng.2015.11.103.

[20] Mander, J. B., M. J. N. Priestley, and R. Park. 1988. “Theoretical Stress-Strain Model for Confined Concrete.” *Journal of Structural Engineering* 114 (8): 1804–26.

[21] Popovics, S. 1973. “A Numerical Approach to the Complete Stress-Strain Curve of Concrete.” *Cement and Concrete Research* 3 (5): 583–99.

[22] Menegotto, M., and P. E. Pinto. 1973. “Method of Analysis for Cyclically Loaded R.C. Plane Frames Including Changes in Geometry and Non-Elastic Behaviour of Elements

under Combined Normal Force and Bending." In *Symposium on the Resistance and Ultimate Deformability of Structures Acted on by Well-Defined Repeated Loads*, 15–22. Zurich: International Association for Bridge and Structural Engineering Zurich. https://doi.org/10.5169/seals-13741.

[23] Taucer, F. F. 1991. "A Fiber Beam-Column Element for Seismic Response Analysis of Reinforced Concrete Structures." UcbEerc-9117, Berkeley.

[24] Plumier, A. 2019. "PT2-BD-59-V4 - About the Definition of Overstrength Factor Ωov, Internal Document of the CEN/TC 250/SC8 Working Group PT2-BD-59-V4."

[25] Louzai, A., and A. Abed. 2015. "Evaluation of the Seismic Behavior Factor of Reinforced Concrete Frame Structures Based on Comparative Analysis between Non-Linear Static Pushover and Incremental Dynamic Analyses." *Bulletin of Earthquake Engineering* 13 (6): 1773–93. https://doi.org/10.1007/s10518-014-9689-7.

[26] Ferraioli, M. 2021. "Behaviour Factor of Ductile Code-Designed Reinforced Concrete Frames." *Advances in Civil Engineering* 2021. https://doi.org/10.1155/2021/6666687.

[27] NTC-RSEE. "*Administración Pública de la Ciudad de México Norma Técnica Complementaria Para La Revisión de La Seguridad Estructural de Las Edificaciones*." Órgano de Difusión del Gobierno de la Ciudad de México: México City, Mexico, 2017.

[28] MOC-2015. *"Manual de Diseño de Obras Civiles: Diseño Por Sismo."* Comisión Federal de Electricidad: México City, Mexico, 2015.

[29] ASCE/SEI7-22. *"Minimum Design Loads and Associated Criteria for Buildings and Other Structures."* American Society of Civil Engineers: Reston, VA, USA, 2021.

5.8 NOMENCLATURE

a_g	Design ground acceleration
d_e	The displacement correspondent to the V_e measured at the buildings´ rooftop
d_1	The displacement correspondent to the V_1
d_r	The displacement correspondent to the V_u
d_y	The displacement at yield
f_{cm}	Mean characteristic compressive strength
f_{ym}	Mean yield strength
ΔK_x	Lateral stiffness shift between storeys in X-axis
ΔK_y	Lateral stiffness shift between storeys in Y-axis
DCM	Ductility class medium
DL, SD, NC	Limit states: Damage limitation, significant damage and near collapse, respectively
MRF	Moment-resisting frames
q	Behaviour factor
q_R , q_S	Behaviour factor component: consider overstrength of structure due to, respectively, redistribution of action effects in redundant structural systems and all other sources
q_D	Behaviour factor component: consider the deformation and energy dissipation capacity
S_α	Soil factor
V_e	The horizontal shear force obtained from the elastic analysis, considering a design spectrum reduced by the behaviour factor
V_1	The horizontal base shear force when the first plastic hinge is observed
V_u	The horizontal shear force at the global plastic mechanism formation

6 Vertical irregularity and seismic response assessment of existing RC buildings: Case studies

Santos, D.; Melo, J.; Varum, V. (submitted in press) Vertical irregularity in existing RC buildings: Seismic codes criteria evaluation and impact on seismic behaviour.

6.1 ABSTRACT

Several seismic events have provided crucial lessons on proper and improper earthquake resisting construction. Despite the design procedures having different approaches regarding irregularities assessment due to regional differences, structural engineers can learn much by studying those procedures in construction materials, techniques and structural design. This study aims to contribute to the field by performing the analysis of several existing buildings regarding vertical irregularity criteria from different seismic codes and their seismic response. The structural response was carried out through adaptative pushover analysis performed in the SeismoStruct v2025. The results were compared to evaluate the influence of irregularity in elevation classification, seismic standard considered in the design and differences in structural properties. The study outcomes revealed that despite the differences in vertical irregularity criteria, the classification of the assessed structures was similar throughout all analysed seismic codes. Additionally, it was observed the impact of seismic design consideration or not in structural response in capacity curves, inter-storey and global drifts. Finally, the results demonstrate the importance of seismic design to better structural behaviour.

6.2 INTRODUCTION

Several seismic events, such as the Kahramanmaraş earthquake (Türkiye) [1,2], Sichuan (China) [3,4], L'Aquila (Italy) [5], Lorca (Spain) [6], Emilia (Italy) [7–9], Gorkha (Nepal) [10–12], Puebla (Mexico) [13], have provided crucial lessons on proper and improper earthquake resisting construction [5,14,15]. In regions that have long been inhabited and subjected to relatively frequent firm ground shaking, seismic design procedures have evolved, resulting in moderately good performance of engineered structures.

Nonetheless the design procedures have different approaches regarding irregularities assessment due to regional differences, as demonstrated in the vertical irregularities comparison throughout several seismic codes performed by Santos et al. [16], structural engineers can learn much by studying those procedures in construction materials, techniques and structural design [17].

Considering this scenario, this study aims to contribute to the field by performing the numerical analysis of several existing buildings regarding vertical irregularities criteria from different seismic codes and their seismic response. Hence, the objectives of this study are:

1. Perform the vertical irregularity classification of 16 existing buildings, considering the EN 1998-1:2004 [18] and prEN 1998-1-2:2024 [19];
2. Compare the irregularity classification of those structures with the following codes: NBR 15421:2023 (Brazil) [20], NTC-RSEE [21] and MOC-2015 [22] (Mexico), ASCE/SEI7-22 (USA) [23] and NZS 1170.5 [24] (New Zealand);

3. Analysis of the seismic response of 4 of assessed existing buildings, considering different plan and vertical geometries, ductility classification, concrete and rebars properties and the seismic load consideration in the design. The assessment was carried out through adaptative pushover analysis performed in the SeismoStruct v2025 [25,26]. The capacity curves, performance points, inter-storey and global drift profile were discussed and compared.

The relevance of this work is contained in the analysis of several existing buildings from different places, with distinct structural characteristics. For that, it was considered the aspect of irregularity in elevation and its criteria throughout several standards and seismic response comparison from those structures with and without seismic design.

6.3 ADOPTED CONVENTIONS AND SIMPLIFICATIONS

All classifications and assessments will be carried out according to the two main orthogonal directions (X and Y), following the axes of each project. The areas considered in the study are the real areas of the pavement, considering not only the surroundings of the resistant vertical elements, but also the cantilever regions.

The height of the building corresponds to the difference in heights between the highest point of the building structure and the one on the ground floor, referred to as floor 0. The ceiling height is the difference in heights between the lower face of the slab ceiling and the upper surface of the floor, in rough.

For mass purposes, the floor area and the mass are those of the floor immediately above, that is, when analysing floor 1, the data relating to the vertical elements that will influence stiffness will be those of floor 1, but for the slab corresponding to that floor will be the floor slab of floor 2. The masses of the floors were calculated according to EN 1990:2002 [27], according to the type of use of the building, whether commercial, residential, etc.

In this study, it will be focused on irregularity in elevation. For the criteria of classification of this irregularity:

1. Lateral stiffness shift between storeys: EN 1998-1:2004 [18] does not quantify this criterion, however, in the draft version, the standard specifies that these reductions must not exceed 30% in relation to the below and above floor;
2. Mass variation: EN 1998-1:2004 [18] specifies that the mass of each storey remain constant or gradually decrease. In the other hand, prEN 1998-1-2:2024 [19] prescribes that the mass of a given floor should not varies more than 150% of a consecutive storey;
3. Ratio of actual storey resistance to the resistance required by the analysis: prEN 1998-1-2:2024 [19] defines that this parameter should not shift more than 30% between consecutive floors, with exception to the top storey;
4. Setbacks: The setbacks will be analysed considering the criteria established in EN 1998-1 [18], according to the type of setback presented, for each orthogonal, X and Y direction. According to EN 1998-1:2004 [18], each setback type has a tolerance limit to be considered;

5. Lateral actions resistant system continuity: It was also observed if the systems resistant to lateral actions are continuous from the foundation to the top, without any interruption, as defined by EN 1998-1:2004 [18] and prEN 1998-1-2:2024 [19].

6.4 IRREGULARITY IN ELEVATION ASSESSMENT IN EXISTING BUILDINGS

6.4.1 BUILDINGS CHARACTERIZATION

It has been performed an assessment of 16 buildings from different parts of Portugal (building adopted nomenclature ED-01 to ED-14) and Spain (ED-15 and ED-16), with different heights, seismic design considerations geometries, among other characteristics. The geometrical and structural aspects of each building were, respectively, presented in Table 1. Table 2 demonstrate the seismic design considerations to design, namely the seismic code considered in the structural design; seismic zone, ductility class, importance class and behaviour factor established to each building. Each parameter is defined as follows:

1. Seismic zone: Dependent on local hazard, the zones are defined by respective national authorities, and prescribed in the seismic code;
2. Ductility class: The classification of seismic-resistant structures based on their capacity to undergo inelastic deformations without significant loss of strength, reflecting how well a structure can dissipate seismic energy through controlled damage mechanisms;
3. Importance class: Building classification is based on the potential consequences of structural collapse, considering risks to human life, public safety, and civil protection in the immediate aftermath of an earthquake, as well as the broader social and economic impacts.

Figure 1 demonstrates images of each studied structure.

Table 6-1: Buildings geometrical characteristics

Building	Year of the project	Height above ground	Number of floors above the ground	Number of underground floors	Inter-storey height of ground floor	Inter-storey height of typical floor	Slab type	Geometry
ED-01	2019	13.8 m	4	2	4.2 m	2.8 m	Flat slab	Rectangular
ED-02	2018	20.3 m	6	1	4.8 m	2.8 m	Flat slab	Irregular
ED-03	2018	19.8 m	6	2	4.2 m	2.8 m	Flat slab	Rectangular in floors 0 and 1 Irregular in subsequent floors
ED-04	2021	42.8 m	9	3	6.1 m	4.2 m	Solid slab	Rectangular

Table 6-1: Cont.

Building	Year of the project	Height above ground	Number of floors above the ground	Number of underground floors	Inter-storey height of ground floor	Inter-storey height of typical floor	Slab type	Geometry
ED-05	2018	29.6 m	9	2	3.8 m	2.8 m	Solid slab	Rectangular in floors 0 and 1 Irregular in subsequent floors
ED-06	2018	36.3 m	12	4	3.9 m	3.0 m	Flat slab	L shape
ED-07	2018	24.5 m	8	4	3.8 m	3.0 m	Flat slab	L shape
ED-08	2019	31.0 m	10	2	2.5 m	2.5 m	Flat slab	Rectangular
ED-09	2019	27.2 m	8	2	3.7 m	2.9 m	Flat slab	Rectangular
ED-10	2020	46.5 m	9	1	7.5 m	3.9 m	Flat slab	Rectangular
ED-11	2018	42.9 m	9	3	5.7 m	3.6 m	Flat slab	Rectangular
ED-12	2019	32.4 m	10	4	3.5 m	2.8 m	Flat slab	Rectangular
ED-13	2019	30.0 m	9	3	2.8 m	2.8 m	Flat slab	Rectangular
ED-14	2007	15.1 m	4	0	4.6 m	3.5 m	Two-way ribbed slab	Rectangular
ED-15	1966	11.1 m	4	0	2.6 m	2.5 m	One-way ribbed slab	H shape
ED-16	1966	33.1 m	11	0	2.6 m	2.5 m	One-way ribbed slab	H shape

Table 6-2: Seismic design parameters

Building	Seismic code	Seismic zones	Ductility class	Importance class	Behaviour coefficient
ED-01	EN 1998-1 [18]	1.6 and 2.4	DCM	II	2.5
ED-02	EN 1998-1 [18]	1.6 and 2.4	DCM	II	2.5
ED-03	EN 1998-1 [18]	1.6 and 2.4	DCM	II	2.5
ED-04	EN 1998-1 [18]	1.6 and 2.4	DCM	II	2.5
ED-05	EN 1998-1 [18]	NI	NI	NI	NI
ED-06	EN 1998-1 [18]	1.3 and 2.3	DCM	II	2.4
ED-07	EN 1998-1 [18]	1.3 and 2.3	DCM	II	3.0
ED-08	EN 1998-1 [18]	1.6 and 2.5	DCL	II	1.5
ED-09	EN 1998-1 [18]	1.6 and 2.5	DCL	II	1.5
ED-10	RSA [28] / REBAP [29]	D	NI	NI	NI
ED-11	EN 1998-1 [18]	1.6 and 2.5	DCL	II	1.5
ED-12	RSA [28] / REBAP [29]	A	NI	NI	1.5
ED-13	RSA [28] / REBAP [29]	A	NI	NI	1.5
ED-14	RSA [28] / REBAP [29]	D	NI	NI	2.5
ED-15	NSD	NSD	NSD	NSD	NSD
ED-16	NSD	NSD	NSD	NSD	NSD

NI: Not informed | NSD: No seismic design

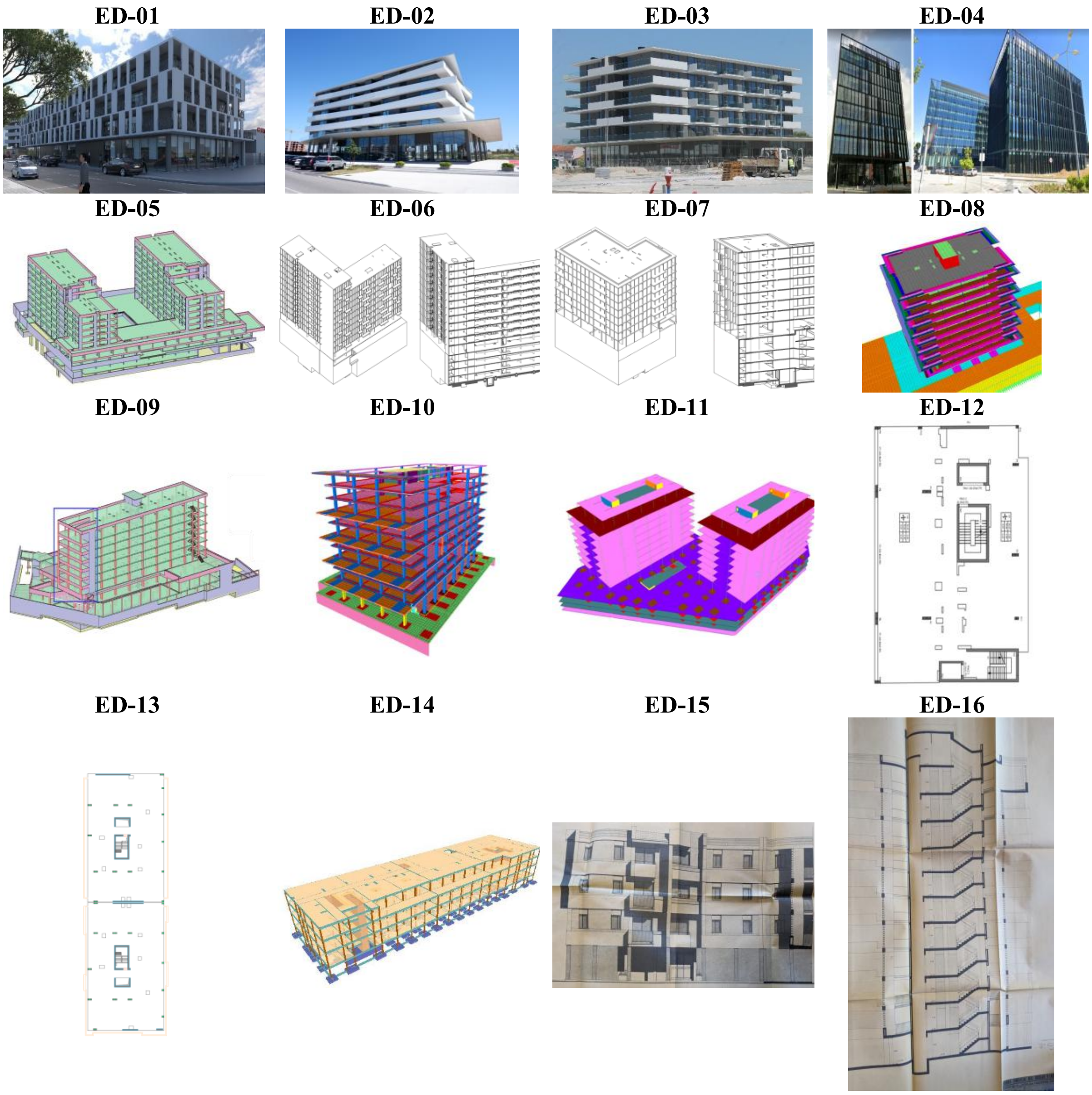


Figure 6-1: Global view of the assessed buildings

6.4.2 IRREGULARITY IN ELEVATION CRITERIA

As described in topic 6.3, to each assessed structure, it has been carried out an analysis of lateral stiffness and mass shift, setbacks and the systems resistant to lateral actions continuity from the foundation to the top. For ratio of actual storey resistance to the resistance required by the analysis criterium, it was considered that the resistance of storey is proportional to the inertia of vertical elements of each floor, such as columns, shear walls, among others. Considering that these inertias was took in account in the lateral stiffness calculus, as demonstrated below, this criterium is covered in this analysis indirectly.

For lateral stiffness variation, it has been considered in calculus of that parameter the lateral stiffness (K_x and K_y)of vertical elements (columns and shear walls). It contemplated all vertical elements elasticity modulus (E_{cm}), inertia (I_{x_c} and I_{y_c}) and height (H_c and H_s) of each accessed floor, as described in the following equations:

$$K_x = \left(\sum_{c=1}^{n} \frac{12 * E_{cm} * I_{x_c}}{{H_c}^3}\right) + \left(\sum_{s=1}^{n} \frac{12 * E_{cm} * I_{x_s}}{{H_s}^3}\right) \tag{1}$$

$$K_y = \left(\sum_{c=1}^{n} \frac{12 * E_{cm} * I_{y_c}}{{H_c}^3}\right) + \left(\sum_{s=1}^{n} \frac{12 * E_{cm} * I_{y_s}}{{H_s}^3}\right) \tag{2}$$

For the resistance criterium, it was assumed, that the strength would be proportional to the ratio of the inertias of the vertical elements on each floor. Hence, this parameter was assessed indirectly in the analysis of lateral stiffness shift.

The setbacks were considered in both directions. Despite the prEN 1998-1-2:2024 [19] does not present this criterium for the vertical irregularity classification, this parameter was considered according to EN 1998-1:2004 [18]. This code establish that each type of setbacks has a limit to be considered. Finally, it was also considered the continuity of resistant system throughout the structure and the irregularity caused by the asymmetrical distribution of infill walls.

6.4.3 BUILDINGS VERTICAL IRREGULARITY ASSESSMENT ACCORDING TO EN 1998-1:2004 AND PREN 1998-1-2:2024

Figure 6-2 presents the criteria of lateral stiffness and mass inter-storey shift assessment of the existing buildings analysed. Table 6-3 demonstrated the verification of setback criteria according to EN 1998-1:2004 [18] in each structure. Finally, Table 6-4 shows all the assessed criteria, including the continuity of resisting system and the classification of irregularity in elevation according to EN 1998-1:2004 [18] and prEN 1998-1-2:2024 [19]. From them it can be drawn:

1. Regarding lateral stiffness and mass inter-storey shift, EN 1998-1:2004 [18] describes those criteria qualitatively, without any numerical comparison methodology. For this study, it was considered the limits established by prEN 1998-1-2:2024 [19] (30% and 150% of inter-storey lateral stiffness and inter-storey mass shift, respectively). From those considerations, all structures fulfilled the mass variation criterium. However, for the lateral stiffness parameter, with exception to structures ED-08, ED-14 and ED-15, the structures do not comply with the limit established by prEN 1998-1-2:2024 [19];
2. For setbacks analysis, only ED-07, ED-08, ED-10, ED-14, ED-15 and ED-16 comply the EN 1998-1:2004 [18] limits. It is important to highlight that, despite prEN 1998-1-2:2024 [19] does not refer to that criterium, it has been considered in the irregularity classification;
3. From Table 6-4, regarding the continuity of resisting system, ED-03, ED-04, ED-05, ED-08, ED-10, ED-15 and ED-16 present discontinuity in ground floor, not fulfilling this criterium;
4. Finally, the classification of vertical irregularity presented in Table 4, demonstrates that only ED-08, ED-15 and ED-16 are regular structures. Even though the differences in criteria assessment between EN 1998-1:2004 [18] and prEN 1998-1-2:2024 [19], the fact that if only one of those aspects were not

complied, the structure should be classified as irregular, the final classification are the same for both codes.

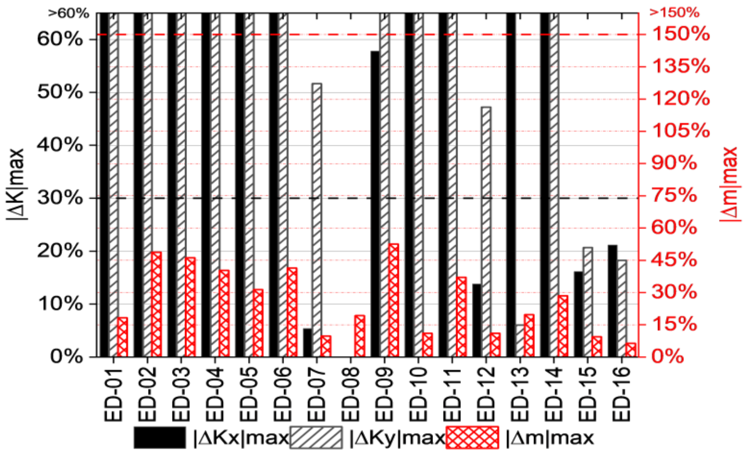


Figure 6-2: Irregularity in elevation criterium: Lateral stiffness and mass inter-storey shift according to prEN1998-1-2:2024 [19]

Table 6-3: Setback verification according to EN 1998-1:2004 [18]

Building	Gradual setbacks preserving axial symmetry		Single setback within the lower 15% of the total height		Sum of the setbacks non symmetrical in both directions		Each setback		Verification
	X	Y	X	Y	X	Y	X	Y	
ED-01	NE	NE	6.6%	26.4%	NE	NE	NE	NE	X
ED-02	NE	NE	16.1%	22.2%	NE	NE	NE	NE	X
ED-03	NE	NE	40.4%	8.5%	NE	NE	NE	NE	X
ED-04	NE	NE	15.9%	24.9%	NE	NE	NE	NE	X
ED-05	NE	NE	NE	NE	52.0%	33.3%	NE	NE	X
ED-06	NE	NE	NE	NE	72.0%	NE	72.0%	NE	X
ED-07	NE	NE	NE	NE	NE	NE	NE	NE	√
ED-08	NE	NE	NE	NE	NE	NE	NE	NE	√
ED-09	NE	NE	NE	NE	4.0%	71.0%	4.0%	46.0%	X
ED-10	NE	NE	NE	NE	4.0%	7.0%	4.0%	7.0%	√
ED-11	16.0%	25.0%	NE	NE	NE	NE	NE	NE	X
ED-12	NE	NE	NE	NE	NE	16.0%	NE	16.0%	X
ED-13	NE	NE	NE	NE	NE	NE	46.0%	34.0%	X
ED-14	NE	NE	NE	NE	NE	NE	NE	NE	√
ED-15	NE	NE	NE	NE	NE	NE	NE	NE	√
ED-16	NE	NE	NE	NE	NE	NE	NE	NE	√

NE: Not existing | √ – Regular criterium | X – Irregular criterium

Table 6-4: Vertical irregularity classification according to EN 1998-1:2004 [18] and prEN1998-1-2:2024 [19]

Vertical irregularity criteria	ED-01	ED-02	ED-03	ED-04	ED-05	ED-06	ED-07	ED-08	ED-09	ED-10	ED-11	ED-12	ED-13	ED-14	ED-15	ED-16
Inter-storey lateral stiffness shift	X	X	X	X	X	X	X	√	X	X	X	X	X	X	√	√
Inter-storey mass shift	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√
Setbacks	X	X	X	X	X	X	√	√	X	√	X	X	X	√	√	√
Continuity of resisting system	X	X	√	√	√	X	X	√	X	√	X	X	X	X	√	√
Regularity in elevation classification	I	I	I	I	I	I	I	R	I	I	I	I	I	I	R	R

√ – Regular criterium | **X** – Irregular criterium. | **R** – Regular | **I** – Irregular

6.4.4 Verification of Regularity in Elevation According to Others Seismic Codes

6.4.4.1 NBR 15421:2023 (Brazil)

NBR 15421:2023 [20] establishes that, to a structure be considered irregular, at least one of the two criteria presented above must not be fulfilled:

1. Discontinuities in the vertical seismic resistance path, such as consecutive vertical resistance elements in the same plane but with axes greater than their length apart, or when resistance between consecutive elements is greater in the upper element;
2. Characterization of an "extremely weak pavement", as one in which its lateral resistance is less than 65% of the resistance of the pavement immediately above. The lateral resistance is computed as the total resistance of all earthquake-resistant elements present in the considered direction.

The assessed buildings vertical irregularity verification regarding NBR 15421:2023 [20] criteria is presented in Table 6-5. Despite the differences of vertical irregularity criteria between EN 1991-1:2002 [18], prEN 19981-2:2024 [19] and NBR 15421:2023 [20], all the structures are classified similarly in each code.

Table 6-5: Criteria for regularity in elevation according to NBR 15421:2023 [20]

Vertical irregularity criteria	ED-01	ED-02	ED-03	ED-04	ED-05	ED-06	ED-07	ED-08	ED-09	ED-10	ED-11	ED-12	ED-13	ED-14	ED-15	ED-16
Discontinuities in the vertical seismic resistance path	X	X	√	√	√	X	X	√	X	√	X	X	X	X	√	√
Characterization of an “extremely weak pavement” (lateral stiffness shift)	X	X	X	X	X	X	X	√	X	X	X	X	X	X	√	√
Regularity in elevation classification	I	I	I	I	I	I	I	R	I	I	I	I	I	I	R	R

√ – Regular criterium | **X** – Irregular criterium | **R** – Regular | **I** – Irregular

6.4.4.2 NTC-RSEE and MOC-2015 (Mexico)

NTC-RSEE [21] and MOC-2015 [22] classifies the building structures as regular, irregular and strongly irregular, without distinction in criteria of irregularity in plan and in elevation.

A structure is classified as irregular when it does not satisfy until three of the requirements indicated below:

1. The distribution in plan of the mass, walls and other resistant elements of the building must be symmetrical with respect to two orthogonal axes;
2. The ratio of elevation and smallest dimension in plan cannot be greater than 4.0 for NTC-RSEE [21] and 2.5 for MOC-2015 [22];
3. The ratio between the length and width of the base cannot exceed 4.0 for NTC-RSEE [21] and 2.5 for MOC-2015 [22];
4. Existing setbacks and overhangs cannot exceed 20% of the dimension in plan, measured parallel to the direction in which the setback or overhang is located;
5. On each floor there must be a rigid diaphragm behaviour;
6. There cannot be any opening in the slab that exceeds 20% of the dimension in plan, measured parallel to the opening. Hollow zones must not appear staggered between adjacent floors and the total opening area must not exceed 20% of the floor area;
7. The mass of each floor cannot exceed 120% for NTC-RSEE [21] and 110% for MOC-2015 [22] of the corresponding floor immediately below, with the exception of the top floor, nor may it be less than 70% of the mass of the floor immediately below. The top floor is exempt from this condition;
8. No floor may have an area greater than 110%, nor less than 70% of the floor area immediately below;
9. Only for NTC-RSEE [21], all the columns have the same height or do not have huge shifts, except for the roof storey;

10. All columns are restricted by floors, in orthogonal directions, by horizontal diaphragms;
11. Stiffness cannot differ between floors by more than 20% for NTC-RSEE [21] and 50% for MOC-2015 [22] from that of the floor immediately below;
12. For MOC-2015 [22], on no floor can the statically calculated torsional eccentricity exceed 10% of the plan dimension of that floor, measured parallel to the mentioned eccentricity;
13. For NTC-RSEE [21], in structures designed considering behaviour factor of 3.0 and 4.0, the lateral strength and design action ratio should be higher than, respectively, 75% and 85% of the average ratios on all floors.

Additionally, for NTC-RSEE [21], a structure is considered strongly irregular when does not comply two or more of the requirements 5, 6, 9, 10, 11, 12 and 13. For MOC-2015 [22], a structure can also be classified as strongly irregular if it meets any of the following requirements:

1. The statically calculated torsional eccentricity exceeds 20% of the floor plan dimension measured parallel to the mentioned eccentricity;
2. The rigidity or shear resistance of the tread exceeds more than 100% that of the tread immediately below;
3. At the same time, fulfil the 2 last conditions described above;
4. Does not fulfil 4 or more regularity conditions described above.

Despite the Mexican codes does not make any difference between plan and vertical irregularity, Table 6-6 presents the assessment of the buildings the criteria normally associated to irregularity in elevation. Considering that if any criterium does not met, the structure should not be regular, the structures was classified as regular and irregular. Analogously of what occurs with Brazilian code, despite the differences between criteria of Mexican codes and Eurocodes, the classification of the structures was the same. Additionally, it was not possible to classify any strongly irregular structure due to the verification of torsional eccentricity.

Table 6-6: Criteria for regularity in elevation according to NTC-RSEE [21] and MOC-2015 [22]

Vertical irregularity criteria	ED-01	ED-02	ED-03	ED-04	ED-05	ED-06	ED-07	ED-08	ED-09	ED-10	ED-11	ED-12	ED-13	ED-14	ED-15	ED-16
Symmetrical distribution resistant elements	X	X	X	X	X	X	X	√	X	X	X	X	X	X	√	√
Ratio of elevation and smallest dimension in plan	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√
Setbacks	√	√	√	√	X	X	√	√	X	√	√	√	X	√	√	√
Rigid diaphragm behaviour	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√
Inter-storey mass shift	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√

√ – Regular criterium | X – Irregular criterium | **R** – Regular | **I** – Irregular

Table 6-6: Cont.

Vertical irregularity criteria	ED-01	ED-02	ED-03	ED-04	ED-05	ED-06	ED-07	ED-08	ED-09	ED-10	ED-11	ED-12	ED-13	ED-14	ED-15	ED-16
Area shift between storeys	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√
Columns restrictions by horizontal diaphragms	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√
Lateral stiffness shift	X	X	X	X	X	X	X	√	X	X	X	X	X	X	√	√
Regularity classification	I	I	I	I	I	I	I	R	I	I	I	I	I	I	R	R

√ – Regular criterium | X – Irregular criterium | R – Regular | I – Irregular

6.4.4.3 ASCE/SEI7-22 (USA)

ASCE/SEI7-22 [23] establishes that a structure must be classified as irregular in elevation if it does not meet one of the following criteria:

1. Stiffness-soft storey irregularity: It exists where there is a storey in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three storeys above;
2. Stiffness-extreme soft storey irregularity: It exists where there is a storey in which the lateral stiffness is less than 60% of that in the story above or less than 70% of the average stiffness of the three floors above;
3. Weight (mass) irregularity: It exists where the effective mass of any storey is more than 150% of the effective mass of an adjacent storey. A roof that is lighter than the floor below need not be considered;
4. Vertical geometric irregularity: It exists where the horizontal dimension of the seismic force-resisting system in any storey is more than 130% of that in an adjacent storey;
5. In-plane discontinuity in vertical lateral force-resisting element irregularity: It exists where there is an in-plane offset of a vertical seismic force-resisting element resulting in overturning demands on supporting structural elements;
6. Discontinuity in lateral strength-weak story irregularity: It exists where the storey lateral strength is less than 80% of that in the story above;
7. Discontinuity in lateral strength-extreme weak story irregularity: It exists where the storey lateral strength is less than 65% of that in the storey above.

Table 6-7 summarizes the ASCE vertical irregularity criteria assessment of studied buildings. Likewise the aforementioned codes, the classification of irregularity in elevation of the buildings according to ASCE/SEI7-22 [23] was similar to observed in EN 1998-1 [18] and prEN 1998-1-2:2024 [19].

Table 6-7: Criteria for regularity in elevation according to ASCE/SEI7-22 [23]

Vertical irregularity criteria	ED-01	ED-02	ED-03	ED-04	ED-05	ED-06	ED-07	ED-08	ED-09	ED-10	ED-11	ED-12	ED-13	ED-14	ED-15	ED-16
Lateral stiffness shift	X	X	X	X	X	X	X	√	X	X	X	X	X	X	√	√
Vertical geometric irregularity	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√
Discontinuity in lateral strength (weak storey)	X	X	X	X	X	X	X	√	X	X	X	X	X	X	√	√
Regularity in elevation classification	**I**	**I**	**I**	**I**	**I**	**I**	**I**	**R**	**I**	**I**	**I**	**I**	**I**	**I**	**R**	**R**

√ – Regular criterium | **X** – Irregular criterium | **R** – Regular | **I** – Irregular

6.4.4.4 NZS 1170.5 (New Zealand)

NZS 1170.5 [24] establishes that a structure must be classified as irregular in elevation if it does not meet at least one of the following criteria:

1. One storey was much taller than the adjoining storeys and the resulting decrease in stiffness that would normally occur was not, or could not be, compensated for;
2. The ratio of mass to stiffness in adjoining storeys differs significantly. It was considered that parameter;
3. The smaller dimension was below the larger dimension, thereby creating an inverted pyramid effect;
4. Significant horizontal offsets in the vertical elements of the horizontal force resisting system at one or more levels,
5. Abrupt changes in strength capacity between storeys

For criterium of height shift, it was considered unmet if the height shift between storey is higher than 1.0 m. For the second criterium, as this code does not present any numerical reference, leading to a qualitative analysis, it was considered the reference of 30% shift in lateral stiffness and 150% of mass shift as prescribed by prEN 1998-1-2:2024 [19]. The third and fourth criteria were assessed qualitatively. Finally, the strength capacity between storeys is considered analogously as defined by prEN 1998-1-2:2024 [19], due to the lack of qualitative parameters in NZS 1170.5 [24].

Table 6-8 demonstrates the analysis of irregularity in elevation criteria assessment of buildings according to NZS 1170.5 [24]. As occurred previously in the aforementioned seismic codes, the vertical irregularity classification according to NZS 1170.5 [24] was similar to EN 1998-1:2004 [18] and prEN 1998-1-2:2024 [19].

Table 6-8: Criteria for regularity in elevation according to NZS 1170.5 [24]

Vertical irregularity criteria	**ED-01**	**ED-02**	**ED-03**	**ED-04**	**ED-05**	**ED-06**	**ED-07**	**ED-08**	**ED-09**	**ED-10**	**ED-11**	**ED-12**	**ED-13**	**ED-14**	**ED-15**	**ED-16**
Height shift	X	X	X	X	√	√	√	√	√	X	X	√	√	X	√	√
Ratio of mass to stiffness	X	X	X	X	X	X	X	√	X	X	X	√	X	X	√	√
Inverted pyramid effect	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√
Significant horizontal offsets	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√	√
Strength capacity shift	X	X	X	X	X	X	X	√	X	X	X	X	X	X	√	√
Regularity in elevation classification	I	I	I	I	I	I	I	R	I	I	I	I	I	I	R	R

√ – Regular criterium | **X** – Irregular criterium | **R** – Regular | **I** – Irregular

6.4.4.5 Seismic codes comparison on vertical irregularity

As a closure of this section, this subsection compares the vertical irregularity classification on all buildings, throughout all assessed seismic codes. Table 6-9 presents those results.

Table 6-9: Vertical irregularity classification comparison throughout all assessed seismic codes

Seismic code	**ED-01**	**ED-02**	**ED-03**	**ED-04**	**ED-05**	**ED-06**	**ED-07**	**ED-08**	**ED-09**	**ED-10**	**ED-11**	**ED-12**	**ED-13**	**ED-14**	**ED-15**	**ED-16**
EN 1998-1:2004 | prEN 1998-1-2:2024	I	I	I	I	I	I	I	R	I	I	I	I	I	I	R	R
NBR 15421:2023	I	I	I	I	I	I	I	R	I	I	I	I	I	I	R	R
NTC-RSEE | MOC-2015	I	I	I	I	I	I	I	R	I	I	I	I	I	I	R	R
ASCE/SEI7-22	I	I	I	I	I	I	I	R	I	I	I	I	I	I	R	R
NZS 1170.5	I	I	I	I	I	I	I	R	I	I	I	I	I	I	R	R

R – Regular | **I** – Irregular

From Table 6-9 and the discussion performed above, it can be inferred that:

1. On vertical irregularity criteria, there are differences between the assessed seismic codes in the qualitative or quantitative nature of the criterium, or in the non-consideration of a specific criterium itself;
2. Despite the differences discussed in (1), since all assessed seismic codes consider a structure irregular in elevation if at least one of the vertical irregularity

criteria was not complied, the final classification of all buildings was similar in all assessed seismic codes;

3. As discussed by Santos et al. [16], there are considerable differences in maturity between several seismic codes. Further studies should be done to improve those standards, especially in the clarification of the irregularity criteria.

6.5 SEISMIC RESPONSE OF EXISTING BUILDINGS

In this section, the seismic response assessment of buildings ED-1, ED-14, ED-15 and ED-16 have been carried out. Those buildings were chosen considering the following aspects:

1. Irregularity in elevation classification;
2. Seismic standard considered in the design;
3. Access to structural blueprints;
4. Access to design criteria information, such as concrete and rebars characteristics, loads consideration and combinations, seismic design information, calculus methodology adopted in design, etc.

Despite the differences in design considerations, structural geometries, number of floors and inter-storey heights observed in those chosen buildings, the main point of this assessment is to evaluate the impact of irregularity in elevation and the seismic design consideration or not in seismic response of existing buildings, even though the inherent limitation of this analysis.

The seismic response assessment of those structures was carried out in the SeismoStruct v2025 [25,26] software by an adaptative push-over analysis. To simulate RC elements, it was employed inelastic force-based frame elements, subdivided into seven integration sections. Each section was further discretized into 150 fibres to accurately capture the nonlinear behaviour of the materials. To represent the mechanical response of reinforcement steel and concrete, the uniaxial constitutive model developed by Monti and Nuti [30] was used for steel, while the model proposed by Mander et al. [31] was utilized for concrete.

Table 6-10 presents the concrete and rebars characteristics considered in each assessed building. As it can be seen ED-1 and ED-14 have similar concrete and rebars properties. Those are more recent projects, as highlighted in Table 2. However, the seismic code considered in the design to each building were different. For ED-15 and ED-16, they considered only gravity loads in the design. Additionally, those buildings were ancient projects, as shown in Table 6-2. Additionally, the concrete and rebars have lower resistance.

Table 6-10: Concrete and rebars mechanical properties considered in each model

Building	ED-01	ED-14	ED-15	ED-16
Concrete class	C30/37	C30/37	C20/25	C20/25
f_{ck}(MPa)	30.0	30.0	20.0	20.0
f_{cm}(MPa)	38.0	38.0	28.0	28.0
f_{ctm}(MPa)	2.9	2.9	2.2	2.2
E_{cm}(GPa)	33.0	33.0	30.0	30.0
Rebar class	S500	S500	S400	S400
f_{yk}(MPa)	500.0	500.0	400.0	400.0
f_{ym}(MPa)	555.0	555.0	444.4	444.4
f_{yd}(MPa)	435.0	435.0	347.0	347.0
E_s(GPa)	200.0	200.0	200.0	200.0

Figure 6-3 demonstrate the seismic response of the aforementioned structures. Figure 6-3a shows the capacity curves and idealized curves of each building, with their correspondent limit states (Damage limitation, significant damage and near collapse). Figures 6-3b and 6-3c illustrate, respectively, the first occurrence of performance criteria related with the correspondent base shear and displacement. Finally, Figures 6-3d and 6-3e exhibit, respectively, the inter-storey drift and global drift of each building. Additionally, Figures 6-4 demonstrate, respectively, the chord rotation and shear capacity performance of columns, shear walls and beams of each assessed structure.

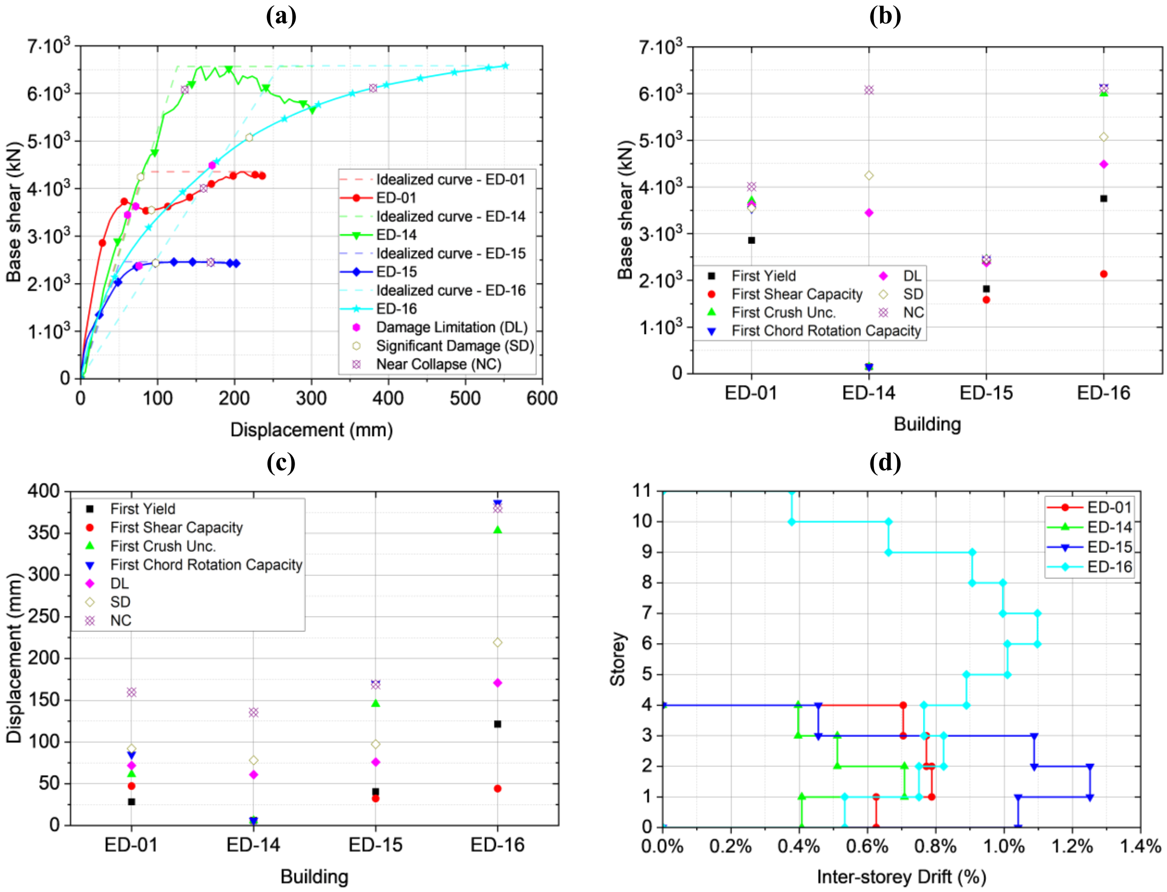


Figure 6-3: Seismic response of existing buildings: (a) Capacity curves; (b) critical points of capacity curves – base shear; (c) critical points of capacity curves – displacement; (d) inter-storey drift and (e) global drift

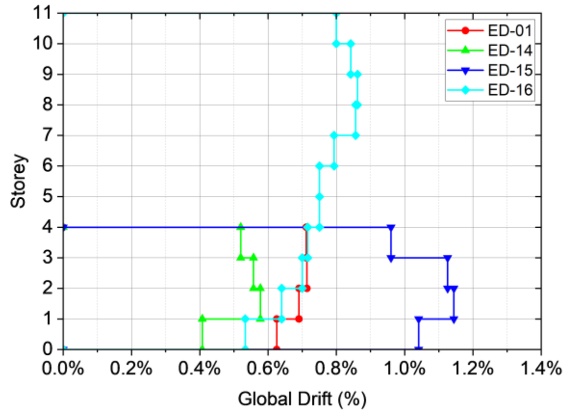


Figure 6-3: Cont.

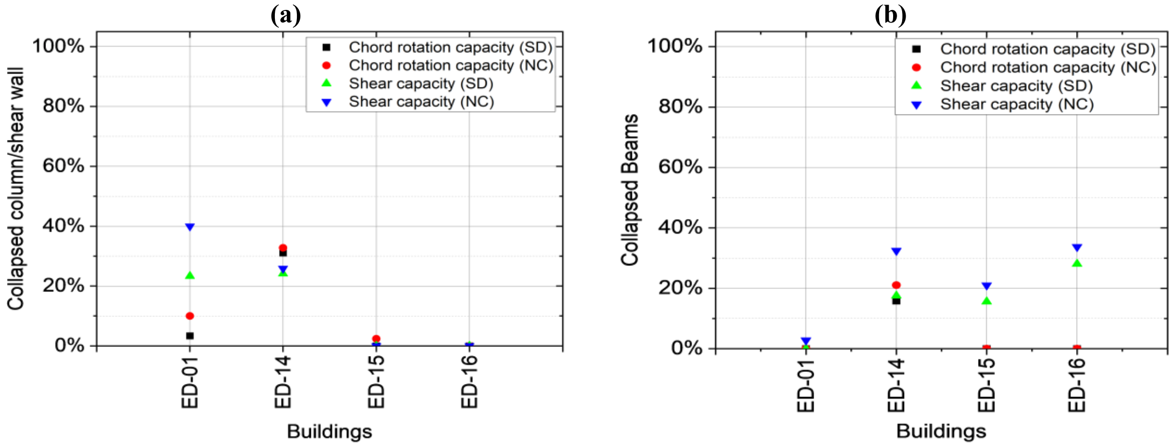


Figure 6-4: Collapsed structural elements on chord rotation and shear capacity (a) Columns/shear walls and (b) beams

From Table 6-10 and Figures 6-3 and 6-4, it can be drawn:

1. Regarding the capacity curves demonstrated in Figure 6-3a, despite ED-01 and ED-14 present the same number of storeys, the maximum load was considerably higher in ED-14. It is important to highlight that ED-01 was a residential building and ED-14 a commercial one. Additionally, they were designed considering different seismic codes;

2. Still in Figure 6-3a, but comparing regular and irregular structures, despite ED-01, ED-14 and ED-15 have the same number of storeys, ED-15 presented a substantial worse behaviour on capacity curve, despite the structural regularity of the structure. However, it can be explained by the fact that ED-15 did not be designed considering seismic loads. For the ED-16, that also was not designed to seismic load, the maximum base shear was similar to ED-14, even though ED-16 was much higher structure, which allows bigger displacements. If this one would have considered seismic load in the design, it probably would behave much better

in the parameters. Additionally, despite the differences between several parameters, all buildings present ductile behaviour;

3. As demonstrated in Figures 6-3b and 6-3c, ED-14, designed according to RSA [28] and REBAP [29], presented first appearance of critical points in capacity curve considerably earlier than the other structures. Despite RSA [28] and REBAP [29] consider seismic design, those does not take into account capacity design and does not assess plastic behaviour of the structures due to seismic load. That might explain that behaviour;
4. Still in Figures 6-3b and 6-3c, ED-01 and ED-15 behave similarly regarding to first appearance of critical points of capacity curves. ED-16, nevertheless, presents more spaced between first occurrence of performance criteria, despite this building was not designed considering seismic load;
5. In Figure 6-4, it can be seen that ED-15 and ED-16 presents failures on beams due to shear capacity. Additionally, despite those structures do not consider seismic loads in design, no columns collapsed due to chord rotation or shear performance capacities. Those structural behaviour on seismic load was defined by low beams capacity. Probably, if seismic load has been considered in the design of those structures, they would present more capacity, by the increasing of beams´ strength. Furthermore, observing ED-01 and ED-14 behaviour in that matter, it can be seen ED-01, designed by EN 1998-1:2004 [18], presents low percentage of collapsed columns and beams on significant damages level. These are expected on structures designed considering seismic loads. ED-14, nonetheless, despite of been designed considering those loads, presents notable ruptures on columns and beams at the significant damage level;
6. In Figures 6-3d and 6-3e, ED-15 present, both in inter-storey drift and global drift, the worst behaviour (maximum of 1.25% and 1.14% respectively). ED-01, ED-14 and ED-16 presents respectively, maximum inter-storey drifts of 0.85%, 0.71% and 1.10%. For global drifts, those buildings demonstrate values lower than 1.0%. Furthermore, ED-15 and ED-16 show considerably higher shifts both in inter-storey drifts and global drifts in comparison to other buildings. That occurs probably to the non-seismic design of those structures.

6.6 SUMMARY AND CONCLUSIONS

This study assessed sixteen existing building frame and shear wall structures, conducting a comprehensive analysis of vertical irregularity classification throughout several seismic codes and seismic response of four of those buildings.

The following main conclusions can be drawn:

1. EN 1998-1:2004 [18], prEN 1998-1-2:2024 [19], NBR 15421:2023 [20], NTC-RSEE [21], MOC-2015 [22], ASCE/SEI7-22 [23] and NZS 1170.5 [24] present important differences in vertical irregularity criteria, whether in the qualitative or quantitative nature or even in the non-consideration of a specific criterium itself. Nonetheless, since all of them consider a structure irregular in elevation if at least one of the vertical irregularity criteria was not complied, the final classification of almost all buildings was similar in all assessed seismic codes.

This first conclusion should not lead to the premise that it would happen in all cases. Further studies should be done to improve those standards, especially in the clarification of the irregularity criteria;

2. The comparison of structural response of ED-01 and ED-14, designed by different seismic codes, demonstrate the impact of the non-consideration of capacity design and the seismic design dismissing the plastic behaviour. Despite the higher capacity observe in ED-14, it is important to observe the differences on those structures, such as structural use (habitational and commercial) and seismic design approaches. Additionally, despite the capacity curve progression on ED-14, this building presents highest failures both on beams and columns on significant damage level in comparison to ED-01;
3. The non-seismic designed assessed buildings presented highest shifts in inter-storey and global drifts. Additionally, the maximum values of those parameters do not overtake 1.25% and 1.14%, respectively, which are higher than the limit defined by EN 1998-1:2004 [18] and prEN 1998-1-2:2024 [19];
4. ED-15 and ED-16, buildings designed dismissing seismic loads, present structural behaviour governed by beams shear capacity. These ones demonstrate low capacities on beams, that limited the structural performance. Those results show the importance of seismic design to better structural behaviour.

6.7 REFERENCES

[1] Vuran, E., C. Serhatoğlu, M. Ö. Timurağaoğlu, E. Smyrou, İ. E. Bal, and R. Livaoğlu. 2024. “Damage Observations of RC Buildings from 2023 Kahramanmaraş Earthquake Sequence and Discussion on the Seismic Code Regulations.” *Bulletin of Earthquake Engineering*, 1–30. https://doi.org/10.1007/s10518-023-01843-3.

[2] Aktas, Y., E. So, C. Johnson, K. Donmez, A. T. Ozden, A. Parammal Vatteri, A. O’Kane, A. Kalkan, A. Andonov, and E. Verrucci. 2024. “The Türkiye Earthquake Sequence of February 2023: A Longitudinal Study Report by EEFIT.” London, UK: EEFIT.

[3] Zhao, C. P., Z. L. Chen, L. Q. Zhou, Z. X. Li, and Y. Kang. 2010. “Rupture Process of the Wenchuan M8.0 Earthquake of Sichuan China: The Segmentation Feature.” *Chinese Science Bulletin* 55 (3): 284–92. https://doi.org/10.1007/s11434-009-0425-7.

[4] Liu-Zeng, J., Z. Zhang, L. Wen, P. Tapponnier, J. Sun, X. Xing, G. Hu, et al. 2009. “Co-Seismic Ruptures of the 12 May 2008, Ms 8.0 Wenchuan Earthquake, Sichuan: East-West Crustal Shortening on Oblique, Parallel Thrusts along the Eastern Edge of Tibet.” *Earth and Planetary Science Letters* 286 (3–4): 355–70. https://doi.org/10.1016/j.epsl.2009.07.017.

[5] Palermo, M., R. R. Hernandez, S. Mazzoni, and T. Trombetti. 2014. “On the Seismic Behavior of a Reinforced Concrete Building with Masonry Infills Collapsed during the 2009 L’aquila Earthquake.” *Earthquake and Structures* 6 (1): 45–69. https://doi.org/10.12989/eas.2014.6.1.045.

[6] Romão, X., A. A. Costa, E. Paupério, H. Rodrigues, R. Vicente, H. Varum, and A. Costa. 2013. “Field Observations and Interpretation of the Structural Performance of

Constructions after the 11 May 2011 Lorca Earthquake." *Engineering Failure Analysis* 34: 670–92. https://doi.org/10.1016/j.engfailanal.2013.01.040.

[7] Magliulo, G., M. Ercolino, C. Petrone, O. Coppola, and G. Manfredi. 2014. "The Emilia Earthquake: Seismic Performance of Precast Reinforced Concrete Buildings." *Earthquake Spectra* 30 (2): 891–912. https://doi.org/10.1193/091012EQS285M.

[8] Bournas, D. A., P. Negro, and F. F. Taucer. 2014. "Performance of Industrial Buildings during the Emilia Earthquakes in Northern Italy and Recommendations for Their Strengthening." *Bulletin of Earthquake Engineering* 12 (5): 2383–2404. https://doi.org/10.1007/s10518-013-9466-z.

[9] Novelli, V., D. Alexander, and E. Verrucci. 2012. "The 29th May 2012 Emilia Romagna Earthquake EPICentre Field Observation Report No. EPI-FO-290512." www.epicentreonline.com.

[10] Varum, H., A. Furtado, H. Rodrigues, J. Dias-Oliveira, N. Vila-Pouca, and A. Arêde. 2017. "Seismic Performance of the Infill Masonry Walls and Ambient Vibration Tests after the Ghorka 2015, Nepal Earthquake." *Bulletin of Earthquake Engineering* 15 (3): 1185–1212. https://doi.org/10.1007/s10518-016-9999-z.

[11] Rodrigues, H., A. Furtado, N. Vila-Pouca, H. Varum, and A. R. Barbosa. 2018. "Seismic Assessment of a School Building in Nepal and Analysis of Retrofitting Solutions." *International Journal of Civil Engineering* 16 (11): 1573–89. https://doi.org/10.1007/s40999-018-0297-9.

[12] Dizhur, D., R. P. Dhakal, J. Bothara, and J. M. Ingham. 2016. "Building Typologies and Failure Modes Observed in the 2015 Gorkha (Nepal) Earthquake." *Bulletin of the New Zealand Society for Earthquake Engineering* 49 (2). https://doi.org/10.5459/bnzsee.49.2.211-232.

[13] Tena-Colunga, A. 2021. "Conditions of Structural Irregularity. Relationships with Observed Earthquake Damage in Mexico City in 2017." *Soil Dynamics and Earthquake Engineering* 143. https://doi.org/10.1016/j.soildyn.2021.106630.

[14] Braga, F., V. Manfredi, A. Masi, A. Salvatori, and M. Vona. 2011. "Performance of Non-Structural Elements in RC Buildings during the L'Aquila, 2009 Earthquake." *Bulletin of Earthquake Engineering* 9 (1): 307–24. https://doi.org/10.1007/s10518-010-9205-7.

[15] Verderame, G. M., F. De Luca, P. Ricci, and G. Manfredi. 2011. "Preliminary Analysis of a Soft-Storey Mechanism after the 2009 L'Aquila Earthquake." *Earthquake Engineering and Structural Dynamics* 40 (8): 925–44. https://doi.org/10.1002/eqe.1069.

[16] Santos, D., J. Melo, and H. Varum. 2024. "Code Requirements for the Seismic Design of Irregular Elevation RC Structures." *Buildings* 14 (5): 1351. https://doi.org/10.3390/buildings14051351.

[17] Varum, H., J. Melo, A. Furtado, and A. Lima. 2022. "Irregularities in RC Buildings: Perspectives in Current Seismic Design Codes, Difficulties in Their Application and Further Research Needs." In *Seismic Behaviour and Design of Irregular and Complex Civil Structures IV*, edited by Rita Bento, Mario De Stefano, Dietlinde Köber, and Zbigniew Zembaty. Vol. 50. Geotechnical, Geological and Earthquake Engineering. Cham: Springer International Publishing. https://doi.org/10.1007/978-3-030-83221-6.

[18] EN 1998-1. *“Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings.”* European Comittee for Standardization: Brussels, Belgium. 2004.

[19] prEN 1998-1-2. *“Eurocode 8: Design of Structures for Earthquake Resistance - Part 1-2: Rules for New Buildings, Draft Post-ENQ 48th Meeting CEN/TC250/SC8.”* European Comittee for Standardization: Brussels, Belgium. 2024.

[20] NBR 15421. *“Projeto de estruturas resistentes a sismos - Procedimento.”* Associação Brasileira de Normas Técnicas, Rio de Janeiro, 2023.

[21] NTC-RSEE. *“Administración Pública de la Ciudad de México Norma Técnica Complementaria Para La Revisión de La Seguridad Estructural de Las Edificaciones.”* Órgano de Difusión del Gobierno de la Ciudad de México: México City, Mexico, 2017.

[22] MOC-2015. *“Manual de Diseño de Obras Civiles: Diseño Por Sismo.”* Comisión Federal de Electricidad: México City, Mexico, 2015.

[23] ASCE/SEI7-22. *“Minimum Design Loads and Associated Criteria for Buildings and Other Structures.”* American Society of Civil Engineers: Reston, VA, USA, 2021.

[24] NZS1170-5. *“Structural Design Actions—Part 5: Earthquake Actions—New Zealand Commentary.”* Standards New Zealand: Wellington, New Zealand, 2004.

[25] Seismosoft. 2021. “SeismoStruct 2021 – A Computer Program for Static and Dynamic Nonlinear Analysis of Framed Structures.”.

[26] Seismosoft. 2022. “SeismoStruct Verification Report.”

[27] EN 1990. *“Eurocode - Basis of structural design.”* European Comittee for Standardization: Brussels, Belgium, 2002.

[28] RSA. *“Regulamento de Segurança e Acções para Estruturas de Edifícios e Pontes.”* Decreto-lei no235/83. Lisbon, Portugal, 1983.

[29] REBAP. *“Regulamento de estruturas de betão armado e pré-esforçado.”* Decreto-Lei n.o 349-C/83. Lisbon, Portugal, 1983.

[30] Monti, G., C. Nuti, and S. Santini. 1996. “CYRUS-Cyclic Response of Upgraded Sections.” *University of Chieti*, Italy.

[31] Mander, J. B., M. J. N. Priestley, and R. Park. 1988. “Theoretical Stress-Strain Model for Confined Concrete.” *Journal of Structural Engineering* 114 (8): 1804–26. https://doi.org/10.1061/(ASCE)0733-9445(1988)114:8(1804).

6.8 NOMENCLATURE

E_{cm}	Elastic modulus of concrete
E_s	Elastic modulus of reinforcement
f_{ck}	Characteristic compressive strength of concrete
f_{cm}	Mean compressive strength of concrete
f_{ctm}	Mean tensile strength of concrete
f_{yk}	Characteristic yield strength of reinforcement
f_{ym}	Mean strength of reinforcement
f_{yd}	Design yield strength of reinforcement
I_{x_c}, I_{x_s}	Inertial moment in the x-direction of columns and shear walls, respectively
I_{y_c}, I_{y_s}	Inertial moment in the y-direction of columns and shear walls, respectively
H_c , H_s	Inter-storey height of columns and shear walls, respectively
DCM	Ductility class medium
DCL	Ductility class low
MRF	Moment-resisting frames
RC	Reinforced concrete

7 Closure and future research

7.1 GENERAL CONCLUSIONS

This thesis presents a comprehensive investigation into the seismic design provisions related to vertical irregularities, as defined by both the current first generation of Eurocode 8 and the evolving guidelines of its second-generation counterpart. Through rigorous analysis and critical evaluation, the study offers new insights into the criteria for identifying and assessing vertical irregularities, as well as their implications for the design, safety, and seismic performance of reinforced concrete structures. The findings contribute meaningfully to the academic discourse while also offering practical value for structural engineering practice.

Amidst the ongoing evolution of seismic design codes, academic studies in seismic engineering field, work environment rising challenges and the impact of several seismic events, the findings of this study underscore the necessity for the engineering community to critically reassess conventional methodologies, embrace emerging innovations, and recalibrate established practices in alignment with the evolving seismic design paradigm.

The discussion of vertical irregularities criteria in several seismic codes reached to few considerations:

1. The standards indicate a consensus regarding the desired characteristics of a seismic resistant structure, which include simplicity, symmetry, uniformity and redundancies;
2. Although not all analysed seismic codes display the same level of development and approach to the subject matter, it is evident that all seismic standards address the issue of structural irregularities;
3. There are significant variations in the representation of seismic hazard. There are different criteria to assess the same structure in the assessed seismic codes. Additionally, there are distinct ways to quantify each criterium throughout the assessed seismic codes or even there are a qualitative approach to analyse some of those criteria. Those facts can cause confusion to the structural engineering professionals, making it challenging to comprehend the extent of the impact caused by vertical irregularities.

In the assessment of 5 storey MRF RC buildings, all regulars in plan, but with imposed irregularity in elevation by height increasing, the following considerations can be done:

1. Increasing the height of the ground or intermediate floor has deleterious effects on lateral stiffness, top displacement, inter-storey drift, and base shear. Normally the soft storey in ground floor is more damaging to the seismic response than the same in middle storey;
2. Columns cross-section shifts plays an important role in base shear and inter storey drift behaviour, therefore in seismic response. However, the height increasing, hence, the shift in lateral stiffness produced higher impacts in that matter;
3. The irregularity in elevation due to the shift in resistance or mass between floors did not significantly affect the seismic response. However, even though this can be corroborated by several published work, this must be more scrutinized by more study and assessment of several structures with different shapes, materials´

properties. Additionally, lateral stiffness imposes more easily the vertical irregularity in comparison to aforementioned variables.

Regarding the combined effect of rebars corrosion and vertical irregularities in MRF RC 5-10 storey buildings, it can be concluded that:

1. 10% of corrosion was the level that started to present a substantial impact in seismic response;
2. Corrosion does not significantly impact the inter-storey drifts, regardless the different levels of vertical irregularities;
3. Despite corrosion impacts decreasing structural capacity, the irregularity in elevation produces worst impacts in seismic response. However, the combined effect substantially increased the seismic vulnerability.

However, it is important to emphasize that, from chapter 4, those conclusions are limited to the assessed structural typology. it is important to assess also shear wall structures, different RC resistance, structural geometries and, especially the combined impact of irregularity in plan and corrosion in seismic response.

From the behaviour factor analysis in RC MRF and shear wall structures, according to first and second generation of Eurocode 8, it could be observed that:

1. Irregular structures due to the maximum lateral stiffness shift slightly above 30% present structural response parameters close to regular buildings;
2. In the behaviour factor assessment according to second generation of Eurocode 8, there is a trend of reduction, in different levels, in behaviour factor as the maximum inter-storey lateral stiffness shift increases, hence the elevation irregularity increment;
3. Finally, considering the results, it was proposed a classification of regular, irregular and strongly irregular structures. It is suggested that the lateral stiffness shift to an irregular structure should be between 30% and 60%. For strongly irregular structures, this value shall be above 60%. Additionally, it is recommended a reduction of 20% of behaviour factor for irregular structures and 30% for strongly irregular ones, as can be seen in Mexican and North American seismic codes.

Finally, it was assessed several RC existing buildings with different geometries, seismic codes considered in design, materials´ properties. From that, the main conclusions were:

1. Corroborating to observed in chapter 2, the assessed seismic codes present important differences in vertical irregularity criteria, whether in the qualitative or quantitative nature of the criterium, or in the non-consideration of a specific criterium itself. However, since all of them consider a structure irregular in elevation if at least one of the vertical irregularity criteria was not complied, the final classification of all buildings was similar in all assessed seismic codes;
2. Capacity design and the seismic design dismissing the plastic behaviour impacts substantially the seismic response. Furthermore, the non-seismic designed assessed buildings presented worst behaviour in inter-storey and global drifts.

Readers are encouraged to refer to the relevant chapters for a more thorough analysis and comprehensive conclusions.

7.2 FUTURE RESEARCH

To further advance the research initiated in this thesis on structural irregularities in seismic design, it is essential to broaden the scope of investigation to encompass a wider range of structural typologies. Such an expansion will facilitate a more comprehensive analysis of the parameters influencing seismic response and enhance the understanding of the performance of reinforced concrete buildings subjected to seismic loading.

For future research, it is proposed to conduct computational analyses on a larger and more diverse set of RC structures. This expanded dataset will enable the development of a robust foundation for capturing a broad range of design scenarios and seismic response characteristics. From the study performed in this thesis, it can be suggested for future studies:

1. Assess the irregularity criteria impact in different ductility levels. This analysis may provide insights into how varying degrees of ductility, combined with structural irregularities, affect the structural capacity to withstand and dissipate seismic energy;
2. Evaluate the impact of irregularity in plan criteria and the combined effect of irregularity in plan and elevation criteria in seismic response. This is important to better understanding the most important criteria in irregularity, to better prevent structural damages and collapses;
3. Conduct a comparative analysis of various structural systems in the context of irregularities. The structural typology, whether moment-resisting frame systems, wall systems, or dual systems, plays a critical role in the distribution and management of seismic forces. Examining these different systems will provide insight into their respective strengths, limitations, and vulnerabilities under seismic loading conditions;
4. Investigate the influence of span variation in seismic response. The number and configuration of spans significantly affect a structure's lateral stiffness and mass distribution, both of which are critical parameters in seismic design. Analysing buildings with different span arrangements will enhance understanding of how span variation influences seismic resilience and overall structural performance under earthquake excitation;
5. Consider the materials´ properties impact in seismic response. It is imperative to assess structures regarding irregularities with different concrete and rebars properties, to better predict ancient buildings non seismic designed and current buildings behaviour in seismic events;
6. Furthermore, considering the impact of corrosion and the growing studies in the field of seismic engineering, combined with corrosion knowledge, it is crucial to assess, numerically and experimentally the impact of corrosion in seismic response in RC structures. This work presented a numerical analysis in this matter. However is important to perform a wider analysis in this matter, specially considering experimental studies;
7. Assess the impact of infill walls in seismic response, considering the aforementioned suggestions for future works and results obtained in this research;

8. Assess a wider range of existing buildings, considering the above discussion. This is important to better evaluate the existing buildings seismic behaviour, to better prevent catastrophes and, combined to the seismic events data, to teach us good lessons on structural behaviour in earthquakes.

Future research should consider a range of finite element modelling (FEM) techniques to advance the understanding of structural performance under seismic loading conditions, namely time history analysis. It is crucial to model the nodes to determine whether their behaviour aligns with the findings of the present study. Furthermore, walls should be modelled using shell or plate elements to accurately capture their response. Various computational models should be implemented to evaluate their influence on design outcomes in accordance with the provisions of the second generation of Eurocode 8.

Additionally, another research avenue involves designing several structures using seismic codes from other regions, such as the United States, New Zealand, Chile, and Mexico. These designs can then be compared to those based on the second generation of Eurocode 8 to evaluate differences and potential improvements in seismic resilience and safety standards.

This structured approach to defining the study object through these variables will ensure that the research comprehensively covers the critical factors influencing seismic design. By comparing and analysing different families of structures defined by these variables, future research can develop more refined and effective seismic design recommendations, enhancing the safety and sustainability of reinforced concrete buildings in seismic regions. This research will validate the changes proposed by the second generation of Eurocode 8 and contribute to its evolution, ensuring it remains responsive to the needs of modern structural engineering.

Appendix A

A.1 Moment resisting frames – Chapter 3 structures

A1.1 Columns

Structure	Structural element	Storey	Section (m) b x h	Long. rebar ratio (%)	Stirrups
A1	External columns	1	0.55x0.55	1.87%	Ø8c7.0cm
		2	0.55x0.55	1.87%	Ø8c7.0cm
		3	0.55x0.55	1.87%	Ø8c7.0cm
		4	0.55x0.55	1.87%	Ø8c7.0cm
		5	0.55x0.55	1.87%	Ø8c7.0cm
	Internal columns	1	0.55x0.55	1.87%	Ø8c7.0cm
		2	0.55x0.55	1.87%	Ø8c7.0cm
		3	0.55x0.55	1.87%	Ø8c7.0cm
		4	0.55x0.55	1.87%	Ø8c7.0cm
		5	0.55x0.55	1.87%	Ø8c7.0cm
A2	External columns	1	0.55x0.55	1.45%	Ø8c7.0cm
		2	0.55x0.55	1.45%	Ø8c7.0cm
		3	0.55x0.55	1.45%	Ø8c7.0cm
		4	0.55x0.55	1.45%	Ø8c7.0cm
		5	0.55x0.55	1.45%	Ø8c7.0cm
	Internal columns	1	0.55x0.55	1.45%	Ø8c7.0cm
		2	0.55x0.55	1.45%	Ø8c7.0cm
		3	0.55x0.55	1.45%	Ø8c7.0cm
		4	0.55x0.55	1.45%	Ø8c7.0cm
		5	0.55x0.55	1.45%	Ø8c7.0cm
A3	External columns	1	0.55x0.55	1.06%	Ø8c7.0cm
		2	0.55x0.55	1.06%	Ø8c7.0cm
		3	0.50x0.50	1.29%	Ø8c7.0cm
		4	0.50x0.50	1.29%	Ø8c7.0cm
		5	0.45x0.45	1.07%	Ø8c7.0cm
	Internal columns	1	0.55x0.55	1.06%	Ø8c7.0cm
		2	0.55x0.55	1.06%	Ø8c7.0cm
		3	0.50x0.50	1.29%	Ø8c7.0cm
		4	0.50x0.50	1.29%	Ø8c7.0cm
		5	0.45x0.45	1.07%	Ø8c7.0cm
A4	External columns	1	0.55x0.55	1.06%	Ø8c7.0cm
		2	0.55x0.55	1.06%	Ø8c7.0cm
		3	0.50x0.50	1.29%	Ø8c7.0cm
		4	0.50x0.50	1.29%	Ø8c7.0cm
		5	0.45x0.45	1.07%	Ø8c7.0cm
	Internal columns	1	0.55x0.55	1.06%	Ø8c7.0cm
		2	0.55x0.55	1.06%	Ø8c7.0cm
		3	0.50x0.50	1.29%	Ø8c7.0cm
		4	0.50x0.50	1.29%	Ø8c7.0cm
		5	0.45x0.45	1.07%	Ø8c7.0cm

Structure	Structural element	Storey	Section (m) b x h	Long. rebar ratio (%)	Stirrups
A5	External columns	1	0.55x0.55	1,06%	Ø8c7.0cm
		2	0.55x0.55	1,06%	Ø8c7.0cm
		3	0.55x0.55	1,06%	Ø8c7.0cm
		4	0.45x0.45	1.07%	Ø8c7.0cm
		5	0.45x0.45	1.07%	Ø8c7.0cm
	Internal columns	1	0.55x0.55	1,06%	Ø8c7.0cm
		2	0.55x0.55	1,06%	Ø8c7.0cm
		3	0.55x0.55	1,06%	Ø8c7.0cm
		4	0.45x0.45	1.07%	Ø8c7.0cm
		5	0.45x0.45	1.07%	Ø8c7.0cm
B1	External columns	1	0.75x0.75	2.44%	Ø8c7.0cm
		2	0.75x0.75	2.44%	Ø8c7.0cm
		3	0.75x0.75	2.44%	Ø8c7.0cm
		4	0.75x0.75	2.44%	Ø8c7.0cm
		5	0.75x0.75	2.44%	Ø8c7.0cm
	Internal columns	1	0.75x0.75	2.44%	Ø8c7.0cm
		2	0.75x0.75	2.44%	Ø8c7.0cm
		3	0.75x0.75	2.44%	Ø8c7.0cm
		4	0.75x0.75	2.44%	Ø8c7.0cm
		5	0.75x0.75	2.44%	Ø8c7.0cm
B2	External columns	1	0.75x0.75	1.75%	Ø8c7.0cm
		2	0.75x0.75	1.75%	Ø8c7.0cm
		3	0.75x0.75	1.75%	Ø8c7.0cm
		4	0.75x0.75	1.75%	Ø8c7.0cm
		5	0.75x0.75	1.75%	Ø8c7.0cm
	Internal columns	1	0.75x0.75	1.75%	Ø8c7.0cm
		2	0.75x0.75	1.75%	Ø8c7.0cm
		3	0.75x0.75	1.75%	Ø8c7.0cm
		4	0.75x0.75	1.75%	Ø8c7.0cm
		5	0.75x0.75	1.75%	Ø8c7.0cm
B3	External columns	1	0.75x0.75	2.44%	Ø8c7.0cm
		2	0.75x0.75	2.44%	Ø8c7.0cm
		3	0.70x0.70	2.80%	Ø8c7.0cm
		4	0.70x0.70	2.80%	Ø8c7.0cm
		5	0.65x0.65	3.25%	Ø8c7.0cm
	Internal columns	1	0.75x0.75	2.44%	Ø8c7.0cm
		2	0.75x0.75	2.44%	Ø8c7.0cm
		3	0.70x0.70	2.80%	Ø8c7.0cm
		4	0.70x0.70	2.80%	Ø8c7.0cm
		5	0.65x0.65	3.25%	Ø8c7.0cm
B4	External columns	1	0.75x0.75	2.27%	Ø8c7.0cm
		2	0.75x0.75	2.27%	Ø8c7.0cm
		3	0.75x0.75	2.27%	Ø8c7.0cm
		4	0.65x0.65	3.02%	Ø8c7.0cm
		5	0.65x0.65	3.02%	Ø8c7.0cm
	Internal columns	1	0.75x0.75	2.27%	Ø8c7.0cm
		2	0.75x0.75	2.27%	Ø8c7.0cm
		3	0.75x0.75	2.27%	Ø8c7.0cm
		4	0.65x0.65	3.02%	Ø8c7.0cm
		5	0.65x0.65	3.02%	Ø8c7.0cm

Structure	Structural element	Storey	Section (m) b x h	Long. rebar ratio (%)	Stirrups
C1	External column	1	0.75x0.75	2.44%	Ø8c7.0cm
		2	0.75x0.75	2.44%	Ø8c7.0cm
		3	0.75x0.75	2.44%	Ø8c7.0cm
		4	0.75x0.75	2.44%	Ø8c7.0cm
		5	0.75x0.75	2.44%	Ø8c7.0cm
	Internal column	1	0.75x0.75	2.44%	Ø8c7.0cm
		2	0.75x0.75	2.44%	Ø8c7.0cm
		3	0.75x0.75	2.44%	Ø8c7.0cm
		4	0.75x0.75	2.44%	Ø8c7.0cm
		5	0.75x0.75	2.44%	Ø8c7.0cm
C2	External column	1	0.75x0.75	1.75%	Ø8c7.0cm
		2	0.75x0.75	1.75%	Ø8c7.0cm
		3	0.75x0.75	1.75%	Ø8c7.0cm
		4	0.75x0.75	1.75%	Ø8c7.0cm
		5	0.75x0.75	1.75%	Ø8c7.0cm
	Internal column	1	0.75x0.75	1.75%	Ø8c7.0cm
		2	0.75x0.75	1.75%	Ø8c7.0cm
		3	0.75x0.75	1.75%	Ø8c7.0cm
		4	0.75x0.75	1.75%	Ø8c7.0cm
		5	0.75x0.75	1.75%	Ø8c7.0cm
C3	External column	1	0.75x0.75	2.44%	Ø8c7.0cm
		2	0.75x0.75	2.44%	Ø8c7.0cm
		3	0.70x0.70	2.80%	Ø8c7.0cm
		4	0.70x0.70	2.80%	Ø8c7.0cm
		5	0.65x0.65	3.25%	Ø8c7.0cm
	Internal column	1	0.75x0.75	2.44%	Ø8c7.0cm
		2	0.75x0.75	2.44%	Ø8c7.0cm
		3	0.70x0.70	2.80%	Ø8c7.0cm
		4	0.70x0.70	2.80%	Ø8c7.0cm
		5	0.65x0.65	3.25%	Ø8c7.0cm
C4	External column	1	0.75x0.75	2.44%	Ø8c7.0cm
		2	0.75x0.75	2.44%	Ø8c7.0cm
		3	0.75x0.75	2.44%	Ø8c7.0cm
		4	0.65x0.65	3.02%	Ø8c7.0cm
		5	0.65x0.65	3.25%	Ø8c7.0cm
	Internal column	1	0.75x0.75	2.44%	Ø8c7.0cm
		2	0.75x0.75	2.44%	Ø8c7.0cm
		3	0.75x0.75	2.44%	Ø8c7.0cm
		4	0.65x0.65	3.25%	Ø8c7.0cm
		5	0.65x0.65	3.25%	Ø8c7.0cm

A1.2 BEAMS

Structure	Structural element	Storey	Section (m) b x h	Long. rebar ratio (%)	Stirrups
A1	External beams	1	0.40x0.55	1.97%	Ø8c7.0cm
		2	0.40x0.55	1.97%	Ø8c7.0cm
		3	0.40x0.55	1.97%	Ø8c7.0cm
		4	0.40x0.55	1.97%	Ø8c7.0cm
		5	0.40x0.55	1.97%	Ø8c7.0cm
	Internal beams	1	0.40x0.55	2.05%	Ø8c7.0cm
		2	0.40x0.55	2.05%	Ø8c7.0cm
		3	0.40x0.55	2.05%	Ø8c7.0cm
		4	0.40x0.55	2.05%	Ø8c7.0cm
		5	0.40x0.55	2.05%	Ø8c7.0cm
A2	External beams	1	0.40x0.55	1.97%	Ø8c7.0cm
		2	0.40x0.55	1.97%	Ø8c7.0cm
		3	0.40x0.55	1.97%	Ø8c7.0cm
		4	0.40x0.55	1.97%	Ø8c7.0cm
		5	0.40x0.55	1.97%	Ø8c7.0cm
	Internal beams	1	0.40x0.55	2.05%	Ø8c7.0cm
		2	0.40x0.55	2.05%	Ø8c7.0cm
		3	0.40x0.55	2.05%	Ø8c7.0cm
		4	0.40x0.55	2.05%	Ø8c7.0cm
		5	0.40x0.55	2.05%	Ø8c7.0cm
A3	External beams	1	0.40x0.55	1.44%	Ø8c7.0cm
		2	0.40x0.55	1.44%	Ø8c7.0cm
		3	0.40x0.50	1.07%	Ø8c7.0cm
		4	0.40x0.50	1.07%	Ø8c7.0cm
		5	0.35x0.45	0.80%	Ø8c7.0cm
	Internal beams	1	0.40x0.55	1.57%	Ø8c7.0cm
		2	0.40x0.55	1.57%	Ø8c7.0cm
		3	0.40x0.50	1.34%	Ø8c7.0cm
		4	0.40x0.50	1.34%	Ø8c7.0cm
		5	0.35x0.45	1.20%	Ø8c7.0cm
A4	External beams	1	0.40x0.55	1.41%	Ø8c7.0cm
		2	0.40x0.55	1.41%	Ø8c7.0cm
		3	0.40x0.55	1.41%	Ø8c7.0cm
		4	0.40x0.55	1.41%	Ø8c7.0cm
		5	0.40x0.55	1.41%	Ø8c7.0cm
	Internal beams	1	0.40x0.55	1.57%	Ø8c7.0cm
		2	0.40x0.55	1.57%	Ø8c7.0cm
		3	0.40x0.55	1.57%	Ø8c7.0cm
		4	0.40x0.55	1.57%	Ø8c7.0cm
		5	0.40x0.55	1.57%	Ø8c7.0cm

Structure	Structural element	Storey	Section (m) b x h	Long. rebar ratio (%)	Stirrups
A5	External beams	1	0.40x0.55	1.41%	Ø8c7.0cm
		2	0.40x0.55	1.41%	Ø8c7.0cm
		3	0.40x0.55	1.41%	Ø8c7.0cm
		4	0.40x0.55	1.41%	Ø8c7.0cm
		5	0.40x0.55	1.41%	Ø8c7.0cm
	Internal beams	1	0.40x0.55	1.57%	Ø8c7.0cm
		2	0.40x0.55	1.57%	Ø8c7.0cm
		3	0.40x0.55	1.57%	Ø8c7.0cm
		4	0.40x0.55	1.57%	Ø8c7.0cm
		5	0.40x0.55	1.57%	Ø8c7.0cm
B1	External beams	1	0.45x0.65	2.85%	Ø10c7.0cm
		2	0.45x0.65	2.85%	Ø10c7.0cm
		3	0.45x0.65	2.85%	Ø10c7.0cm
		4	0.45x0.65	2.85%	Ø10c7.0cm
		5	0.45x0.65	2.85%	Ø10c7.0cm
	Internal beams	1	0.45x0.65	2.52%	Ø10c7.0cm
		2	0.45x0.65	2.52%	Ø10c7.0cm
		3	0.45x0.65	2.52%	Ø10c7.0cm
		4	0.45x0.65	2.52%	Ø10c7.0cm
		5	0.45x0.65	2.52%	Ø10c7.0cm
B2	External beams	1	0.45x0.65	2.85%	Ø10c7.0cm
		2	0.45x0.65	2.85%	Ø10c7.0cm
		3	0.45x0.65	2.85%	Ø10c7.0cm
		4	0.45x0.65	2.85%	Ø10c7.0cm
		5	0.45x0.65	2.85%	Ø10c7.0cm
	Internal beams	1	0.45x0.65	2.35%	Ø10c7.0cm
		2	0.45x0.65	2.35%	Ø10c7.0cm
		3	0.45x0.65	2.35%	Ø10c7.0cm
		4	0.45x0.65	2.35%	Ø10c7.0cm
		5	0.45x0.65	2.35%	Ø10c7.0cm
B3	External beams	1	0.45x0.65	2.85%	Ø10c7.0cm
		2	0.45x0.65	2.85%	Ø10c7.0cm
		3	0.45x0.65	2.85%	Ø10c7.0cm
		4	0.45x0.65	2.85%	Ø10c7.0cm
		5	0.45x0.65	2.85%	Ø10c7.0cm
	Internal beams	1	0.45x0.65	2.35%	Ø10c7.0cm
		2	0.45x0.65	2.35%	Ø10c7.0cm
		3	0.45x0.65	2.35%	Ø10c7.0cm
		4	0.45x0.65	2.35%	Ø10c7.0cm
		5	0.45x0.65	2.35%	Ø10c7.0cm
B4	External beams	1	0.45x0.65	2.85%	Ø10c7.0cm
		2	0.45x0.65	2.85%	Ø10c7.0cm
		3	0.45x0.65	2.85%	Ø10c7.0cm
		4	0.45x0.65	2.85%	Ø10c7.0cm
		5	0.45x0.65	2.85%	Ø10c7.0cm
	Internal beams	1	0.45x0.65	2.35%	Ø10c7.0cm
		2	0.45x0.65	2.35%	Ø10c7.0cm
		3	0.45x0.65	2.35%	Ø10c7.0cm
		4	0.45x0.65	2.35%	Ø10c7.0cm
		5	0.45x0.65	2.35%	Ø10c7.0cm

Structure	Structural element	Storey	Section (m) b x h	Long. rebar ratio (%)	Stirrups
C1	External beams	1	0.40x0.55	3.19%	Ø10c7.0cm
		2	0.40x0.55	3.19%	Ø10c7.0cm
		3	0.40x0.55	3.19%	Ø10c7.0cm
		4	0.40x0.55	3.19%	Ø10c7.0cm
		5	0.40x0.55	3.19%	Ø10c7.0cm
	Internal beams	1	0.40x0.55	2.69%	Ø10c7.0cm
		2	0.40x0.55	2.69%	Ø10c7.0cm
		3	0.40x0.55	2.69%	Ø10c7.0cm
		4	0.40x0.55	2.69%	Ø10c7.0cm
		5	0.40x0.55	2.69%	Ø10c7.0cm
C2	External beams	1	0.45x0.65	3.02%	Ø10c7.0cm
		2	0.45x0.65	3.02%	Ø10c7.0cm
		3	0.45x0.65	3.02%	Ø10c7.0cm
		4	0.45x0.65	3.02%	Ø10c7.0cm
		5	0.45x0.65	3.02%	Ø10c7.0cm
	Internal beams	1	0.45x0.65	2.69%	Ø10c7.0cm
		2	0.45x0.65	2.69%	Ø10c7.0cm
		3	0.45x0.65	2.69%	Ø10c7.0cm
		4	0.45x0.65	2.69%	Ø10c7.0cm
		5	0.45x0.65	2.69%	Ø10c7.0cm
C3	External beams	1	0.45x0.65	3.02%	Ø10c7.0cm
		2	0.45x0.65	3.02%	Ø10c7.0cm
		3	0.45x0.65	3.02%	Ø10c7.0cm
		4	0.45x0.65	3.02%	Ø10c7.0cm
		5	0.45x0.65	3.02%	Ø10c7.0cm
	Internal beams	1	0.45x0.65	2.69%	Ø10c7.0cm
		2	0.45x0.65	2.69%	Ø10c7.0cm
		3	0.45x0.65	2.69%	Ø10c7.0cm
		4	0.45x0.65	2.69%	Ø10c7.0cm
		5	0.45x0.65	2.69%	Ø10c7.0cm
C4	External beams	1	0.45x0.65	3.02%	Ø10c7.0cm
		2	0.45x0.65	3.02%	Ø10c7.0cm
		3	0.45x0.65	3.02%	Ø10c7.0cm
		4	0.45x0.65	3.02%	Ø10c7.0cm
		5	0.45x0.65	3.02%	Ø10c7.0cm
	Internal beams	1	0.45x0.65	2.69%	Ø10c7.0cm
		2	0.45x0.65	2.69%	Ø10c7.0cm
		3	0.45x0.65	2.69%	Ø10c7.0cm
		4	0.45x0.65	2.69%	Ø10c7.0cm
		5	0.45x0.65	2.69%	Ø10c7.0cm

A.2 Moment resisting frames and shear walls – Chapters 4 and 5 structures

A2.1 Columns

Structure	Structural element	Storey	Section (m) b x h	Long. rebar ratio (%)	Stirrups
FR5SH3.0q3.9 FR5SH3.5q3.9 FR5SH4.0q3.12 FR5SH4.5q3.12 FR5SH5.0q3.12 FR5SH5.5q3.12	External columns	1	0.55x0.55	1.06%	Ø8c15.0cm
		2	0.55x0.55	1.06%	Ø8c15.0cm
		3	0.50x0.50	0.97%	Ø8c15.0cm
		4	0.50x0.50	0.97%	Ø8c15.0cm
		5	0.45x0.45	1.19%	Ø8c15.0cm
	Internal columns	1	0.55x0.55	1.06%	Ø8c15.0cm
		2	0.55x0.55	1.06%	Ø8c15.0cm
		3	0.50x0.50	0.97%	Ø8c15.0cm
		4	0.50x0.50	0.97%	Ø8c15.0cm
		5	0.45x0.45	1.19%	Ø8c15.0cm
FR10SH3.0q3.12 FR10SH3.5q3.9 FR10SH4.0q3.9 FR10SH4.5q3.12 FR10SH5.0q3.12 FR10SH5.5q3.12	External columns	1	0.60x0.60	1.05%	Ø8c15.0cm
		2	0.55x0.55	1.25%	Ø8c15.0cm
		3	0.55x0.55	1.25%	Ø8c15.0cm
		4	0.55x0.55	1.25%	Ø8c15.0cm
		5	0.50x0.50	1.51%	Ø8c15.0cm
		6	0.50x0.50	1.51%	Ø8c15.0cm
		7	0.50x0.50	1.51%	Ø8c15.0cm
		8	0.50x0.50	1.51%	Ø8c15.0cm
		9	0.45x0.45	1.86%	Ø8c15.0cm
		10	0.45x0.45	1.86%	Ø8c15.0cm
	Internal columns	1	0.60x0.60	1.05%	Ø8c15.0cm
		2	0.55x0.55	1.25%	Ø8c15.0cm
		3	0.55x0.55	1.25%	Ø8c15.0cm
		4	0.55x0.55	1.25%	Ø8c15.0cm
		5	0.50x0.50	1.51%	Ø8c15.0cm
		6	0.50x0.50	1.51%	Ø8c15.0cm
		7	0.50x0.50	1.51%	Ø8c15.0cm
		8	0.50x0.50	1.51%	Ø8c15.0cm
		9	0.45x0.45	1.86%	Ø8c15.0cm
		10	0.45x0.45	1.86%	Ø8c15.0cm
WA5SH3.0q3.0 WA5SH3.5q3.0 WA5SH4.0q3.0 WA5SH4.5q2.4 WA5SH5.0q2.4 WA5SH5.5q2.4	External columns	1	0.55x0.55	1.25%	Ø8c15.0cm
		2	0.55x0.55	1.25%	Ø8c15.0cm
		3	0.50x0.50	1.51%	Ø8c15.0cm
		4	0.50x0.50	1.51%	Ø8c15.0cm
		5	0.45x0.45	1.86%	Ø8c15.0cm
	Internal columns	1	0.55x0.55	1.25%	Ø8c15.0cm
		2	0.55x0.55	1.25%	Ø8c15.0cm
		3	0.50x0.50	1.51%	Ø8c15.0cm
		4	0.50x0.50	1.51%	Ø8c15.0cm
		5	0.45x0.45	1.86%	Ø8c15.0cm

Structure	Structural element	Storey	Section (m) b x h	Long. rebar ratio (%)	Stirrups
WA10SH3.0q3.0 WA10SH3.5q3.0 WA10SH4.0q2.4 WA10SH4.5q2.4 WA10SH5.0q2.4 WA10SH5.5q2.4	External columns	1	0.60x0.60	1.05%	Ø10c15.0cm
		2	0.55x0.55	1.25%	Ø10c15.0cm
		3	0.55x0.55	1.25%	Ø10c15.0cm
		4	0.55x0.55	1.25%	Ø10c15.0cm
		5	0.50x0.50	1.51%	Ø10c15.0cm
		6	0.50x0.50	1.51%	Ø10c15.0cm
		7	0.50x0.50	1.51%	Ø10c15.0cm
		8	0.50x0.50	1.51%	Ø10c15.0cm
		9	0.45x0.45	1.86%	Ø10c15.0cm
		10	0.45x0.45	1.86%	Ø10c15.0cm
	Internal columns	1	0.60x0.60	1.05%	Ø10c15.0cm
		2	0.55x0.55	1.25%	Ø10c15.0cm
		3	0.55x0.55	1.25%	Ø10c15.0cm
		4	0.55x0.55	1.25%	Ø10c15.0cm
		5	0.50x0.50	1.51%	Ø10c15.0cm
		6	0.50x0.50	1.51%	Ø10c15.0cm
		7	0.50x0.50	1.51%	Ø10c15.0cm
		8	0.50x0.50	1.51%	Ø10c15.0cm
		9	0.45x0.45	1.86%	Ø10c15.0cm
		10	0.45x0.45	1.86%	Ø10c15.0cm
WA15SH3.0q3.0 WA15SH3.5q3.0 WA15SH4.0q2.4 WA15SH4.5q2.4 WA15SH5.0q2.4 WA15SH5.5q2.4	External columns	1	0.75x0.75	0.67%	Ø10c15.0cm
		2	0.70x0.70	0.77%	Ø10c15.0cm
		3	0.70x0.70	0.77%	Ø10c15.0cm
		4	0.65x0.65	0.89%	Ø10c15.0cm
		5	0.65x0.65	0.89%	Ø10c15.0cm
		6	0.60x0.60	1.05%	Ø10c15.0cm
		7	0.60x0.60	1.05%	Ø10c15.0cm
		8	0.55x0.55	1.25%	Ø10c15.0cm
		9	0.55x0.55	1.25%	Ø10c15.0cm
		10	0.50x0.50	1.51%	Ø10c15.0cm
		11	0.50x0.50	1.51%	Ø10c15.0cm
		12	0.50x0.50	1.51%	Ø10c15.0cm
		13	0.50x0.50	1.51%	Ø10c15.0cm
		14	0.45x0.45	1.86%	Ø10c15.0cm
		15	0.45x0.45	1.86%	Ø10c15.0cm
	Internal columns	1	0.75x0.75	0.67%	Ø10c15.0cm
		2	0.70x0.70	0.77%	Ø10c15.0cm
		3	0.70x0.70	0.77%	Ø10c15.0cm
		4	0.65x0.65	0.89%	Ø10c15.0cm
		5	0.65x0.65	0.89%	Ø10c15.0cm
		6	0.60x0.60	1.05%	Ø10c15.0cm
		7	0.60x0.60	1.05%	Ø10c15.0cm
		8	0.55x0.55	1.25%	Ø10c15.0cm
		9	0.55x0.55	1.25%	Ø10c15.0cm
		10	0.50x0.50	1.51%	Ø10c15.0cm
		11	0.50x0.50	1.51%	Ø10c15.0cm
		12	0.50x0.50	1.51%	Ø10c15.0cm
		13	0.50x0.50	1.51%	Ø10c15.0cm
		14	0.45x0.45	1.86%	Ø10c15.0cm
		15	0.45x0.45	1.86%	Ø10c15.0cm

Structure	Structural element	Storey	Section (m) b x h	Long. rebar ratio (%)	Stirrups
WA20SH3.0q3.0 WA20SH3.5q2.4 WA20SH4.0q2.4 WA20SH4.5q2.4 WA20SH5.0q2.4 WA20SH5.5q2.4	External columns	1	0.85x0.85	0.52%	Ø10c15.0cm
		2	0.85x0.85	0.52%	Ø10c15.0cm
		3	0.80x0.80	0.59%	Ø10c15.0cm
		4	0.80x0.80	0.59%	Ø10c15.0cm
		5	0.75x0.75	0.67%	Ø10c15.0cm
		6	0.75x0.75	0.67%	Ø10c15.0cm
		7	0.70x0.70	0.77%	Ø10c15.0cm
		8	0.70x0.70	0.77%	Ø10c15.0cm
		9	0.65x0.65	0.89%	Ø10c15.0cm
		10	0.65x0.65	0.89%	Ø10c15.0cm
		11	0.60x0.60	1.05%	Ø10c15.0cm
		12	0.60x0.60	1.05%	Ø10c15.0cm
		13	0.55x0.55	1.25%	Ø10c15.0cm
		14	0.55x0.55	1.25%	Ø10c15.0cm
		15	0.50x0.50	1.51%	Ø10c15.0cm
		16	0.50x0.50	1.51%	Ø10c15.0cm
		17	0.45x0.45	1.86%	Ø10c15.0cm
		18	0.45x0.45	1.86%	Ø10c15.0cm
		19	0.45x0.45	1.86%	Ø10c15.0cm
		20	0.45x0.45	1.86%	Ø10c15.0cm
	Internal columns	1	0.85x0.85	0.52%	Ø10c15.0cm
		2	0.85x0.85	0.52%	Ø10c15.0cm
		3	0.80x0.80	0.59%	Ø10c15.0cm
		4	0.80x0.80	0.59%	Ø10c15.0cm
		5	0.75x0.75	0.67%	Ø10c15.0cm
		6	0.75x0.75	0.67%	Ø10c15.0cm
		7	0.70x0.70	0.77%	Ø10c15.0cm
		8	0.70x0.70	0.77%	Ø10c15.0cm
		9	0.65x0.65	0.89%	Ø10c15.0cm
		10	0.65x0.65	0.89%	Ø10c15.0cm
		11	0.60x0.60	1.05%	Ø10c15.0cm
		12	0.60x0.60	1.05%	Ø10c15.0cm
		13	0.55x0.55	1.25%	Ø10c15.0cm
		14	0.55x0.55	1.25%	Ø10c15.0cm
		15	0.50x0.50	1.51%	Ø10c15.0cm
		16	0.50x0.50	1.51%	Ø10c15.0cm
		17	0.45x0.45	1.86%	Ø10c15.0cm
		18	0.45x0.45	1.86%	Ø10c15.0cm
		19	0.45x0.45	1.86%	Ø10c15.0cm
		20	0.45x0.45	1.86%	Ø10c15.0cm

A2.2 SHEAR WALLS

Structure	Structural element	Section (m) b x h	Corners Long. rebar ratio (%)	Corners Stirrups	Middle Long. rebar ratio (%)	Middle Stirrups
WA5SH3.0q3.0 WA5SH3.5q3.0 WA5SH4.0q3.0 WA5SH4.5q2.4 WA5SH5.0q2.4 WA5SH5.5q2.4 WA10SH3.0q3.0 WA10SH3.5q3.0 WA10SH4.0q2.4 WA10SH4.5q2.4 WA10SH5.0q2.4 WA10SH5.5q2.4 WA15SH3.0q3.0 WA15SH3.5q3.0 WA15SH4.0q2.4 WA15SH4.5q2.4 WA15SH5.0q2.4 WA15SH5.5q2.4 WA20SH3.0q3.0 WA20SH3.5q2.4 WA20SH4.0q2.4 WA20SH4.5q2.4 WA20SH5.0q2.4 WA20SH5.5q2.4	Shear walls	4.00x0.30	1.26%	Ø10c15.0cm	0.26%	Ø10c30.0cm

A2.3 BEAMS

Structure	Structural element	Storey	Section (m) b x h	Long. rebar ratio (%)	Stirrups
FR5SH3.0q3.9 FR5SH3.5q3.9	External beams	1	0.40x0.55	1.29%	Ø8c12.0cm
		2	0.40x0.55	1.29%	Ø8c12.0cm
		3	0.40x0.50	1.41%	Ø8c12.0cm
		4	0.40x0.50	1.26%	Ø8c12.0cm
		5	0.35x0.45	1.60%	Ø8c11.0cm
	Internal beams	1	0.40x0.55	1.29%	Ø8c12.0cm
		2	0.40x0.55	1.29%	Ø8c12.0cm
		3	0.40x0.50	1.41%	Ø8c12.0cm
		4	0.40x0.50	1.26%	Ø8c12.0cm
		5	0.35x0.45	1.60%	Ø8c11.0cm
FR5SH4.0q3.12 FR5SH4.5q3.12	External beams	1	0.40x0.55	1.57%	Ø8c12.0cm
		2	0.40x0.55	1.43%	Ø8c12.0cm
		3	0.40x0.50	1.26%	Ø8c12.0cm
		4	0.40x0.50	1.26%	Ø8c12.0cm
		5	0.35x0.45	1.60%	Ø8c11.0cm
	Internal beams	1	0.40x0.55	1.57%	Ø8c12.0cm
		2	0.40x0.55	1.43%	Ø8c12.0cm
		3	0.40x0.50	1.41%	Ø8c12.0cm
		4	0.40x0.50	1.26%	Ø8c12.0cm
		5	0.35x0.45	1.60%	Ø8c11.0cm
FR5SH5.0q3.12 FR5SH5.5q3.12	External beams	1	0.40x0.55	1.57%	Ø8c12.0cm
		2	0.40x0.55	1.29%	Ø8c12.0cm
		3	0.40x0.50	1.26%	Ø8c12.0cm
		4	0.40x0.50	1.26%	Ø8c12.0cm
		5	0.35x0.45	1.60%	Ø8c11.0cm
	Internal beams	1	0.40x0.55	1.57%	Ø8c12.0cm
		2	0.40x0.55	1.29%	Ø8c12.0cm
		3	0.40x0.50	1.41%	Ø8c12.0cm
		4	0.40x0.50	1.26%	Ø8c12.0cm
		5	0.35x0.45	1.60%	Ø8c11.0cm
FR10SH3.0q3.12 FR10SH3.5q3.9 FR10SH4.0q3.9 FR10SH4.5q3.12	External beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.14%	Ø8c12.0cm
		3	0.40x0.55	1.14%	Ø8c12.0cm
		4	0.40x0.55	1.14%	Ø8c12.0cm
		5	0.40x0.50	1.26%	Ø8c12.0cm
		6	0.40x0.50	1.26%	Ø8c12.0cm
		7	0.40x0.50	1.26%	Ø8c12.0cm
		8	0.40x0.50	1.26%	Ø8c12.0cm
		9	0.35x0.45	1.60%	Ø8c11.0cm
		10	0.35x0.45	1.60%	Ø8c11.0cm
	Internal beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.14%	Ø8c12.0cm
		3	0.40x0.55	1.14%	Ø8c12.0cm
		4	0.40x0.55	1.14%	Ø8c12.0cm
		5	0.40x0.50	1.26%	Ø8c12.0cm
		6	0.40x0.50	1.26%	Ø8c12.0cm
		7	0.40x0.50	1.41%	Ø8c12.0cm
		8	0.40x0.50	1.26%	Ø8c12.0cm
		9	0.35x0.45	1.80%	Ø8c11.0cm
		10	0.35x0.45	1.60%	Ø8c11.0cm

Structure	Structural element	Storey	Section (m) b x h	Long. rebar ratio (%)	Stirrups
FR10SH5.0q3.12 FR10SH5.5q3.12	External beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.14%	Ø8c12.0cm
		3	0.40x0.55	1.14%	Ø8c12.0cm
		4	0.40x0.55	1.14%	Ø8c12.0cm
		5	0.40x0.50	1.26%	Ø8c12.0cm
		6	0.40x0.50	1.26%	Ø8c12.0cm
		7	0.40x0.50	1.26%	Ø8c12.0cm
		8	0.40x0.50	1.26%	Ø8c12.0cm
		9	0.35x0.45	1.60%	Ø8c11.0cm
		10	0.35x0.45	1.60%	Ø8c11.0cm
	Internal beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.14%	Ø8c12.0cm
		3	0.40x0.55	1.14%	Ø8c12.0cm
		4	0.40x0.55	1.14%	Ø8c12.0cm
		5	0.40x0.50	1.26%	Ø8c12.0cm
		6	0.40x0.50	1.41%	Ø8c12.0cm
		7	0.40x0.50	1.41%	Ø8c12.0cm
		8	0.40x0.50	1.26%	Ø8c12.0cm
		9	0.35x0.45	1.80%	Ø8c11.0cm
		10	0.35x0.45	1.60%	Ø8c11.0cm
WA5SH3.0q3.0	External beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.57%	Ø8c12.0cm
		3	0.40x0.50	1.73%	Ø8c12.0cm
		4	0.40x0.50	1.73%	Ø8c12.0cm
		5	0.35x0.45	1.60%	Ø8c11.0cm
	Internal beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.29%	Ø8c12.0cm
		3	0.40x0.50	1.41%	Ø8c12.0cm
		4	0.40x0.50	1.41%	Ø8c12.0cm
		5	0.35x0.45	1.60%	Ø8c11.0cm
WA5SH3.5q3.0	External beams	1	0.40x0.55	1.29%	Ø8c12.0cm
		2	0.40x0.55	1.57%	Ø8c12.0cm
		3	0.40x0.50	1.73%	Ø8c12.0cm
		4	0.40x0.50	1.57%	Ø8c12.0cm
		5	0.35x0.45	1.60%	Ø8c11.0cm
	Internal beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.29%	Ø8c12.0cm
		3	0.40x0.50	1.41%	Ø8c12.0cm
		4	0.40x0.50	1.41%	Ø8c12.0cm
		5	0.35x0.45	1.60%	Ø8c11.0cm
WA5SH4.0q3.0	External beams	1	0.40x0.55	1.29%	Ø8c12.0cm
		2	0.40x0.55	1.71%	Ø8c12.0cm
		3	0.40x0.50	1.73%	Ø8c12.0cm
		4	0.40x0.50	1.57%	Ø8c12.0cm
		5	0.35x0.45	1.60%	Ø8c11.0cm
	Internal beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.29%	Ø8c12.0cm
		3	0.40x0.50	1.41%	Ø8c12.0cm
		4	0.40x0.50	1.41%	Ø8c12.0cm
		5	0.35x0.45	1.60%	Ø8c11.0cm

Structure	Structural element	Storey	Section (m) b x h	Long. rebar ratio (%)	Stirrups
WA5SH4.5q2.4	External beams	1	0.40x0.55	1.71%	Ø8c12.0cm
		2	0.40x0.55	2.14%	Ø8c12.0cm
		3	0.40x0.50	2.04%	Ø8c12.0cm
		4	0.40x0.50	1.88%	Ø8c12.0cm
		5	0.35x0.45	1.60%	Ø8c11.0cm
	Internal beams	1	0.40x0.55	1.29%	Ø8c12.0cm
		2	0.40x0.55	1.29%	Ø8c12.0cm
		3	0.40x0.50	1.41%	Ø8c12.0cm
		4	0.40x0.50	1.41%	Ø8c12.0cm
		5	0.35x0.45	1.60%	Ø8c11.0cm
WA5SH5.0q2.4	External beams	1	0.40x0.55	1.86%	Ø8c12.0cm
		2	0.40x0.55	2.14%	Ø8c12.0cm
		3	0.40x0.50	2.04%	Ø8c12.0cm
		4	0.40x0.50	1.88%	Ø8c12.0cm
		5	0.35x0.45	1.60%	Ø8c11.0cm
	Internal beams	1	0.40x0.55	1.29%	Ø8c12.0cm
		2	0.40x0.55	1.29%	Ø8c12.0cm
		3	0.40x0.50	1.41%	Ø8c12.0cm
		4	0.40x0.50	1.41%	Ø8c12.0cm
		5	0.35x0.45	1.60%	Ø8c11.0cm
WA5SH5.5q2.4	External beams	1	0.40x0.55	1.57%	Ø8c12.0cm
		2	0.40x0.55	2.14%	Ø8c12.0cm
		3	0.40x0.50	2.20%	Ø8c12.0cm
		4	0.40x0.50	2.04%	Ø8c12.0cm
		5	0.35x0.45	1.60%	Ø8c11.0cm
	Internal beams	1	0.40x0.55	1.29%	Ø8c12.0cm
		2	0.40x0.55	1.29%	Ø8c12.0cm
		3	0.40x0.50	1.41%	Ø8c12.0cm
		4	0.40x0.50	1.41%	Ø8c12.0cm
		5	0.35x0.45	1.60%	Ø8c11.0cm
WA10SH3.0q3.0 WA10SH3.5q3.0	External beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.14%	Ø8c12.0cm
		3	0.40x0.55	1.14%	Ø8c12.0cm
		4	0.40x0.55	1.14%	Ø8c12.0cm
		5	0.40x0.50	1.26%	Ø8c12.0cm
		6	0.40x0.50	1.26%	Ø8c12.0cm
		7	0.40x0.50	1.26%	Ø8c12.0cm
		8	0.40x0.50	1.26%	Ø8c12.0cm
		9	0.35x0.45	1.60%	Ø8c11.0cm
		10	0.35x0.45	1.60%	Ø8c11.0cm
	Internal beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.14%	Ø8c12.0cm
		3	0.40x0.55	1.14%	Ø8c12.0cm
		4	0.40x0.55	1.14%	Ø8c12.0cm
		5	0.40x0.50	1.26%	Ø8c12.0cm
		6	0.40x0.50	1.26%	Ø8c12.0cm
		7	0.40x0.50	1.26%	Ø8c12.0cm
		8	0.40x0.50	1.26%	Ø8c12.0cm
		9	0.35x0.45	1.60%	Ø8c11.0cm
		10	0.35x0.45	1.60%	Ø8c11.0cm

Structure	Structural element	Storey	Section (m) b x h	Long. rebar ratio (%)	Stirrups
WA10SH4.0q2.4	External beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.14%	Ø8c12.0cm
		3	0.40x0.55	1.14%	Ø8c12.0cm
		4	0.40x0.55	1.14%	Ø8c12.0cm
		5	0.40x0.50	1.26%	Ø8c12.0cm
		6	0.40x0.50	1.26%	Ø8c12.0cm
		7	0.40x0.50	1.26%	Ø8c12.0cm
		8	0.40x0.50	1.26%	Ø8c12.0cm
		9	0.35x0.45	1.60%	Ø8c11.0cm
		10	0.35x0.45	1.60%	Ø8c11.0cm
	Internal beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.14%	Ø8c12.0cm
		3	0.40x0.55	1.14%	Ø8c12.0cm
		4	0.40x0.55	1.14%	Ø8c12.0cm
		5	0.40x0.50	1.26%	Ø8c12.0cm
		6	0.40x0.50	1.26%	Ø8c12.0cm
		7	0.40x0.50	1.41%	Ø8c12.0cm
		8	0.40x0.50	1.26%	Ø8c12.0cm
		9	0.35x0.45	1.60%	Ø8c11.0cm
		10	0.35x0.45	1.60%	Ø8c11.0cm
WA10SH4.5q2.4 WA10SH5.0q2.4 WA10SH5.5q2.4	External beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.14%	Ø8c12.0cm
		3	0.40x0.55	1.14%	Ø8c12.0cm
		4	0.40x0.55	1.14%	Ø8c12.0cm
		5	0.40x0.50	1.26%	Ø8c12.0cm
		6	0.40x0.50	1.26%	Ø8c12.0cm
		7	0.40x0.50	1.26%	Ø8c12.0cm
		8	0.40x0.50	1.26%	Ø8c12.0cm
		9	0.35x0.45	1.60%	Ø8c11.0cm
		10	0.35x0.45	1.60%	Ø8c11.0cm
	Internal beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.14%	Ø8c12.0cm
		3	0.40x0.55	1.14%	Ø8c12.0cm
		4	0.40x0.55	1.14%	Ø8c12.0cm
		5	0.40x0.50	1.26%	Ø8c12.0cm
		6	0.40x0.50	1.26%	Ø8c12.0cm
		7	0.40x0.50	1.41%	Ø8c12.0cm
		8	0.40x0.50	1.26%	Ø8c12.0cm
		9	0.35x0.45	1.80%	Ø8c11.0cm
		10	0.35x0.45	1.60%	Ø8c11.0cm

Structure	Structural element	Storey	Section (m) b x h	Long. rebar ratio (%)	Stirrups
WA15SH3.0q3.0 WA15SH3.5q3.5	External beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.14%	Ø8c12.0cm
		3	0.40x0.55	1.14%	Ø8c12.0cm
		4	0.40x0.55	1.14%	Ø8c12.0cm
		5	0.40x0.55	1.14%	Ø8c12.0cm
		6	0.40x0.55	1.14%	Ø8c12.0cm
		7	0.40x0.55	1.14%	Ø8c12.0cm
		8	0.40x0.55	1.14%	Ø8c12.0cm
		9	0.40x0.55	1.14%	Ø8c12.0cm
		10	0.40x0.50	1.26%	Ø8c12.0cm
		11	0.40x0.50	1.26%	Ø8c12.0cm
		12	0.40x0.50	1.26%	Ø8c12.0cm
		13	0.40x0.50	1.26%	Ø8c12.0cm
		14	0.35x0.45	1.60%	Ø8c11.0cm
		15	0.35x0.45	1.60%	Ø8c11.0cm
	Internal beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.14%	Ø8c12.0cm
		3	0.40x0.55	1.14%	Ø8c12.0cm
		4	0.40x0.55	1.14%	Ø8c12.0cm
		5	0.40x0.55	1.14%	Ø8c12.0cm
		6	0.40x0.55	1.14%	Ø8c12.0cm
		7	0.40x0.55	1.14%	Ø8c12.0cm
		8	0.40x0.55	1.14%	Ø8c12.0cm
		9	0.40x0.55	1.14%	Ø8c12.0cm
		10	0.40x0.50	1.26%	Ø8c12.0cm
		11	0.40x0.50	1.26%	Ø8c12.0cm
		12	0.40x0.50	1.26%	Ø8c12.0cm
		13	0.40x0.50	1.26%	Ø8c12.0cm
		14	0.35x0.45	1.60%	Ø8c11.0cm
		15	0.35x0.45	1.60%	Ø8c11.0cm
WA15SH4.0q2.4 WA15SH4.5q2.4 WA15SH5.0q2.4 WA15SH5.5q2.4	External beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.14%	Ø8c12.0cm
		3	0.40x0.55	1.14%	Ø8c12.0cm
		4	0.40x0.55	1.14%	Ø8c12.0cm
		5	0.40x0.55	1.14%	Ø8c12.0cm
		6	0.40x0.55	1.14%	Ø8c12.0cm
		7	0.40x0.55	1.14%	Ø8c12.0cm
		8	0.40x0.55	1.14%	Ø8c12.0cm
		9	0.40x0.55	1.14%	Ø8c12.0cm
		10	0.40x0.50	1.26%	Ø8c12.0cm
		11	0.40x0.50	1.26%	Ø8c12.0cm
		12	0.40x0.50	1.26%	Ø8c12.0cm
		13	0.40x0.50	1.26%	Ø8c12.0cm
		14	0.35x0.45	1.60%	Ø8c11.0cm
		15	0.35x0.45	1.60%	Ø8c11.0cm

Structure	Structural element	Storey	Section (m) b x h	Long. rebar ratio (%)	Stirrups
WA15SH4.0q2.4 WA15SH4.5q2.4 WA15SH5.0q2.4 WA15SH5.5q2.4	Internal beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.14%	Ø8c12.0cm
		3	0.40x0.55	1.14%	Ø8c12.0cm
		4	0.40x0.55	1.14%	Ø8c12.0cm
		5	0.40x0.55	1.14%	Ø8c12.0cm
		6	0.40x0.55	1.29%	Ø8c12.0cm
		7	0.40x0.55	1.29%	Ø8c12.0cm
		8	0.40x0.55	1.29%	Ø8c12.0cm
		9	0.40x0.55	1.29%	Ø8c12.0cm
		10	0.40x0.50	1.41%	Ø8c12.0cm
		11	0.40x0.50	1.41%	Ø8c12.0cm
		12	0.40x0.50	1.41%	Ø8c12.0cm
		13	0.40x0.50	1.41%	Ø8c12.0cm
		14	0.35x0.45	1.80%	Ø8c11.0cm
		15	0.35x0.45	1.60%	Ø8c11.0cm
WA20SH3.0q3.0 WA20SH3.5q2.4 WA20SH4.0q2.4 WA20SH4.5q2.4 WA20SH5.0q2.4 WA20SH5.5q2.4	External beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.14%	Ø8c12.0cm
		3	0.40x0.55	1.14%	Ø8c12.0cm
		4	0.40x0.55	1.14%	Ø8c12.0cm
		5	0.40x0.55	1.14%	Ø8c12.0cm
		6	0.40x0.55	1.14%	Ø8c12.0cm
		7	0.40x0.55	1.14%	Ø8c12.0cm
		8	0.40x0.55	1.14%	Ø8c12.0cm
		9	0.40x0.55	1.14%	Ø8c12.0cm
		10	0.40x0.55	1.14%	Ø8c12.0cm
		11	0.40x0.55	1.14%	Ø8c12.0cm
		12	0.40x0.55	1.14%	Ø8c12.0cm
		13	0.40x0.55	1.14%	Ø8c12.0cm
		14	0.40x0.50	1.26%	Ø8c12.0cm
		15	0.40x0.50	1.26%	Ø8c12.0cm
		16	0.35x0.45	1.60%	Ø8c12.0cm
		17	0.35x0.45	1.60%	Ø8c12.0cm
		18	0.35x0.45	1.60%	Ø8c12.0cm
		19	0.35x0.45	1.60%	Ø8c11.0cm
		20	0.35x0.45	1.60%	Ø8c11.0cm

Structure	Structural element	Storey	Section (m) b x h	Long. rebar ratio (%)	Stirrups
WA20SH3.0q3.0 WA20SH3.5q2.4 WA20SH4.0q2.4 WA20SH4.5q2.4 WA20SH5.0q2.4 WA20SH5.5q2.4	Internal beams	1	0.40x0.55	1.14%	Ø8c12.0cm
		2	0.40x0.55	1.14%	Ø8c12.0cm
		3	0.40x0.55	1.14%	Ø8c12.0cm
		4	0.40x0.55	1.14%	Ø8c12.0cm
		5	0.40x0.55	1.14%	Ø8c12.0cm
		6	0.40x0.55	1.14%	Ø8c12.0cm
		7	0.40x0.55	1.29%	Ø8c12.0cm
		8	0.40x0.55	1.29%	Ø8c12.0cm
		9	0.40x0.55	1.29%	Ø8c12.0cm
		10	0.40x0.55	1.29%	Ø8c12.0cm
		11	0.40x0.55	1.29%	Ø8c12.0cm
		12	0.40x0.55	1.29%	Ø8c12.0cm
		13	0.40x0.55	1.29%	Ø8c12.0cm
		14	0.40x0.50	1.41%	Ø8c12.0cm
		15	0.40x0.50	1.41%	Ø8c12.0cm
		16	0.35x0.45	1.80%	Ø8c12.0cm
		17	0.35x0.45	1.80%	Ø8c12.0cm
		18	0.35x0.45	1.80%	Ø8c12.0cm
		19	0.35x0.45	1.60%	Ø8c11.0cm
		20	0.35x0.45	1.60%	Ø8c11.0cm