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**CONCRETE MASONRY STRUCTURES: NUMERICAL MODELING,
SEISMIC PERFORMANCE, AND BUILDING APPLICATIONS**

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Tese apresentada ao Programa de Pós-Graduação em Engenharia Civil da Universidade Federal de São Carlos como parte dos requisitos para a obtenção do título de Doutor em Engenharia Civil.

A thesis submitted to the Faculty of Graduate Studies in partial fulfilment of the requirements for the degree of Doctor of Philosophy in Civil Engineering.

Concentration Area: Structures and Geotechnics

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ABSTRACT

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Partially grouted reinforced masonry (PGRM) structures are widely used worldwide, especially in regions with low to moderate seismicity, due to their economic and structural advantages. The nonlinear behavior of PGRM walls is complex, as it is influenced by the interaction between grouted/reinforced and ungrouted regions, which differ in stiffness and introduce non-uniformity in the cross-section. Additionally, factors such as aspect ratio, cross-sectional area, material properties, reinforcement detailing, and axial load significantly influence the nonlinear response. At the building level, further complexities arise from layout configurations, horizontal diaphragm interactions (slabs), and axial load distributions, making it insufficient to rely solely on isolated wall responses to predict overall performance. This thesis addresses these complexities by developing numerical modeling approaches to simulate multi-storey PGRM walls, with the aim of evaluating the seismic performance of building systems and propose practical design recommendations. The research comprises four independent but interconnected studies, each presented as a separate chapter in this thesis.

In the first study, a simplified macro-modeling approach is evaluated to simulate the nonlinear in-plane behavior of PGRM walls with vertical reinforcement concentrated at the pier ends. The model, validated based on experimental data from four walls tested by the author's research group, accurately predicted global behavior, including force-displacement responses and damage mechanisms, while presenting low computational costs. However, further refinement and validation are needed to address additional detailing scenarios, such as cases where vertical reinforcement and grouted regions are not continuous across stories or where reinforcement and grout are distributed non-uniformly along the wall length.

In the second study, a unified finite element-based modeling strategy is introduced that combines macro-modeling for grouted regions and simplified micro-modeling for ungrouted regions. This approach facilitates the accurate simulation of various masonry wall typologies, including unreinforced masonry (URM), fully grouted reinforced masonry (FGRM), and PGRM. The numerical model effectively captured the nonlinear response across different experimental cases, and sensitivity analyses identified key parameters that influence numerical accuracy. This study provides valuable insights for future applications in the simulation of in-plane loaded masonry structures.

The third study extends the unified modeling approach to investigate the seismic behavior of multi-storey PGRM walls within three-dimensional structural systems. Nonlinear static (pushover) analysis results revealed the significant impact of orthogonal walls (flanges), aspect ratios, axial load levels, and vertical reinforcement arrangements. Flanges have been shown to increase lateral stiffness and force capacity, higher axial loads enhance force capacity but reduce ductility, and a greater number of stories decreases stiffness and increases ultimate displacement. The vertical reinforcement concentrated at the wall ends exhibited an efficiency comparable to uniformly distributed reinforcement, demonstrating it as a suitable alternative.

In the fourth study, the seismic performance of two real-world reinforced masonry buildings representative of Canadian (B1) and Brazilian (B2) practices is examined. Using a multi-step methodology that integrates eigenvalue analyses and pushover simulations, the study highlighted the critical role of structural layout in the seismic performance of buildings.

Balanced wall distribution and flange contributions significantly improved performance. Notably, building B2, despite lacking explicit seismic design provisions and detailing, outperformed B1 under higher seismic demands, underlining the importance of optimized structural layouts in enhancing seismic performance.

Overall, this thesis establishes a comprehensive framework for evaluating and improving the seismic performance of concrete masonry buildings. The proposed modeling approaches, validated based on experimental data, provide support for analyzing masonry systems and informing potential updates to seismic design standards. The findings emphasize the importance of regional adaptations in building design and structural layout optimization to achieve better seismic performance.

Keywords: partially grouted masonry, building systems, numerical models, eigenvalue analyses, pushover analysis, seismic performance.

RESUMO

Palhares, Rodolfo de Azevedo. Estruturas de alvenaria de concreto: modelos numéricos, desempenho sísmico e aplicação em edifícios. 334p. Tese de doutorado em Engenharia Civil – Universidade Federal de São Carlos, São Carlos - SP, Brasil, 2025.

Estruturas de alvenaria estrutural parcialmente grauteadas e armadas (APGA) são amplamente utilizadas em todo o mundo, especialmente em regiões com sismicidade baixa a moderada, devido às suas vantagens econômicas e estruturais. O comportamento não linear de paredes de APGA é complexo, pois é influenciado pela interação entre regiões grauteadas/armadas e não grauteadas, que diferem em rigidez e introduzem não uniformidade na seção transversal. Além disso, fatores como a relação de aspecto, área da seção transversal, propriedades dos materiais, detalhamento das armaduras e nível de carga axial influenciam significativamente a resposta não linear. Em nível de edifício, surgem complexidades adicionais devido às configurações do *layout* estrutural, interações entre os diafragmas horizontais (lajes) e distribuições de carga axial, tornando insuficiente a análise isolada de paredes para prever o desempenho global. Esta tese aborda essas complexidades por meio do desenvolvimento de abordagens de modelagem numérica para simular o comportamento de paredes de APGA em edifícios de múltiplos pavimentos, com o objetivo de avaliar o desempenho sísmico de sistemas estruturais e propor recomendações práticas de projeto. A pesquisa é composta por quatro estudos independentes, porém interligados, apresentados em capítulos distintos desta tese.

No primeiro estudo, é avaliada uma abordagem simplificada de macro modelagem para simular o comportamento não linear no plano de paredes de APGA com armaduras verticais concentradas nas extremidades. O modelo, validado com base em dados experimentais de quatro paredes ensaiadas pelo grupo de pesquisa do autor, previu com precisão o comportamento global, incluindo respostas força-deslocamento e mecanismos de dano, apresentando baixo custo computacional. No entanto, são necessários refinamentos e validações adicionais para considerar cenários de detalhamento mais complexos, como casos em que as armaduras verticais e as regiões grauteadas não são contínuas entre os pavimentos ou quando há distribuições não uniformes de armadura e graute ao longo do comprimento da parede.

No segundo estudo, é apresentada uma estratégia unificada de modelagem numérica baseada em elementos finitos, que combina macro modelagem para regiões grauteadas e micro modelagem simplificada para regiões não grauteadas. Esta abordagem facilita a simulação precisa de diferentes tipologias de paredes de alvenaria, incluindo alvenaria não armada (ANA), alvenaria totalmente grauteada e armada (ATGA) e parcialmente grauteada e armada (APGA). O modelo numérico capturou de forma eficaz o comportamento não linear em diferentes casos experimentais, e análises de sensibilidade identificaram parâmetros-chave que influenciam a precisão dos resultados. Este estudo fornece *insights* valiosos para futuras aplicações na simulação de estruturas de alvenaria submetidas a ações no plano.

O terceiro estudo expande a abordagem unificada de modelagem para investigar o comportamento sísmico de paredes de APGA em sistemas estruturais tridimensionais de múltiplos pavimentos. Os resultados das análises estáticas não lineares (*pushover*) revelaram o impacto significativo de paredes ortogonais (flanges), esbeltez, níveis de carga axial e arranjos de armadura vertical. Verificou-se que flanges aumentam a rigidez lateral e a capacidade de resistência, cargas axiais mais elevadas aumentam a capacidade de força, mas reduzem a ductilidade, e um maior número de pavimentos diminui a rigidez e aumenta o deslocamento

último. A concentração de armadura vertical nas extremidades das paredes apresentou eficiência comparável à distribuição uniforme das armaduras, demonstrando ser uma alternativa viável de detalhamento.

No quarto estudo, é avaliado o desempenho sísmico de dois edifícios reais de alvenaria estrutural armada representativos das práticas canadense (B1) e brasileira (B2). Utilizando uma metodologia de múltiplas etapas que integra análises de autovalores e simulações *pushover*, o estudo destacou o papel crítico do *layout* estrutural no desempenho sísmico dos edifícios. A distribuição balanceada de paredes e a contribuição de flanges melhoraram significativamente o desempenho estrutural. Notavelmente, o edifício B2, apesar de não possuir dimensionamento e detalhamento sísmico explícitos, superou o desempenho do edifício B1 sob maiores demandas sísmicas, ressaltando a importância de *layouts* estruturais otimizados para melhorar o desempenho sísmico.

De forma geral, esta tese estabelece uma estrutura abrangente para avaliar e aprimorar o desempenho sísmico de edifícios de alvenaria estrutural de concreto. As abordagens de modelagem propostas, validadas com base em dados experimentais, fornecem suporte para analisar sistemas de alvenaria e informar possíveis atualizações em normas de projeto sísmico. Os resultados destacam a importância de adaptações regionais no projeto dos edifícios e na otimização do *layout* estrutural para alcançar melhor desempenho sísmico.

Palavras-chave: alvenaria parcialmente grauteada, sistemas estruturais, modelos numéricos, análises de autovalores, análise *pushover*, desempenho sísmico.

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LIST OF ABBREVIATIONS

ABNT	Associação Brasileira de Normas Técnicas
ANA	Alvenaria Não Armada
APGA	Alvenaria Parcialmente Grauteada e Armada
ASCE	American Society of Civil Engineers
ATGA	Alvenaria Totalmente Grauteada e Armada
BL	Bilinear Approach
CDP	Concrete Damage Plasticity
CMU	Concrete Masonry Units
CSA	Canadian Standards Association
CSM	Capacity Spectrum Method
DL	Dead Load
DR	Double Reinforcement
DR-JR	Double-grouted cells with bed-joint reinforcement
DS	Damage State
DSFM	Disturbed Stress Field Model
DOF	Degrees of Freedom
EC8	Eurocode 8
EN	Euro Norm
FE	Finite Element
FEM	Finite Element Method
FEMA	Federal Emergency Management Agency
FG	Fully Grouted
FGRM	Fully Grouted Reinforced Masonry
GM	Grouted Masonry
HSC	Hollow Core Slabs
IDA	Incremental Dynamic Analysis
MC	Masonry Cement
MCE	Maximum Considered Earthquake
MCFT	Modified Compression Field Theory
MDOF	Multi-Degree-of-Freedom
MPCLB	Multi-Perforated Clay Bricks

NBCC	National Building Code of Canada
NBR	Norma Brasileira
NIST	National Institute of Standards and Technology
NLTH	Non-Linear Time History
NTC	Norme Tecniche per le Costruzioni (in Italian)
PCL	Type S Portland Cement Lime
PG	Partially Grouted
PGA	Peak Ground Acceleration
PGRM	Partially Grouted Reinforced Masonry
RC	Reinforced Concrete
RM	Reinforced Masonry
SDOF	Single-Degree-of-Freedom
SFRS	Seismic Force-Resisting System
SR	Single Reinforcement
SPF	Seismic Performance Factor
TL	Trilinear
TMS	The Masonry Society
TSCM	Total Strain Crack Model
UCalgary	University of Calgary
UFRJ	Universidade Federal do Rio de Janeiro
UFSCar	Universidade Federal de São Carlos
UGM	UngROUTED Masonry
URM	Unreinforced Masonry

LIST OF SYMBOLS

A_B	Area of the base of the structure
A_{eh}	Effective horizontal cross-sectional area of masonry
A_{ev}	Effective vertical cross-sectional area of masonry
A_i	Web area of the shear wall i
A_t	Cross-sectional area
A_v	Total cross-sectional area of vertical reinforcement
B_{max}	Maximum floor plan width
B_{min}	Minimum floor plan width
C	Vertical reinforcement concentrated at the wall ends
c	Cohesion
C_d	Displacement amplification factor
C_h	Base shear coefficient
C_r	Compactness ratio
D	Vertical reinforcement distributed along the wall
d	Lateral displacement
d_{Vmax}	Displacement at the level of the maximum lateral force
d_{Vu}	Ultimate displacement
E	Elastic modulus
f_b	Block nominal compressive strength
F_{cy}	Compressive yielding forces
f_m	Masonry compressive strength
f_{ti}	Tensile strength of interfaces
f_{tm}	Masonry tensile strength
F_{ty}	Tensile yielding forces
f_u	Ultimate strength
f_{v0}	Initial shear resistance
f_{vh}	Yield strength of horizontal reinforcement
f_{vy}	Yield strength of vertical reinforcement
f_y	Yield strength of steel reinforcements
g	Average gravity load per storey floor area
G	Initial shear modulus

G^*	Post elastic shear modulus
g_{max}	Maximum structural wall gravity load per floor
g_{min}	Minimum structural wall gravity load per floor
G_c	Compressive fracture energy
G_f	Tensile fracture energy
h_{int}	Mortar joint height
h_p	Height of plastic hinge regions
h_w	Height of the wall
I_e	Importance factor
k_{gh}	Factor concerning the horizontal grouting
k_{gv}	Factor concerning the vertical grouting
k_n	Normal stiffness
K_p	Initial stiffness
K_{sec}	Secant stiffness
k_t	Tangential stiffness
l_d	Length of diagonal elements
l_w	Length of wall
M	Flexural moment
m_i	Lumped mass at the i -th floor
M_v	Factor specified in NBCC (2020)
P	Axial compressive load
P_e	External perimeter
q	Structural behavior factor
R	Response modification factor
R_0	Component of the response modification factor due to overstrength
R_d	Component of the response modification factor due to ductility
R_s	Component of the response modification factor due to overstrength from other sources
$S(T)_{max}$	Maximum attainable spectral accelerations
S_a	Spectral acceleration
$S_{a,5\%}$	Spectral acceleration for 5% damping
$S_{a, reduced}$	Reduced spectral acceleration
S_d	Spectral displacement

s_{max}	Maximum spacing between the flexural springs
T	Natural period
t_n	Normal traction
t_t	Shear traction
U	Dissipated energy
u_{cu}	Ultimate compressive displacements
u_m	Internal displacement
u_{tu}	Ultimate tensile displacements
V	Shear force
V_e	Maximum base shear force if the structure remains elastic
V_{el}	Elastic load level
V_{max}	Maximum lateral force
V_n	Nominal shear load capacity
V_y	Yield load level
W	Seismic weight of the building
α	Hardening/softening parameter
α_m	Effective mass ratio
β	Coefficient to account for the shear span ratio
Γ	Modal participation factor
γ_u	Ultimate angular deformation
δ_u	Displacement of the panel
ε_{c0}	Transition compressive strain
ε_{cu}	Ultimate compressive strain
ε_{tu}	Ultimate tensile strain
λ	Slenderness ratio
λ_{max}	Maximum slenderness ratio
λ_{min}	Minimum slenderness ratio
μ	Coefficient of friction
μ_d	Displacement ductility factor
ν	Poisson's ratio
ξ_0	Initial elastic damping
ξ_{eff}	Effective damping
ρ_h	Total horizontal reinforcement ratio

ρ_v	Total vertical reinforcement ratio
ρl_w	Average wall length per floor area
ρ_{reinf}	Vertical reinforcement ratio
$\rho_{reinf,x}$	Vertical reinforcement ratio in the x direction
$\rho_{reinf,y}$	Vertical reinforcement ratio in the y direction
σ	Average axial stress
σ_c	Compressive stress
$\sigma_{s,Vmax}$	Stress values in the outermost rebar at peak load
σ_t	Tensile stress
$\sigma_{v,max}$	Maximum axial load stresses
$\sigma_{v,min}$	Minimum axial load stresses
τ	Lateral stress distribution
τ_{max}	Maximum shear stress
τ_y	Yield shear stress
ϕ	Friction angle
φ_i	i -th mode shape
Ω	Overstrength factor
β_r	Factor concerning the shear span ratio
F_a	Amplification factor
S_a	Spectral acceleration
θ	Angle formed between the wall axis and the strut
Δu_n	Normal relative displacement
Δu_t	Shear relative displacement

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1. INTRODUCTION

Engineered masonry has undergone significant advancements in the last decades, consolidating itself as a prominent construction technique in many world regions. Masonry structural members can be classified as unreinforced (URM) or reinforced (RM), with RM further divided into fully grouted (FGRM) or partially grouted (PGRM) systems. The introduction of reinforcement in RM addresses masonry's inherent low tensile strength, improving its structural performance under lateral forces caused by wind or seismic events (Tomaževič; Weiss, 1994). Several studies have consistently demonstrated that RM exhibit excellent response in terms of lateral load capacity, ductility, crack control, and energy dissipation in contrast to URM, e.g., Alves (2011), Haach (2009), Magenes et al. (1996), Voon and Ingham (2006). Among RM systems, PGRM is widely adopted worldwide, mainly in regions with low to moderate seismicity, including countries such as the United States, Canada, New Zealand, Australia, Chile, Brazil, and several European nations, due to its economic advantages (Drysedale; Hamid, 2005; Haach; Vasconcelos; Lourenço, 2011; Janaraj; Dhanasekar, 2016; Koutras; Shing, 2021; Medeiros et al., 2022a; Sandoval; Calderón; Almazán, 2018; Voon; Ingham, 2006).

Despite advances in understanding the behavior of individual masonry walls, e.g., Bolhassani et al. (2016b), Calderón et al. (2020; 2021a), Elmapruk et al. (2020), Elmeligy et al. (2021), Ghanem et al. (1992), Minaie et al. (2010), Nolph and Elgawady (2012), Ramírez et al., (2016), Voon and Ingham (2006), Zhang et al. (2021), research at the building system level remains limited. Masonry buildings often incorporate walls with varying characteristics, such as cross-sectional shapes, lengths, aspect ratios, axial load levels, connections, and reinforcement detailing. These variations introduce complexity into the seismic behavior since the overall lateral response of interlocked walls in a building differs from the simple summation of individual wall responses (Heerema et al., 2015; Heerema; Shedid; El-Dakhakhni, 2015; Noor-E-Khuda, 2022).

Seismic design practices vary significantly across regions, influenced by seismicity levels and construction traditions. In Brazil, the seismic design standard ABNT NBR 15421 (2023) establishes the criteria for quantifying and verifying seismic actions in structural design. However, based on seismic hazard analysis and design requirements, the standard waives the need to check structures in Zone 0, which covers approximately 80% of Brazil's territory.

Moreover, ABNT NBR 15421 (2023) does not specify explicit seismic detailing requirements for masonry structures, leaving a gap in design guidance. In contrast, North American standards offer more specific regulations. Canadian code CSA S304 (2024) provides detailed design and reinforcement requirements for different categories of masonry structures. In addition, the NBCC (2020) restricts the use of masonry buildings based on the seismic category, defined by the spectral acceleration value, $S_a(0.2)$.

Seismic analysis methods can be broadly categorized into linear and nonlinear approaches, further divided into static and dynamic analysis procedures. These different methods remain specified in modern seismic design standards (e.g., EN 1998-1 (2004); NTC (2018); ABNT NBR 15421 (2023), and include:

- Linear static analysis (equivalent lateral force method): It involves the application of horizontal static forces equivalent to seismic inertia forces obtained from a design spectrum.
- Linear dynamic analysis (modal response spectrum method): It considers the responses from significant vibration modes, ensuring modal masses represent at least 90% of the structure's total mass.
- Nonlinear static (pushover) analysis: This method considers increments of horizontal forces to a nonlinear structural model until collapse conditions are reached, resulting in a force-displacement (capacity) curve representing the seismic response.
- Nonlinear dynamic (time-history) analysis: It integrates the equilibrium equations of motion using representative ground acceleration records, providing the most accurate insight into seismic behavior.

Nonlinear dynamic analysis is the most realistic representation of seismic response, but its high computational demand makes it less attractive for routine engineering practice. Nonlinear static analysis (i.e., pushover) and linear elastic (i.e., equivalent lateral force and response spectrum) methods are more commonly applied, particularly for geometrically regular buildings, given their good balance between computational simplicity and accuracy (Magenes, 2006; Morandi et al., 2022; Morandi, 2006).

Numerical modeling has become an indispensable tool for simulating the behavior of structures, especially given the high costs and complexities of experimental tests. Modeling approaches for masonry are generally classified into micro- and macro-modeling, based on the level of detail (Haach; Vasconcelos; Lourenço, 2011; Lourenço, 1996):

- **Micro-modeling:** This approach discretizes masonry at the level of its constituent elements — units, mortar, and interfaces — offering a detailed understanding of localized behavior and damage mechanisms. It is further divided into:

- Detailed micro-modeling: Units and mortar are modeled as continuum elements, with the interfaces represented as discontinuous elements.

- Simplified micro-modeling: Mortar joints and interfaces are replaced with zero-thickness interfaces, while units are expanded to preserve geometry.

- **Macro-modeling:** Masonry is treated as a homogeneous continuum to capture the global structural response. The macro-modeling approach is computationally efficient and best suited for analyzing large-scale masonry systems, particularly in practical design contexts.

For PGRM systems, the interaction between grouted and ungrouted regions poses unique challenges. These interactions create stiffness disparities and non-uniform cross-sections, complicating the nonlinear behavior modeling. Developing comprehensive yet computationally efficient modeling strategies remains a critical area of masonry research.

Modern seismic design philosophy allows consideration of inelastic response in structures such that design elastic seismic forces are reduced based on ductility and overstrength parameters. However, the force reduction assumes a level of ductility that may not be realistic if the seismic modeling is not accurate. The seismic parameters are often misinterpreted, leading to rather unsafe or uneconomic design even in low to moderate seismicity regions (Brzev; Anderson, 2018; The Masonry Society (TMS), 2022).

In Brazil, many designers fail to consider seismic actions due to a lack of technical knowledge or misconceptions about seismic risks. However, studies have revealed structural vulnerabilities in small reinforced concrete buildings designed without considering seismic actions or specific detailing (Dantas, 2013; Galvão, 2013; Miranda, 2013, 2021; Parisenti, 2011). In the context of masonry buildings, the lack of understanding is even more pronounced, emphasizing the need for more research to address seismic effects on Brazilian masonry structures.

Structural typologies, wall configurations, and regional seismic demands play a key role in the seismic performance of masonry buildings. Low to mid-rise buildings, representing most masonry constructions, are particularly critical regarding seismic assessment and safety due to their fundamental periods often aligning with the peak of the design spectra, where seismic demands are higher. Despite their importance, research into the behavior of these structures across diverse regions remains limited. In particular, Brazil's seismic design framework lacks

specificity in key areas, such as a comprehensive definition of the response modification factor, R , and detailing requirements, underscoring the need for further investigation.

1.1 OBJECTIVES

The research described in this thesis aims to expand the knowledge about the seismic behavior and performance of concrete masonry structures, particularly in multi-storey PGRM systems, mainly through numerical analysis. More specifically, the objectives of the study are:

- To develop a simplified low-cost numerical modeling approach to accurately simulate the nonlinear behavior of concrete masonry walls under lateral loads.
- To calibrate and validate the numerical model approach using experimental datasets for URM, FGRM, and PGRM walls, considering variables such as wall aspect ratios, axial stress levels, and reinforcement detailing.
- To assess three-dimensional masonry building seismic performance through nonlinear static (pushover) analyses to explore system-level behavior regarding lateral response, including stiffness evolution, base shear resistance, displacement capacity, and ductility.
- To evaluate the influence of factors such as orthogonal walls, building height, aspect ratio, axial load, and reinforcement distribution on the nonlinear behavior of masonry systems.
- To compare the seismic performance of masonry buildings designed according to Brazilian and Canadian standards, focusing on regional and structural differences.
- To examine the influence of structural layouts, material properties, and reinforcement detailing on the seismic performance of masonry buildings in Brazil and Canada.
- To analyze, through modal analyses of building models with varying heights, the accuracy and potential adjustments of simplified equations provided by design standards for estimating the fundamental natural period, T .
- To derive seismic design/performance parameters, especially the response modification factor, R , from pushover analysis results, to be used in linear seismic design of masonry buildings.

1.2 MOTIVATION AND RESEARCH SIGNIFICANCE

Although substantial advances have been made in understanding the lateral response of individual masonry walls, a comprehensive perspective on the seismic behavior of complete building systems is still lacking. At the individual wall level, parameters such as cross-sectional area, material properties, reinforcement detailing, aspect ratio, and axial load significantly influence the nonlinear behavior. When moving to the building level, additional parameters — including layout configurations, slab diaphragm effects, and axial load distributions — introduce further complexity, highlighting the limitations of summing isolated wall responses to predict overall building performance.

Stiffness disparities, non-uniform cross-sections, and complex interactions between grouted/reinforced and ungrouted regions in PGRM systems present significant challenges for accurate modeling, particularly in nonlinear regimes. Developing comprehensive numerical modeling approaches that effectively address these complexities while maintaining a balance between computational efficiency and predictive accuracy is crucial for understanding the behavior of these systems.

Linear seismic analysis, based on the specification of a response modification factor, R , remains the most common design method in low to moderate seismic regions. However, limited research on R factors for concrete masonry buildings affirms the need for studies to refine these parameters and improve standard provisions. The Brazilian seismic design standard, ABNT NBR 15421 (2023), specifies an R value of 1.5 for structural masonry systems with “usual detailing”. However, neither ABNT NBR 15421 (2023) nor the Brazilian Structural Masonry Design standard ABNT NBR 16868 (2020) explicitly defines “usual detailing”. This lack of specificity reflects the limited research on the topic and highlights a significant gap in the Brazilian seismic design framework for masonry structures. Addressing this gap requires further research to develop and formalize detailed criteria and guidelines for masonry systems.

Furthermore, research into the seismic performance of concrete masonry buildings across different seismic regions remains limited. Therefore, it is crucial to develop studies that support incorporating current seismic design criteria into standards for structural masonry, with particular emphasis on defining rationally appropriate values for response modification factors to be used in linear analyses. Addressing these research gaps is essential to ensure continued progress in the field, more sustainable construction, and safer buildings.

1.3 THESIS SCOPE AND ORGANIZATION

This thesis is part of collaborative research involving the Universidade Federal de São Carlos (UFSCar) — Brazil and the University of Calgary (UCalgary) — Canada. Based on previous studies by the research group, this thesis is composed of seven chapters, four of which are based on peer-reviewed journal papers.

In this chapter (Chapter 1), the research themes and their relevance have been contextualized, as well as the objectives to be achieved, the motivation, and an overall view of how the thesis is organized.

The relevant literature is reviewed in Chapter 2, in which the key findings from experimental studies, numerical modeling efforts, and standard-based specifications concerning masonry structures' behavior and seismic performance, focusing on R factors, are noted and synthesized.

A simplified macro-modeling approach is developed in Chapter 3 for accurately analyzing the non-linear behavior of multi-storey perforated PGRM walls with vertical reinforcement concentrated at the pier ends. This chapter is based on:

- R. A. Palhares, K. A. S. Medeiros, G. A. Parsekian, N. G. Shrive, and R. Marques, “A macro-modeling approach for non-linear analysis of multi-storey perforated masonry walls with grout and reinforcement concentrated at their pier ends,” *J. Build. Eng.*, vol. 73, p. 106785, Aug. 2023, doi: 10.1016/j.job.2023.106785.

In Chapter 4, a unified macro-modeling approach for simulating the seismic response of concrete masonry shear walls is established and validated, encompassing various configurations and detailing scenarios. This chapter is based on:

- R. A. Palhares, K. A. S. Medeiros, G. A. Parsekian, N. G. Shrive, and R. Marques, “A unified modeling approach for concrete masonry shear walls: Conception, validation and insights,” *Eng. Struct.*, vol. 321, p. 119016, Dec. 2024, doi: 10.1016/j.engstruct.2024.119016.

The seismic performance of multi-storey PGRM walls within three-dimensional building systems, highlighting critical parameters such as flanges, aspect ratios, and reinforcement strategies, is investigated in Chapter 5. This chapter is based on:

- R. A. Palhares, K. A. S. Medeiros, G. A. Parsekian, N. G. Shrive, and R. Marques, “Seismic assessment of partially grouted and reinforced multi-storey masonry walls in 3D systems,” in review, 2025.

In Chapter 6, the seismic performance of concrete masonry buildings in distinct regional contexts, such as Brazil and Canada, is compared, considering differences in structural design practices and seismic demands. The integration of deterministic and probabilistic approaches strengthens the findings. This chapter is based on:

- R. A. Palhares, K. A. S. Medeiros, G. A. Parsekian, N. G. Shrive, and R. Marques, “Seismic performance evaluation of concrete masonry buildings in different regional contexts: Canada vs. Brazil,” in review, 2025.

The main findings, highlighting contributions and suggestions for future research directions are summarized in Chapter 7.

Two appendices are provided at the end of the thesis to present supplementary information to the previous chapters.

2. LITERATURE REVIEW

2.1 INTRODUCTION

A comprehensive review of the previous research on the seismic behavior of concrete masonry structures is provided in this chapter. The literature review focuses on experimental and numerical investigations, numerical modeling approaches, and the findings on the response modification factors, R , which are essential to the linear seismic design of structures.

The chapter is structured as follows: In Section 2.2, experimental studies that offer insights into the inelastic behavior of concrete masonry structures are discussed, covering both individual walls and building systems. In Section 2.3, different numerical modeling approaches, such as macro- and micro-modeling, used to simulate the nonlinear response of concrete masonry structures are examined. The variability of R factors across different seismic design standards and research presented in the literature is examined in Section 2.4. Final remarks on the considered literature are given in Section 2.5. This literature review aims to provide a cohesive understanding of the progress made in masonry seismic design and highlight areas requiring further exploration.

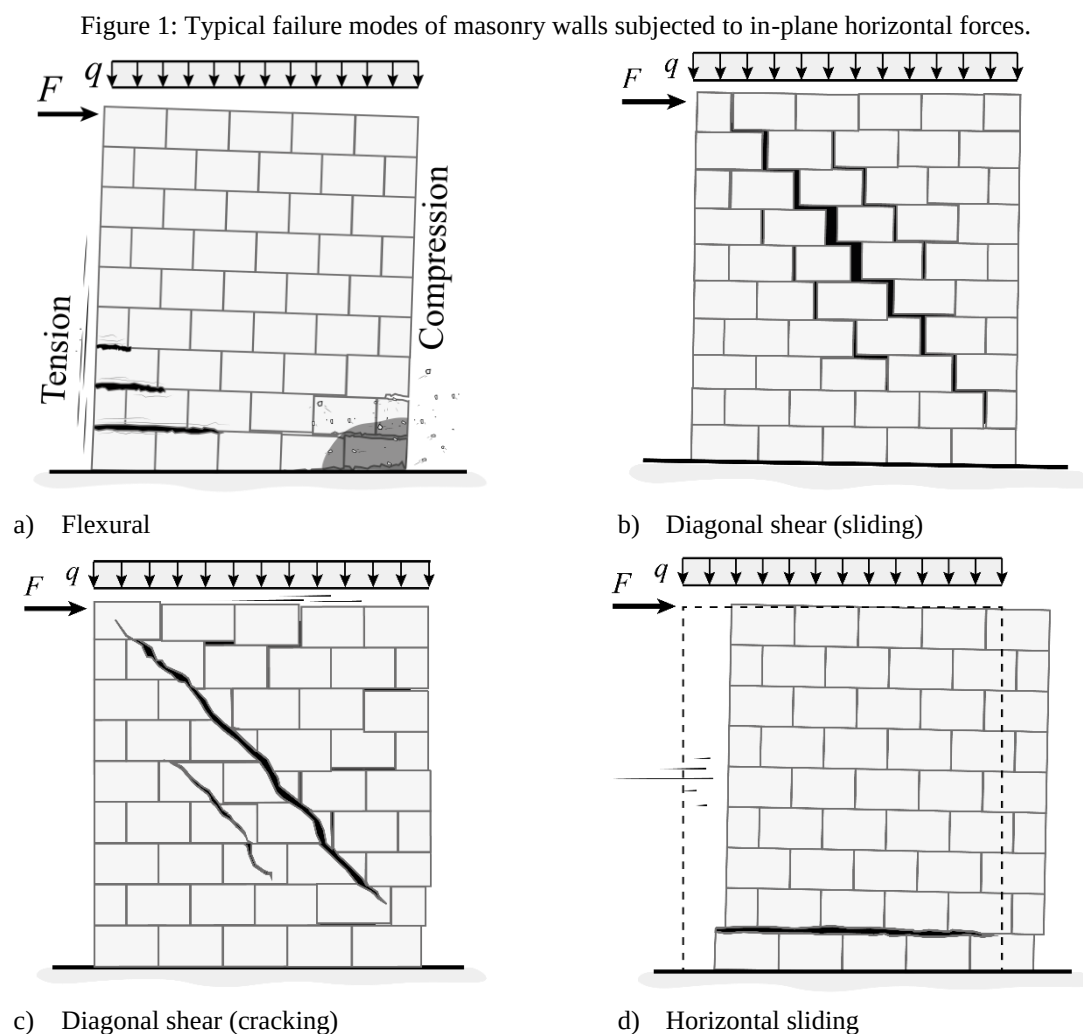
2.2 SEISMIC RESPONSE OF CONCRETE MASONRY STRUCTURES

A brief literature review of more recent research efforts on the seismic behavior of reinforced concrete block masonry shear walls, considering both individual walls (members) and systems (buildings), is provided in this section.

2.2.1 Lateral resistance mechanisms of in-plane walls

The resistance mechanisms and failure modes of masonry walls subjected to in-plane horizontal loads are influenced by several factors, including the aspect ratio (the height-to-width ratio of the wall), the level of applied axial load, the properties of the constituent materials, geometry boundary conditions, and the detailing of vertical and horizontal reinforcements. Generally, three main failure modes can be identified (or a combination thereof): flexural, diagonal shear, and sliding shear (Drysdale; Hamid, 2005; Shedid; Drysdale; El-Dakhakhni, 2008). As noted by Marques and Lourenço (2011), the diagonal shear failure mode can manifest

through two distinct mechanisms: diagonal sliding and diagonal cracking. The typical mechanisms mentioned are illustrated in Figure 1.



Fonte: Adapted from Marques and Lourenço (2011).

Based on the literature considerations (Alves, 2011; Drysdale; Hamid, 2005; Morandi, 2006; Shedi; Drysdale; El-Dakhakhni, 2008; Voon; Ingham, 2006), these mechanisms can be described as follows:

- **Flexural:** Flexural-dominated walls are characterized by a ductile failure mode. As the applied horizontal force increases, horizontal cracks form in the mortar bed joints at the base of the tensioned section (most stressed zones). The masonry resists the load until failure, either through crushing combined with reinforcement buckling in the compression zones or the vertical reinforcement yielding and fracture in the tension zones.

- **Shear:** Shear-dominated are characterized by their brittle failure mode, in which walls lose their lateral stiffness and strength relatively abruptly due to diagonal cracks that may follow the mortar joints (horizontal and vertical) or traverse both the joints and the masonry blocks.

The shear strength of the mortar joints, the tensile strength of the masonry, and the tensile strength of the masonry units are critical parameters influencing this failure mode.

- Sliding: Sliding planes develop along continuous horizontal joints. This failure mode is common in cases of low vertical load and/or a low friction coefficient at the mortar-block interface. The shear strength of the joints is the key parameter governing this mechanism.
- Shear-sliding combined: A combined shear-sliding failure mode can occur, particularly in walls with low aspect ratios (squat walls). This mechanism is characterized by the simultaneous development of diagonal shear cracks and sliding along horizontal joints or at the wall base. The interaction between these failure modes results in a complex response where both shear strength and friction play crucial roles.

2.2.2 Perspective on individual wall components

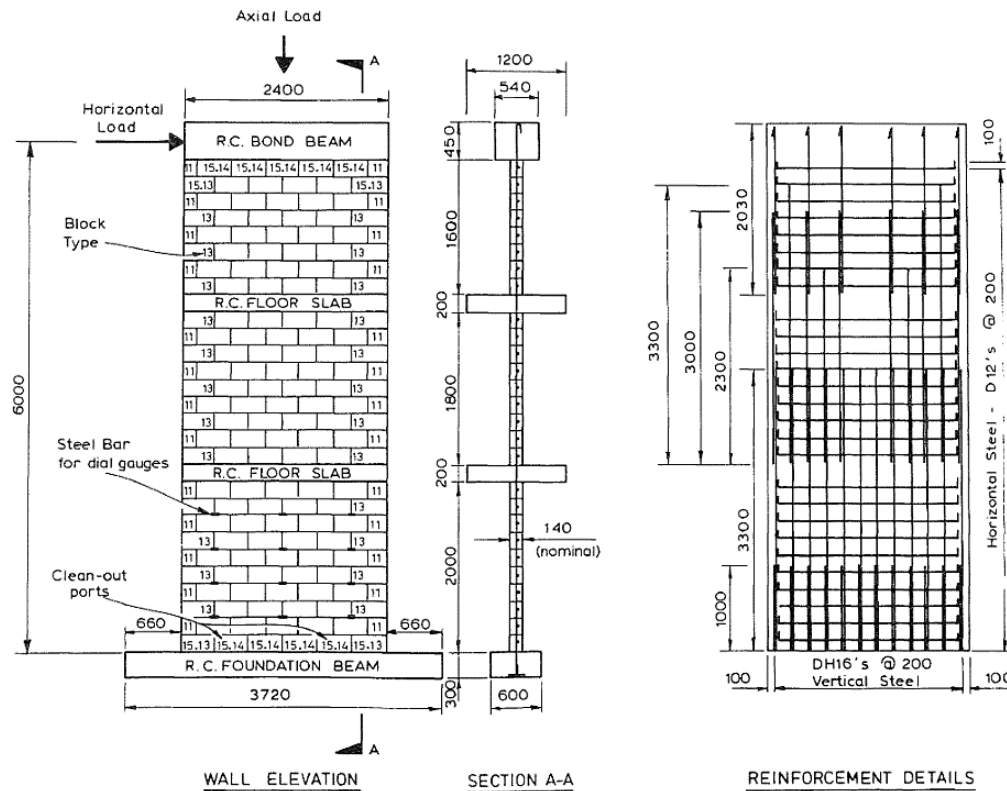
Due to the challenges of conducting large-scale building tests, shear walls are often analyzed individually as they play a fundamental role in building systems, providing essential lateral load resistance and energy dissipation. The nonlinear behavior of shear walls is influenced by several parameters, including the aspect ratio (height-to-length), grouting, the amount and spacing of reinforcement, the presence of openings, the strength of individual components, boundary conditions, and axial load (Calderón et al., 2020; Calderón; Sandoval; Arnau, 2017; Ghanem et al., 1992; Haach; Vasconcelos; Lourenço, 2011; Priestley; Elder, 1982; Shedid; Drysdale; El-Dakhakhni, 2008; Shing et al., 1989; Shing; Schuller; Hoskere, 1990; Voon; Ingham, 2006). The wide range of possible combinations of these parameters, coupled with the inherent heterogeneity of masonry, reflects the complexity of fully comprehending wall inelastic behavior. Some experimental studies focusing on the characteristics of the tested walls and the influence of various parameters on their performance are presented as follows.

2.2.2.1 Priestley and Elder (1982)

Priestley and Elder (1982) tested three slender reinforced masonry (RM) walls with axial load variations and included steel confining plates in the plastic hinge regions. These walls were FGRM, constructed with concrete blocks, and scaled down at a 1:0.737 ratio to represent multi-storey structures. Each wall had a total height of six meters, representing approximately a nine-meter height in a full-scale three-storey building. The walls were 2400 mm long, giving them an effective aspect ratio of 2.5. The walls were subjected to in-plane cyclic displacements

applied at their tops to simulate seismic loading conditions. The general characteristics of the walls are illustrated in Figure 2.

Figure 2: Arrangement of a wall tested by Priestley and Elder (1982). Dimensions in mm.



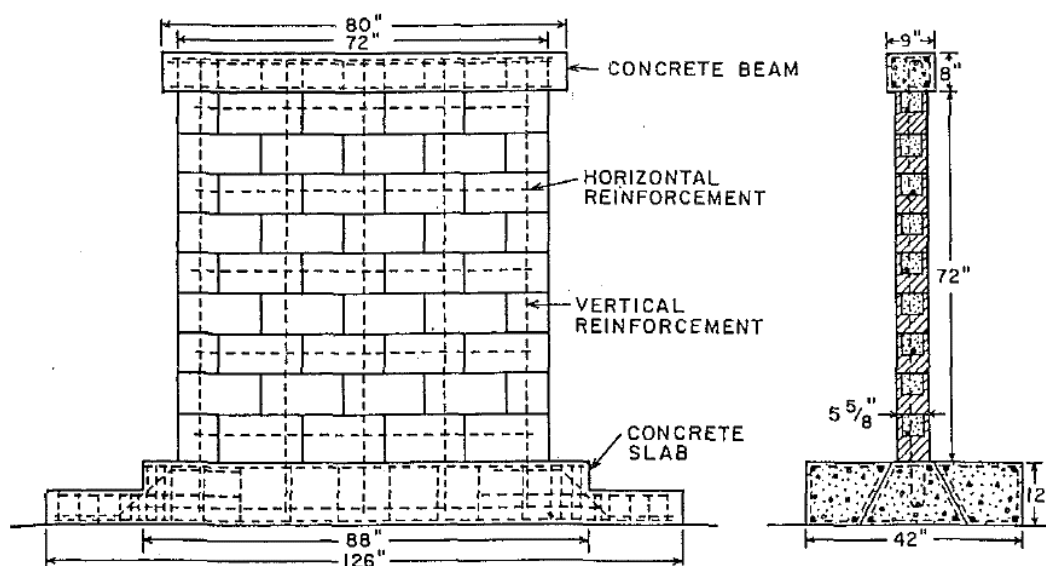
Source: Priestley and Elder (1982).

The study highlighted the critical influence of axial loading and confinement on the seismic behavior of the walls. Higher axial loads led to pronounced strength degradation and reduced ductility, particularly in walls without confinement, as the increased forces amplified internal stresses and accelerated failure mechanisms such as cracking and bond loss between reinforcement and masonry. Confining plates in the plastic hinge regions significantly enhanced wall performance by improving ductility and reducing damage. Additionally, the study identified the detrimental effects of overlapping vertical reinforcement within the plastic hinge region, as this practice led to premature bond failure and concentrated plastic deformation, ultimately limiting ductility. As a result, Priestley and Elder (1982) recommended avoiding reinforcement overlaps at the base of walls and other critical regions where plastic hinges are likely to form.

2.2.2.2 Shing et al. (1989, 1990)

Shing et al. (1989; 1990) investigated the seismic performance of RM shear walls under in-plane loading, focusing on the effects of axial stress and vertical and horizontal reinforcement on lateral resistance, ductility, and energy dissipation. The experimental program included 16 fully grouted reinforced masonry (FGRM) wall specimens, each measuring 1830 mm x 1830 mm, with uniformly distributed reinforcement (Figure 3). Vertical reinforcement ratios were set at 0.38%, 0.54%, and 0.74%, while horizontal reinforcement ratios were 0.14% and 0.24%.

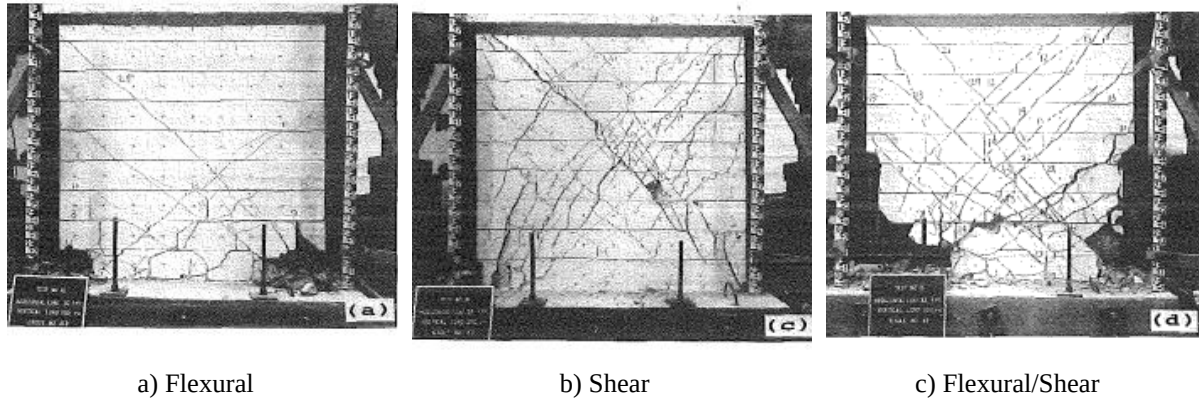
Figure 3: General arrangement of walls tested by Shing et al. (1989, 1990). Dimensions in inches.



Source: Shing et al. (1989, 1990).

As shown in Figure 4, two dominant failure mechanisms were identified: flexural failure, associated with higher axial stress and vertical reinforcement ratios, and shear failure, characterized by diagonal cracking. Walls exhibiting flexural failure demonstrated greater ductility and energy dissipation than those with shear failures, which were limited by brittle mechanisms and lower deformation capacities. Maximum ductility ratios reached 18 for flexural-dominated specimens, compared to 4-7 for shear-dominated walls.

Figure 4: Observed failure modes in RM walls tested by Shing et al. (1989, 1990).



Source: Shing et al. (1989, 1990).

Vertical and horizontal reinforcement roles were significant in governing the post-cracking phase. Increasing vertical reinforcement from 0.38% to 0.74% improved ductility in flexural walls, with the maximum ductility nearly doubling. In contrast, shear strength was governed by diagonal cracking and influenced by masonry tensile strength and horizontal reinforcement. Higher horizontal reinforcement ratios (e.g., 0.24% versus 0.14%) shifted behavior from brittle shear to more ductile flexural/shear modes, enhancing energy dissipation by up to 55% in normalized terms.

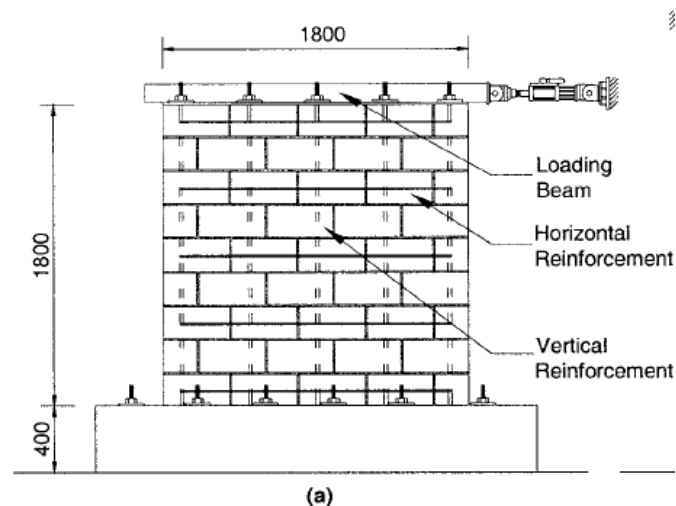
The study found that failure modes are highly sensitive to axial stress. Increasing axial stress on the walls transitioned failure from ductile flexural to brittle shear. While axial stress improved flexural and shear strength, it led to brittle shear failures in shear-dominated walls at higher stress levels. Mixed flexural-shear failure modes occurred at intermediate reinforcement levels, balancing strength and ductility. Shing et al. (1989; 1990) emphasized that balanced reinforcement detailing is critical to achieving ductile behavior and improved seismic performance.

Additionally, Shing et al. (1989; 1990) reported that the masonry tensile strength governs the initiation of diagonal shear cracking. Once the diagonal cracks occur, the residual wall capacity is due to the dowel forces of the vertical flexural reinforcement, the horizontal reinforcement, and the aggregate-interlock forces. These forces, particularly aggregate interlock, were influenced by axial loading, which limited crack opening and contributed to shear resistance after cracking.

2.2.2.3 Voon and Ingham (2006, 2008)

Voon and Ingham (2006) investigated the in-plane shear strength of RM walls, focusing on axial compression, reinforcement, grouting, and aspect ratio effects. The study included ten single-storey walls, varied between fully and partially grouted walls tests, each measuring 1800 mm x 1800 mm, as depicted in Figure 5.

Figure 5: Arrangement of walls tested by Voon and Ingham (2006). Dimensions in mm.



Source: Voon and Ingham (2006).

The experimental results identified diagonal cracking and shear-dominated failures, characterized by rapid strength degradation immediately after reaching their peak resistance. The axial stress directly influenced the lateral strength and inversely affected the ductility of the walls. An increase in axial compression from 0.0 to 0.5 MPa resulted in up to a 22% improvement in shear strength. However, due to brittle failure mechanisms, higher axial stress levels reduced post-cracking deformation capacity.

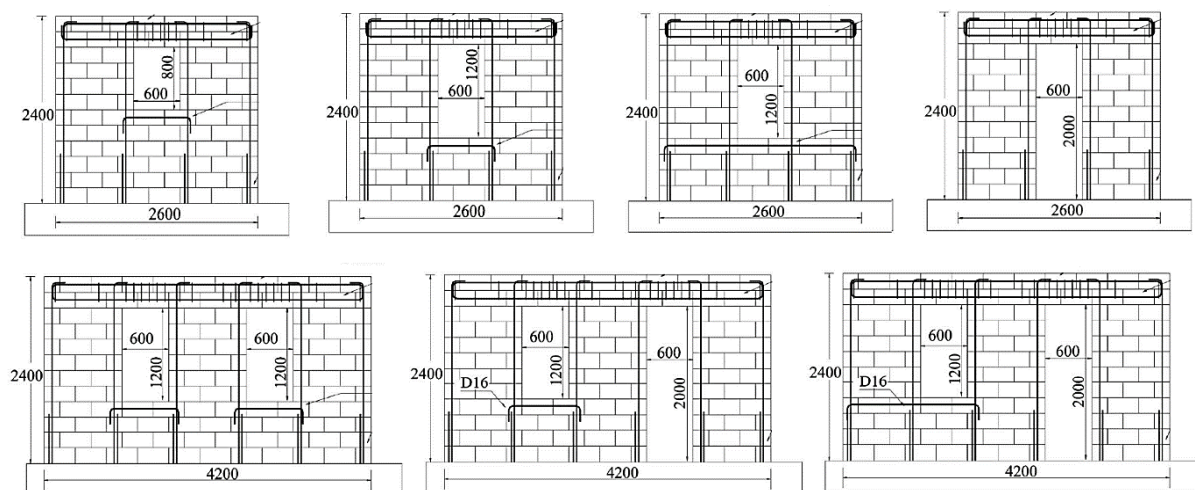
The test results also demonstrated that the lateral strength decreased inversely with the aspect ratio. Furthermore, the results indicated that, although partial grouting significantly reduced the masonry shear strength, the effect of grouting became less significant when the net shear stress was calculated using the cross-sectional area of both the masonry units and the grouted cells. This observation was also reported by Medeiros et al. (2022a).

Horizontal shear reinforcement enhanced wall strength and ductility, with a 10% increase in shear strength observed when the horizontal reinforcement ratio was increased from 0.02% to 0.14%. The ductility, particularly after crack formation, was higher when the same

horizontal reinforcement ratio was distributed along the wall height using more bars with smaller diameters.

Voon and Ingham (2008) extended their experimental program to include walls with openings, illustrated in Figure 6, aiming to assess their impact on lateral strength and behavior. Eight partially grouted reinforced masonry (PGRM) single-storey walls with varying opening sizes and reinforcement details were tested under cyclic lateral loading.

Figure 6: Arrangement of walls tested by Voon and Ingham (2008). Dimensions in mm.



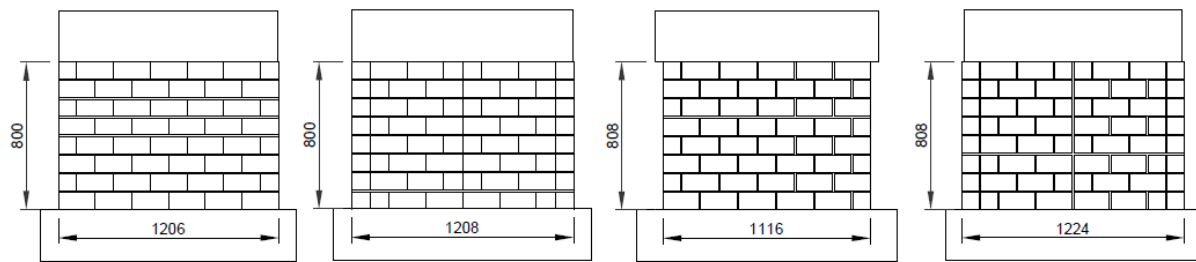
Source: Voon and Ingham (2008).

The experimental results revealed that openings significantly reduced the lateral load capacity of the walls, with larger and taller openings causing greater reductions in lateral strength. Moreover, the study demonstrated that extending horizontal reinforcement below the openings improved lateral strength, highlighting its importance in mitigating the weakening effects of openings.

2.2.2.4 Haach (2009)

Haach (2009) conducted an extensive experimental program on concrete masonry walls at half-scale, with dimensions of approximately 800 mm in height and 1200 mm in length, as illustrated in Figure 7. The specimens were constructed with distinct reinforcement configurations, including variations in horizontal and vertical reinforcement placement.

Figure 7: Arrangement of walls tested by Haach (2009). Dimensions in mm.



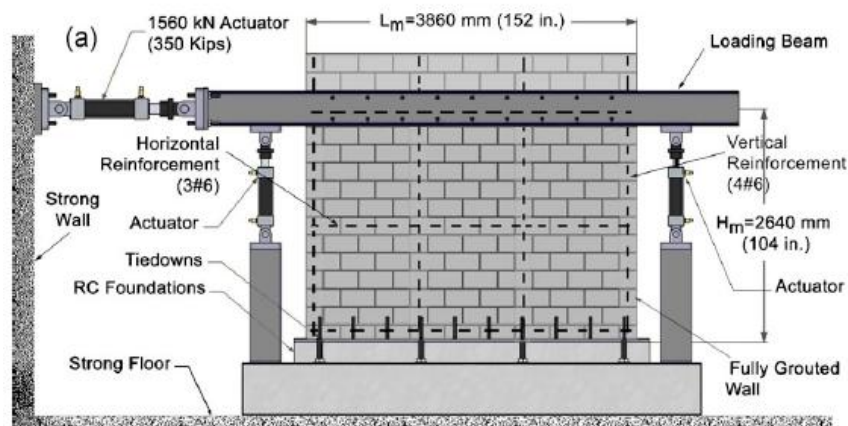
Source: Haach (2009).

The study observed a brittle response in unreinforced masonry walls, while the inclusion of reinforcement significantly improved deformation capacity, controlled crack propagation, and ensured a better distribution of internal stresses. Vertical reinforcement enhanced the flexural strength of the walls, although it showed minimal impact on shear strength. Conversely, horizontal reinforcement increased the shear strength but did not notably affect flexural behavior. The results also highlighted that higher pre-compression levels increased lateral strength at the expense of a more brittle failure mode.

2.2.2.5 Minaie et al. (2010)

Minaie et al. (2010) investigated the cyclic in-plane behavior of four full-scale PGRM walls, each measuring 2640 mm x 3860 mm, as shown in Figure 8. The experimental program examined the effects of varying mortar formulations (Portland cement lime vs. masonry cement), boundary conditions (cantilever behavior vs. double curvature), and vertical stress levels.

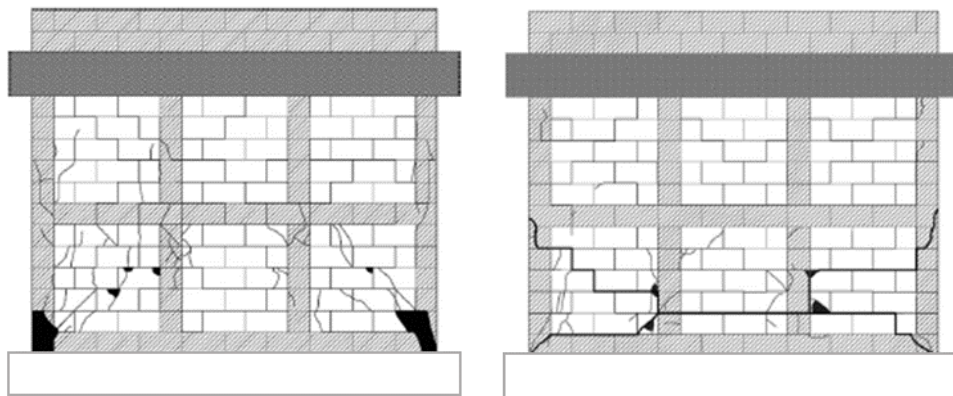
Figure 8: Test setup of walls evaluated by Minaie et al. (2010). Dimensions in mm.



Source: Minaie et al. (2010).

The experimental tests showed diagonal stair-stepped cracking as the predominant failure mode, as illustrated in Figure 9. The authors concluded that the PGRM walls behave similarly to in-filled frames: the grouted and reinforced regions deform like a concrete frame, while the unreinforced masonry panels act as masonry infill. This behavior is notably distinct from that of FGRM walls.

Figure 9: Crack pattern of walls evaluated by Minaie et al. (2010).



Source: Minaie et al. (2010).

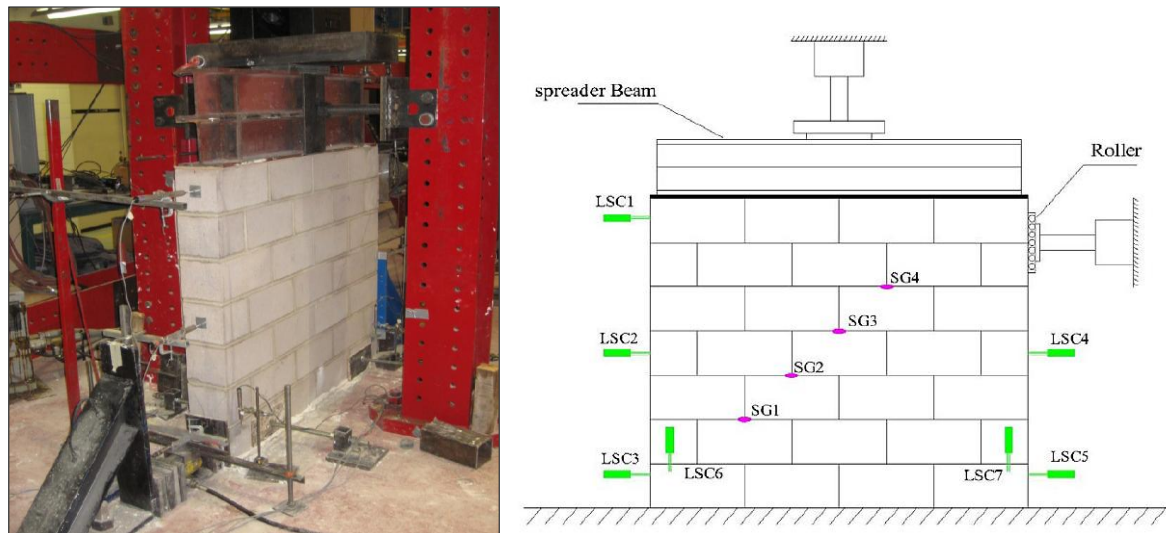
The axial stress level significantly influenced the behavior of the walls. Higher axial stress levels improved shear strength by enhancing aggregate interlock and delaying failure. However, increased axial stress in shear-dominated walls led to brittle failure mechanisms, reducing ductility and energy dissipation. The mortar formulation significantly influenced the behavior of the PGRM walls tested in the experimental program. Two types of mortar were evaluated: Type S Portland Cement Lime (PCL) and Masonry Cement (MC). Walls constructed with PCL mortar, with a higher bond strength of 1.2 MPa, exhibited improved ductility and greater energy dissipation, especially under cantilever boundary conditions. In contrast, walls constructed with MC mortar, with a lower bond strength of 0.5 MPa, demonstrated reduced ductility and energy dissipation, resulting in an earlier onset of diagonal cracking and more brittle failure mechanisms. The boundary conditions further influenced the observed failure mechanisms. Walls with cantilever (single-curvature) conditions experienced localized compressive damage at the wall base under high drift levels, while walls with fixed-fixed (double-curvature) conditions developed sliding planes along the lower portion of the wall.

2.2.2.6 Oan and Shrive (2010)

Oan and Shrive (2010) conducted an experimental study to examine the effects of horizontal reinforcement, specifically bed joint reinforcement, vertical reinforcement, axial

stress, and their combinations on the shear behavior of PGRM walls. The study involved testing 42 walls submitted to monotonic in-plane loading, each measuring 1600 mm in length and 1400 mm in height. The arrangement of the wall specimens is illustrated in Figure 10.

Figure 10: Arrangement of tests performed by Oan and Shrive (2010).



Source: Oan and Shrive (2010).

The results showed that bed joint reinforcement did not enhance the shear strength of the walls. Instead, the reinforcement only became active after diagonal cracking, contributing to post-cracking ductility rather than initial strength. Vertical reinforcement alone provided limited improvements in ductility but did not significantly increase the shear strength unless anchored in a concrete base beam, allowing flexural resistance. The combination of horizontal and vertical reinforcement offered the best post-peak performance, showing enhanced ductility and less degradation in load capacity after cracking.

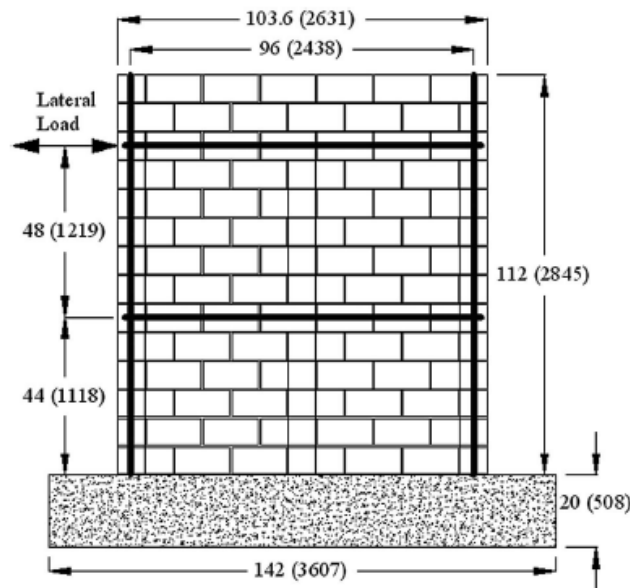
Axial stress was found to be a critical factor influencing shear resistance. Higher axial stress improved shear strength by enhancing friction and aggregate interlock along the cracking planes, delaying failure and altering the failure modes. Failure mechanisms transitioned from a combination of diagonal cracks across the walls and step cracking along the mortar joints at lower axial stresses to predominantly diagonal cracking at higher stresses.

2.2.2.7 Nolph et al. (2012)

Nolph et al. (2012) investigated the static cyclic response of five PGRM walls measuring 2845 mm x 2631 mm (Figure 11). The study examined the influence of different horizontal reinforcement ratios (0.085%, 0.120%, and 0.169%) placed within bond beams and

varying horizontal spacing between vertical grouted cells (1219 mm, 813 mm, and 610 mm) on the shear behavior of the walls.

Figure 11: Dimensions of tests performed by Nolph et al. (2012). Dimensions in inches (mm).



Source: Nolph et al. (2012).

The results demonstrated that increasing the horizontal reinforcement ratio from 0.085% to 0.120% resulted in an insignificant increase in lateral load capacity of approximately 2.7%. Furthermore, no additional improvement in lateral load capacity was observed when the reinforcement ratio was increased to 0.169%. Despite this ratio being double that of 0.085%, the corresponding lateral load capacity was only 92% of the strength achieved with the 0.085% reinforcement ratio.

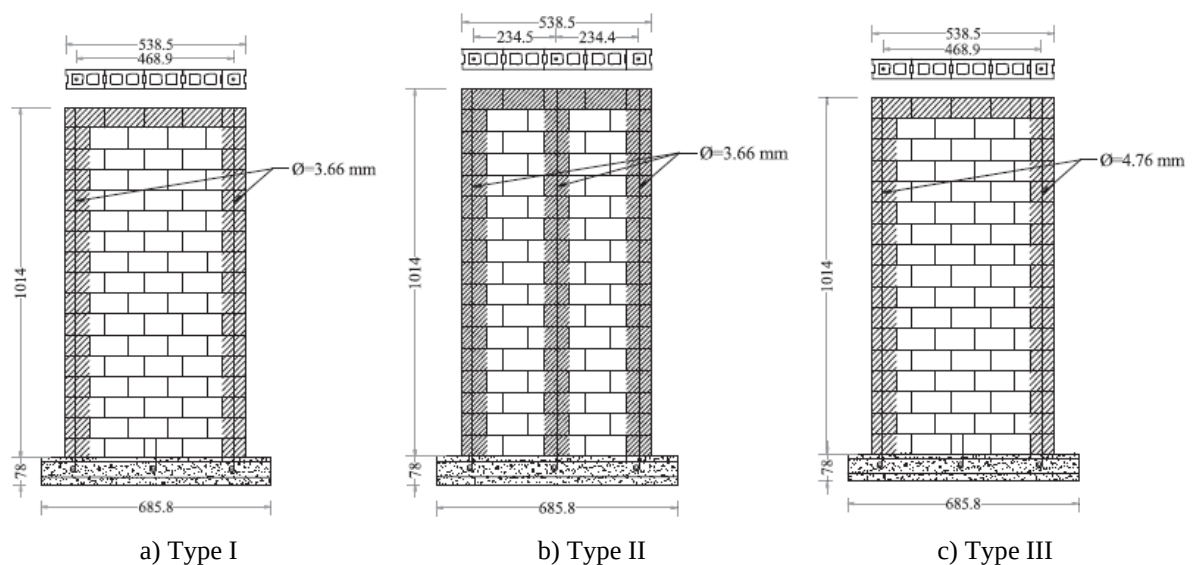
The horizontal spacing between vertical grouted cells had minimal impact on the deformability and displacement ductility of the tested specimens, as all specimens exhibited similar ductility levels. However, increasing the horizontal distance between grouted cells reduced the lateral load capacity of the walls. This reduction is primarily attributed to decreased net cross-sectional area and diminished confinement effects from the grouted regions.

The overall crack patterns of the tested specimens were similar, characterized by diagonal step cracks along the mortar joints. However, for specimens with a smaller horizontal distance between vertical grouted cells, the crack widths were smaller, and the number of cracks was significantly higher than those occurring in walls with a larger horizontal distance between vertical grouted cells.

2.2.2.8 Kasparik et al. (2014)

Kasparik et al. (2014) conducted an experimental program to assess the performance of PGRM walls under in-plane seismic loading on a shake table. Five reduced-scale wall specimens (30% scale) measuring 1014 x 538.5 mm² were tested under in-plane loading using the 1940 El Centro earthquake (N-S component) input motion. The walls were divided into three types based on reinforcement configuration. Type I and III walls were only reinforced at the end cells, while Type II walls included additional reinforcement at the mid-length (Figure 12).

Figure 12: Dimensions and reinforcement arrangements of tests performed by Kasparik et al. (2014).
Dimensions in mm.



Source: Kasparik et al. (2014).

The results demonstrated that reinforcement configuration significantly influenced the walls' strength and ductility. With a more uniform reinforcement distribution, Type II walls achieved the highest lateral load capacity due to the additional anchorage provided by the mid-length bar. Type I and Type III walls, reinforced only at the end cells, exhibited similar behavior, with Type III walls achieving slightly higher lateral displacements and load capacities due to their larger reinforcement diameter.

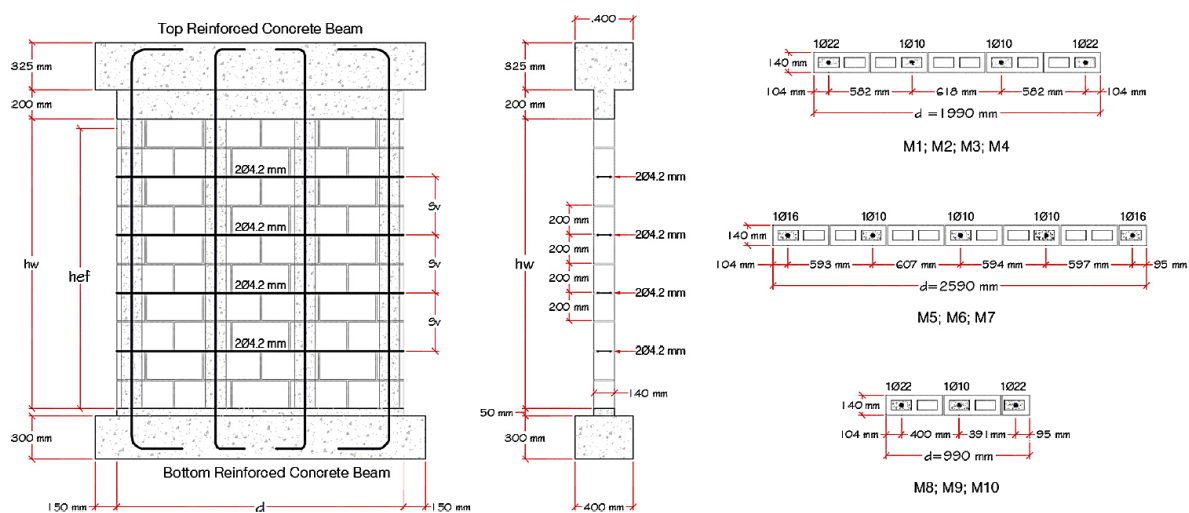
All walls exhibited a restrained-rocking mechanism, characterized by rotation about the compression toe while restrained by tension reinforcement at the end cells. This behavior produced horizontal cracks concentrated at the wall-foundation interface. Notably, no crushing,

sliding, or diagonal shear cracks were observed, highlighting the stability and ductility of the restrained-rocking response.

2.2.2.9 Ramirez et al. (2016)

Ramirez et al. (2016) conducted an experimental study on the cyclic in-plane response of ten full-scale PGRM walls, categorized by aspect ratio into short (0.44), ordinary (0.97), and slender walls (1.95). The arrangement of the tested specimens and their design details are in Figure 13.

Figure 13: Arrangement of the walls tested by Ramirez et al. (2016).



Source: Ramirez et al. (2016).

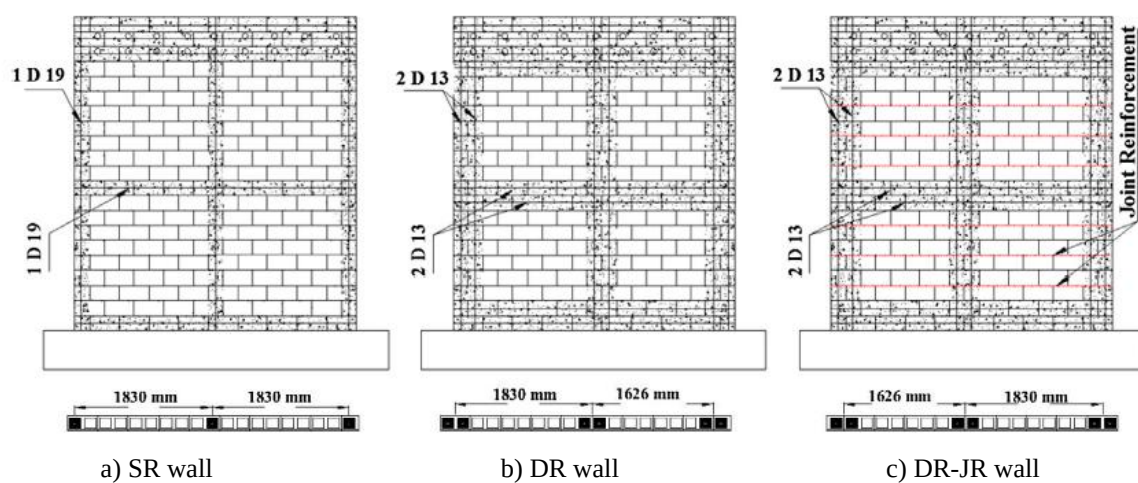
The experimental results revealed that the walls were predominantly governed by shear failure mechanisms characterized by diagonal cracking. Aspect ratio played a significant role in lateral strength, with lower aspect ratios (short walls) exhibiting higher strength than slender walls. Horizontal reinforcement ratios enhanced shear strength and energy dissipation, but excessive horizontal reinforcement reduced ductility. Vertical reinforcement primarily improved post-cracking performance by limiting diagonal cracking and sliding, though its impact on ductility was minimal. Axial pre-compression significantly increased shear strength, particularly in short walls, but it also reduced ductility and led to more brittle failure modes.

The study underscored the complex interplay between reinforcement, aspect ratio, and axial load, emphasizing that reinforcement configuration should be optimized to balance strength, ductility, and energy dissipation in seismic design.

2.2.2.10 Bolhassani et al. (2016c)

Bolhassani et al. (2016c) investigated the seismic performance of three full-scale PGRM walls to assess the effects of reinforcement detailing on cyclic lateral in-plane behavior. The wall specimens, each measuring 3860 x 3860 mm, were categorized as follows: a) the SR wall, with single-grouted cells and bond beams; b) the DR wall, with double-grouted cells; and c) the DR-JR wall, incorporating bed-joint reinforcement in addition to double-grouted cells. The arrangement and design details of the walls are illustrated in Figure 14.

Figure 14: Arrangement of the walls tested by Bolhassani et al. (2016c).



Source: Bolhassani et al. (2016c).

The study revealed distinct differences in failure mechanisms and performance among the configurations. The SR wall exhibited a shear failure mode, characterized by diagonal cracking and toe crushing, indicative of brittle behavior. In contrast, the DR and DR-JR walls demonstrated flexural failure modes, with distributed cracking and enhanced post-cracking performance. The DR wall showed significant improvements over the SR wall, with a 34% increase in shear strength, a 47% rise in displacement ductility, and a 60% enhancement in elastic stiffness.

Although the DR-JR wall displayed a more uniform crack distribution due to the bed-joint reinforcement, this feature did not result in notable improvements in strength or ductility compared to the DR wall. The results highlight the benefits of double-grouted cells in shifting the failure mode from shear to flexural behavior while enhancing overall seismic performance.

2.2.2.11 Fortes (2017)

Fortes (2017) conducted an experimental study on the lateral behavior of three-storey PGRM walls with openings under cyclic in-plane loading, as shown in Figure 15. Nine half-scale wall assemblies were tested, incorporating door or window openings, with coupling provided by either reinforced masonry or reinforced concrete beams. The walls were constructed using $185 \times 90 \times 90 \text{ mm}^3$ hollow concrete blocks and were vertically reinforced at the ends of piers and adjacent to openings, following typical Brazilian detailing practices.

Figure 15: Arrangement of the walls studied by Fortes (2017).



Source: Fortes (2017).

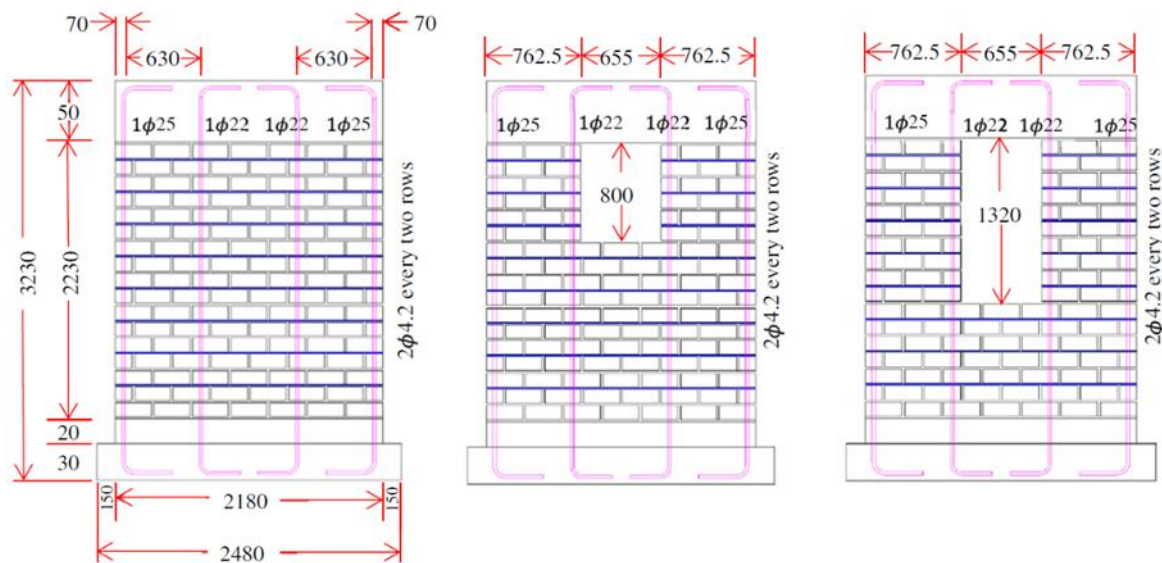
The results indicated that all tested walls exhibited a gradual degradation of strength without sudden failure, demonstrating a ductile response. The coupling beams contributed to a frame-like behavior, enhancing displacement capacity. No significant differences were observed in lateral displacement or maximum lateral load capacity between walls coupled with masonry or concrete beams, nor between walls with door or window openings.

The study confirmed that the PGRM shear walls with openings, regardless of coupling type, behaved as masonry frames and provided stable performance under lateral loads. These findings highlighted the role of coupling beams in improving the seismic performance of concrete masonry structures.

2.2.2.12 Calderón et al. (2017, 2019a, 2021a)

Calderón et al. (2019a; 2017) conducted an experimental study on the seismic response of PGRM walls made of clay blocks and submitted to cyclic lateral loading. Their research focused on the influence of central window-type openings on wall behavior. Three full-scale walls were studied: one solid wall and two perforated walls with consistent external dimensions but varying window heights, as depicted in Figure 16.

Figure 16: Arrangement of the walls studied by Calderón et al. (2017, 2019a). Dimensions in mm.

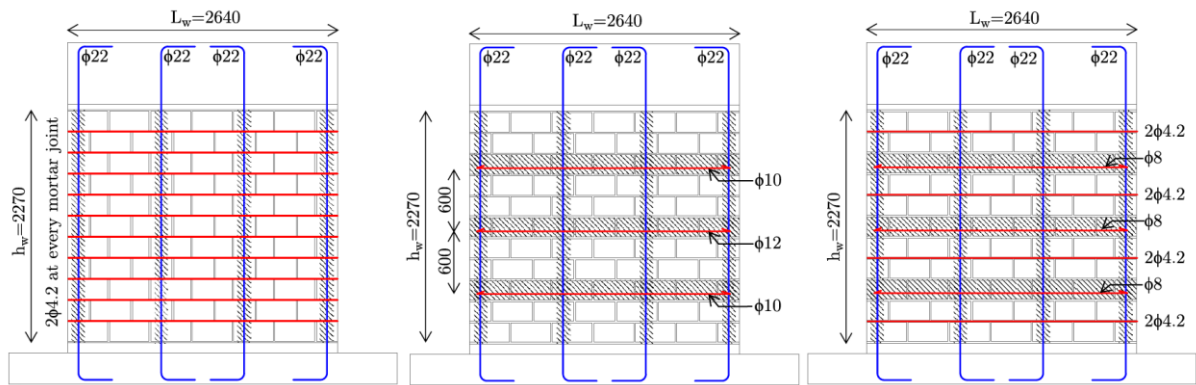


Source: Calderón et al. (2017, 2019a)

The results indicated that central openings significantly affected wall performance, reducing shear strength and stiffness compared to solid walls. Walls with openings exhibited more significant lateral stiffness degradation and distinct crack propagation patterns. Specifically, cracks in walls with openings tended to concentrate around the corners of the openings and along the piers rather than developing uniformly as in solid walls. It was also observed that varying the height of the openings while maintaining the same length had little effect on resistance capacity.

More recently, Calderón et al. (2021a) evaluated the influence of the type of horizontal reinforcement scheme on the in-plane cyclic behavior of PGRM walls. Four full-scale walls, each measuring 2640 mm in length, 2270 mm in height, and 140 mm in width, were tested under cyclic lateral displacements (Figure 17). The walls were constructed with a high vertical reinforcement ratio to induce shear failure modes, while the horizontal reinforcement ratio and axial load were kept constant across all specimens to isolate the effects of horizontal reinforcement detailing.

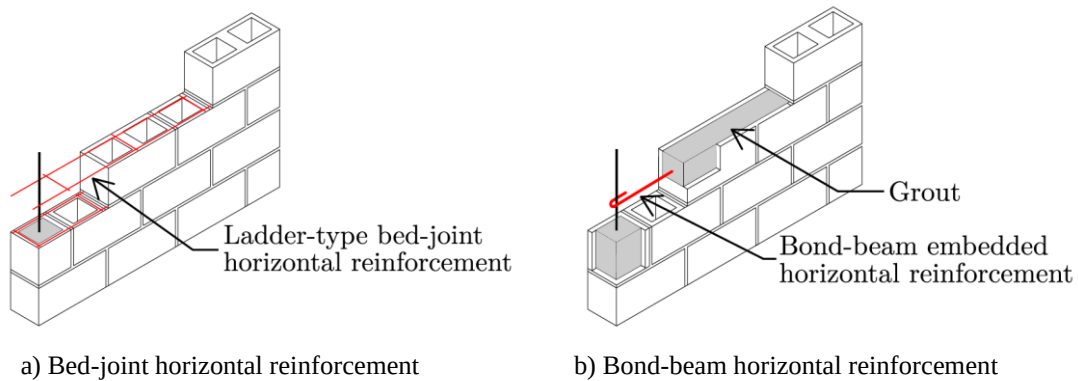
Figure 17: Arrangement of the walls tested by Calderón et al. (2021a). Dimensions in mm.



Source: Calderón et al. (2021a).

The horizontal reinforcement layouts studied included bed-joint reinforcement (Figure 18a), where ladder-type steel was embedded in every mortar bed joint; bond-beam reinforcement (Figure 18b), where horizontal bars were grouted into bond beams spaced every three courses; and a combined scheme incorporating both bed-joint and bond-beam reinforcements.

Figure 18: Types of horizontal reinforcement scheme.



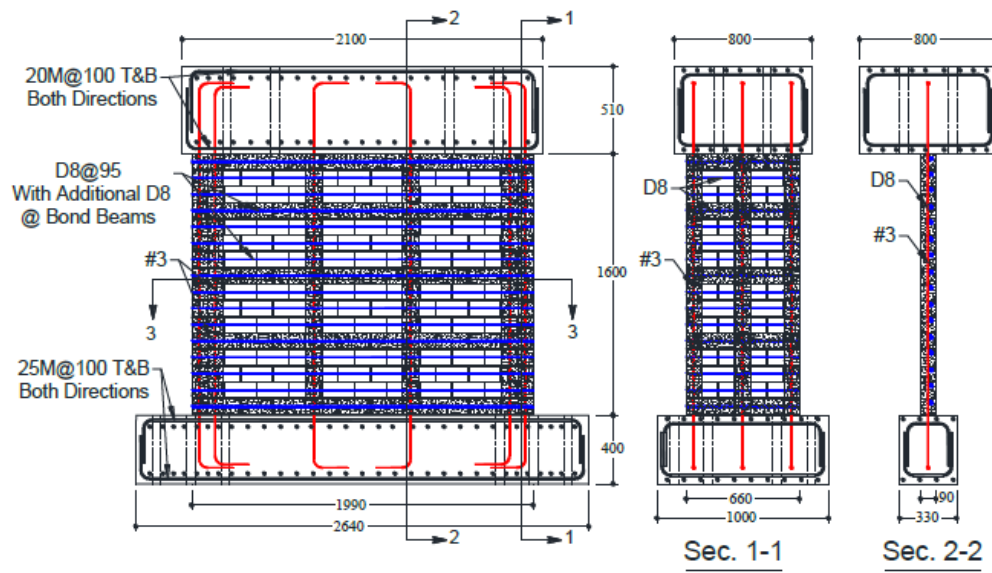
Source: Calderón et al. (2021a).

The results showed that the type of horizontal reinforcement layout had minimal influence on lateral capacity. However, the reinforcement scheme significantly influenced crack control, energy dissipation, and ductility. Walls with bed-joint reinforcement exhibited narrower and more distributed cracks, whereas bond-beam reinforcement resulted in localized cracks around the bond beams, leading to larger crack widths and reduced ductility. The combined reinforcement scheme performed best, achieving superior energy dissipation, ductility, and uniform crack distribution.

2.2.2.13 Elmeligy et al. (2025)

More recently, Elmeligy et al. (2025) investigated the seismic performance of slender flanged PGRM walls, focusing on the effects of varying shear span-to-depth ratios ($M/(V \cdot d)$) on in-plane cyclic behavior. This structural parameter describes the relationship between flexural moments, M , and shear forces, V , in a wall, normalized by the effective depth of the walls. Two half-scale walls, representing four- and eight-storey configurations in full-scale equivalents, were tested under constant axial stress and quasistatic cyclic loading. The walls, each measuring 1600 mm in height and 1900 in length (Figure 19), were designed to fail in flexure.

Figure 19: Arrangement of the walls tested by Elmeligy et al. (2025). Dimensions in mm.



Source: Elmeligy et al. (2025)

Wall W1, with a lower $M/(V \cdot d)$ ratio of 2.54, exhibited shear-dominant behavior, characterized by earlier and more distributed cracking throughout the web and flanges. This resulted in gradual shear-related damage and stiffness degradation, leading to a less stable response. In contrast, Wall W2, with a higher $M/(V \cdot d)$ ratio of 5.08, demonstrated a flexural-dominant response. The higher ratio delayed damage onset, resulting in limited cracking in both the web and flanges until advanced loading stages, where localized crushing near the flange bases marked failure. This behavior enhanced displacement ductility, better energy dissipation, and more stable hysteretic damping characteristics.

The flanges were critical in the observed damage progression and failure mechanisms. Horizontal cracks formed along the bed joints of the flanges, concentrating near the lower sections, while vertical cracks at the flange bases indicated localized compressive stresses. In

Wall W2, the flanges delayed cracking and distributed damage more uniformly, contributing to the wall's stability and flexural performance. In contrast, Wall W1 experienced more pronounced and widespread damage in the flanges and web, consistent with its shear-dominant behavior.

2.2.3 Perspective on complete building systems

Masonry walls are known to exhibit greater resistance to forces acting in-plane compared to those acting out-of-plane. To ensure adequate seismic performance of masonry buildings, it is essential to transfer seismic forces to the walls through a sufficiently rigid diaphragm. This allows the in-plane resistance and deformation capacity of the walls to be fully developed, mitigating the risk of out-of-plane collapse.

The masonry building floors and their connections to walls significantly influence overall structural behavior. Poor or inadequate connections between masonry panels and flexible floors can greatly reduce wall strength, as the walls may predominantly experience out-of-plane forces, their most vulnerable direction. Conversely, efficient connections between walls and floors, enabling them to act as rigid diaphragms, can substantially improve seismic performance. Such connections help prevent out-of-plane failure and enhance the development of flexural and shear resistance mechanisms in the walls' plane (Alves, 2011).

Structural masonry buildings must meet several minimum requirements related to material properties, geometry, structural configuration, and construction practices. Magenes (2006) and Morandi (2006) highlight the following critical points:

- Masonry units must be regular, robust, and exhibit good adhesion and interlocking to ensure monolithic behavior.
- Wall slenderness must be limited to reduce the risk of out-of-plane failure.
- Effective interlocking at wall intersections, along with connecting elements such as bond beams, ring beams, or ties, is essential to promote a rigid diaphragm effect at floor levels (box behavior). This is a fundamental assumption in seismic assessment procedures based on in-plane wall response.
- Floors must be sufficiently rigid and strong to distribute not only vertical forces but also horizontal forces to structural walls, control out-of-plane wall vibrations, and increase structural redundancy.

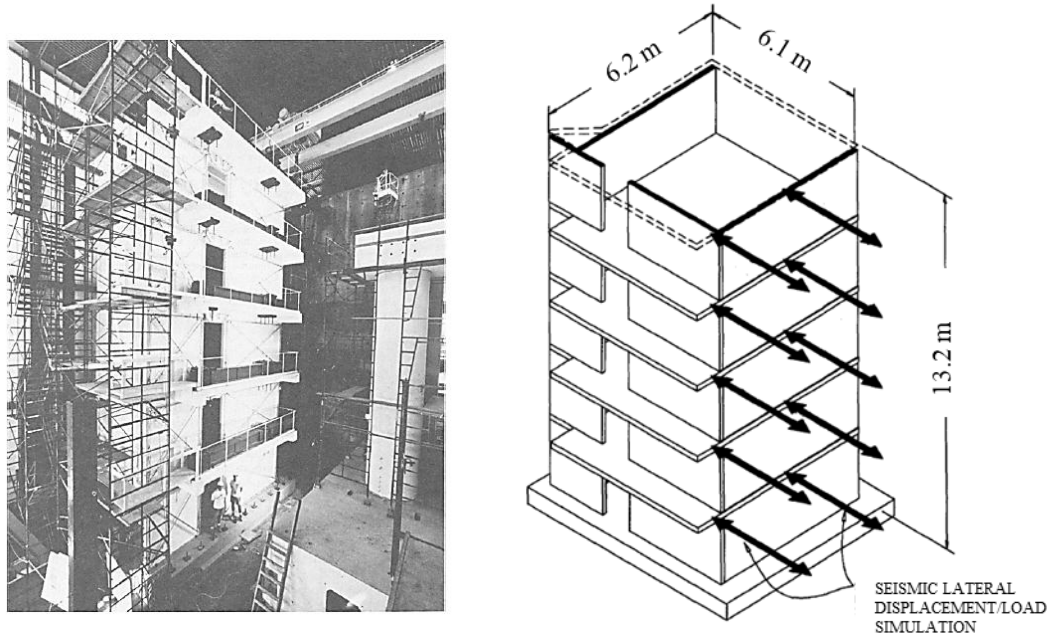
- Walls should be regularly spaced with sufficient density in both principal directions of the building to provide adequate resistance to loads in any direction.
- Mass and weight distribution should produce low stress-to-strength ratios.
- Regular structural configurations with adequate torsional resistance are essential. Highly asymmetrical arrangements, which can result in complex torsional effects, should be avoided.

Furthermore, masonry buildings can vary significantly in their structural layout configurations, incorporating walls with uniform or diverse characteristics (e.g., cross-sectional shapes, lengths, aspect ratios, axial load levels, and reinforcement detailing). These variations introduce complexity in seismic response, as the overall behavior of a building with interlocked walls often deviates from the simple sum of individual wall responses. This section reviews experimental studies focusing on the behavior of entire building systems rather than isolated wall components.

2.2.3.1 Seible et al. (1994a, 1994b)

Seible et al. (1994a; 1994b) investigated the seismic performance of a full-scale, five-storey FGRM building through simulated seismic load tests. The building dimensions were approximately 6200 mm x 6100 mm in plan, with a total height of 13200 mm, as shown in Figure 20. The building featured coupled flanged RM walls and precast prestressed hollow-core slabs, representing a subsection of a prototype structure designed for seismic conditions.

Figure 20: Arrangement of the building tested by Seible et al. (1994a, 1994b). Dimensions in m.



Source: Seible et al. (1994a, 1994b)

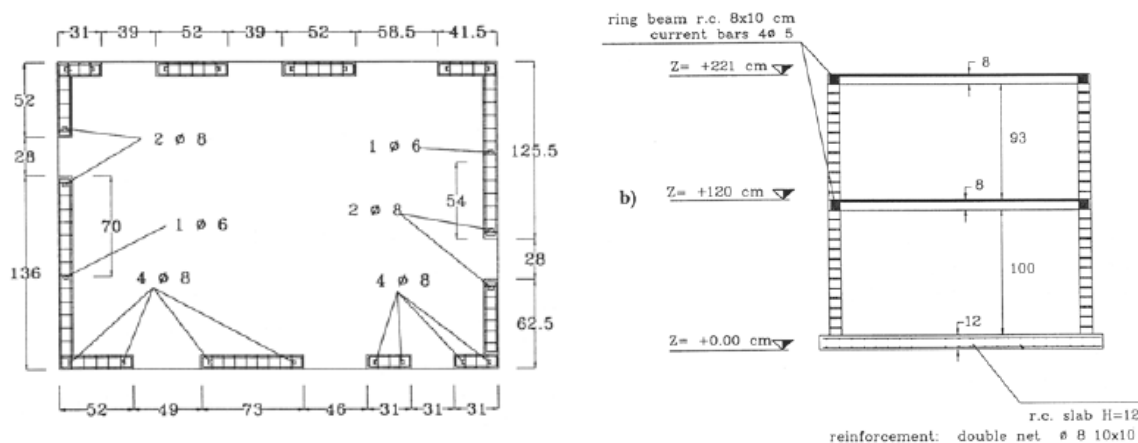
The coupling elements significantly influenced the lateral load resistance of the building. The floor slabs acted as horizontal diaphragms and coupling elements, transferring shear forces between the flanged masonry walls. This coupling effect enhanced the seismic performance by connecting the walls into a unified lateral force-resisting system. The slabs were designed to effectively distributing seismic forces across the structure. Approximately 50% of the overturning moment was resisted by a force couple generated by the interaction between axial forces in the flanged walls and shear forces transferred through the coupling slabs. This mechanism contributed significantly to the building's lateral load resistance and stability.

The seismic response was dominated by first-mode deformation, with maximum drift ratios of 1.5% in the pull direction and 1.0% in the push direction. Observed damage patterns included toe crushing and sliding shear at the base of the walls under extreme lateral loads. Walls orthogonal to the loading direction exhibited continuous horizontal cracks, functioning as tension flanges that contributed to the capacity of the loaded walls. Despite localized failures, the structure maintained its lateral strength with minimal degradation, demonstrating substantial displacement ductility and the ability to sustain significant inelastic deformations without collapse.

2.2.3.2 Zonta et al. (2001)

Zonta et al. (2001) investigated the seismic performance of a 1:3 scale RM building measured around 3130 mm x 2160 mm in plan with a height of 2210 mm (Figure 21). Reduced-scale testing provides a practical means of overcoming limitations in test facilities while capturing the overall behavior of structural systems under seismic loading. However, such testing involves inherent limitations. Achieving dynamic similarity between scaled models and full-scale structures is particularly difficult, especially in the presence of nonlinear behavior, due to challenges in scaling mass, stiffness, time, and material properties consistently. Consequently, scale effects must be carefully considered when interpreting the experimental results and extrapolating them to full-scale applications.

Figure 21: Layout of the building tested by Zonta et al. (2001). Dimensions in mm.



Source: Zonta et al. (2001).

The building exhibited flexural-dominated behavior under lateral loading, with plastic hinges forming at the bases of the piers and at the pier-edge beam joints. Horizontal reinforcement effectively prevented significant diagonal cracking. Damage progressed gradually through spalling and flexural cracking, which intensified at higher accelerations. Despite localized damages, the structure demonstrated satisfactory seismic performance, sustaining multiple simulated earthquakes without significant stiffness or strength degradation. The ability to dissipate energy through controlled flexural mechanisms highlighted the importance of ductile detailing.

2.2.3.3 Alves (2011)

Alves (2011) investigated the seismic performance of two reduced-scale (1:2), two-storey concrete masonry building prototypes through experimental shaking table tests: one

URM building and the other PGRM building. The PGRM building incorporated vertical steel bars placed in grouted cells and ladder-type horizontal reinforcement embedded in the bed joints.

The study focused on the influence of reinforcement on the seismic behavior of regular concrete masonry buildings. The PGRM building demonstrated significantly superior seismic performance compared to its URM counterpart. While the PGRM structure withstood seismic intensities up to 400% of the reference ground motion for a moderate seismicity zone, the URM model failed at 250%. At 250% PGA, the reinforced masonry exhibited only minor cracks, whereas the unreinforced masonry suffered substantial cracking and partial loss of stability (Figure 22).

Figure 22: Damage patterns observed at the ultimate seismic intensity phase.



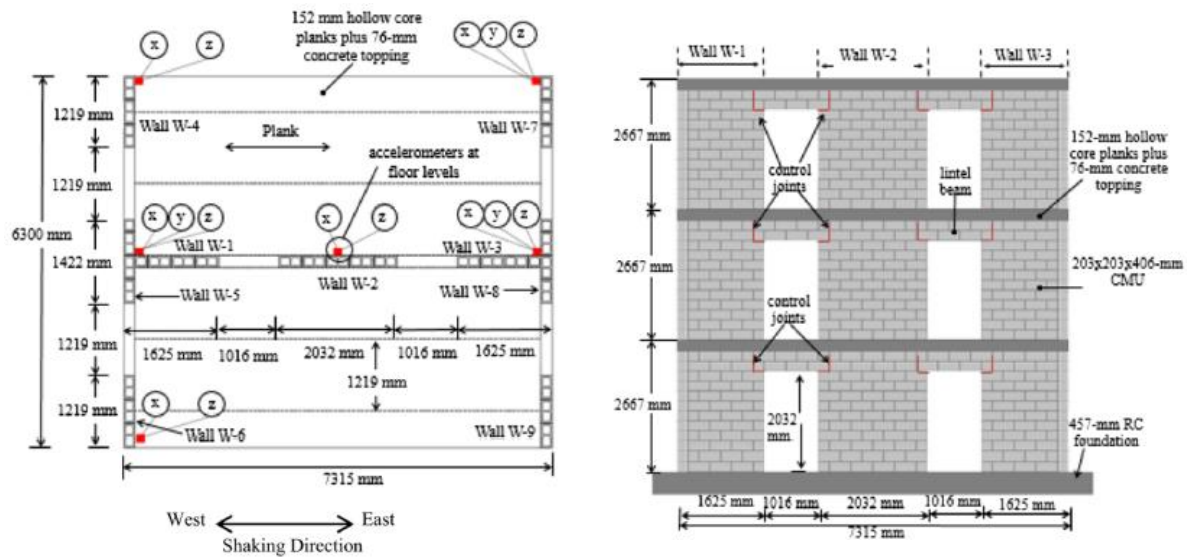
Source: Alves (2011).

2.2.3.4 Heerema et al. (2015)

Heerema et al. (2015) analyzed the seismic performance of a two-storey, third-scale PGRM building with asymmetric wall configurations under quasi-static cyclic loading. The building system, measuring 2160 mm in height with a 3000 mm x 3000 mm footprint, comprises eight RM shear walls with varying lengths, cross-sectional shapes, and reinforcement details. Slotted floor slabs were incorporated to minimize wall coupling, which limited shear force transfer between walls, allowing torsional responses induced by eccentric loading to dominate (Figure 23).

The study investigated the relationship between individual wall behavior and the system-level dynamics of RM buildings under asymmetric configurations. During the elastic phase, walls aligned with the loading direction provided the primary lateral resistance. As inelastic deformation progressed, stiffness degradation in these walls caused resistance demands to shift to orthogonal walls, amplifying torsional effects. Flanged walls, such as Wall

Figure 24: Layout of the building tested by Stavridis et al. (2016). Dimensions in mm.



Source: Stavridis et al. (2016).

The dynamic testing demonstrated a progressive increase in the fundamental period, T , from 0.09 s in its undamaged state to 0.25 s in its severely damaged state, highlighting the impact of cumulative damage on structural stiffness.

Horizontal diaphragms and lintels played a crucial role in the building's performance, acting as coupling elements to distribute lateral forces and enhance energy dissipation. The floor diaphragms, composed of precast prestressed hollow-core planks with concrete topping, effectively connected the walls, enabling the structure to behave as an integrated lateral force-resisting system. The building achieved a base shear capacity of approximately 1.35 times its seismic weight, significantly exceeding the design base shear coefficient, C_h , of 0.2. This coefficient represents the portion of a building's seismic weight expected to be resisted as lateral force during an earthquake, defined as $C_h = V/W$, where V is the base shear and W is the seismic weight. According to Stavridis et al. (2016), the full-scale masonry structure was designed for a base shear of 203 kN, corresponding to $C_h = 0.2$ for a total seismic weight of 1,024 kN. However, shake-table tests showed that the structure developed a base shear capacity of $1.35W$ —approximately 6.75 times greater than the design value. This substantial overstrength was primarily attributed to the coupling effects of floor diaphragms and lintels, which improved lateral load redistribution, as well as to inherent material and geometric reserve capacities not explicitly considered in the simplified design model.

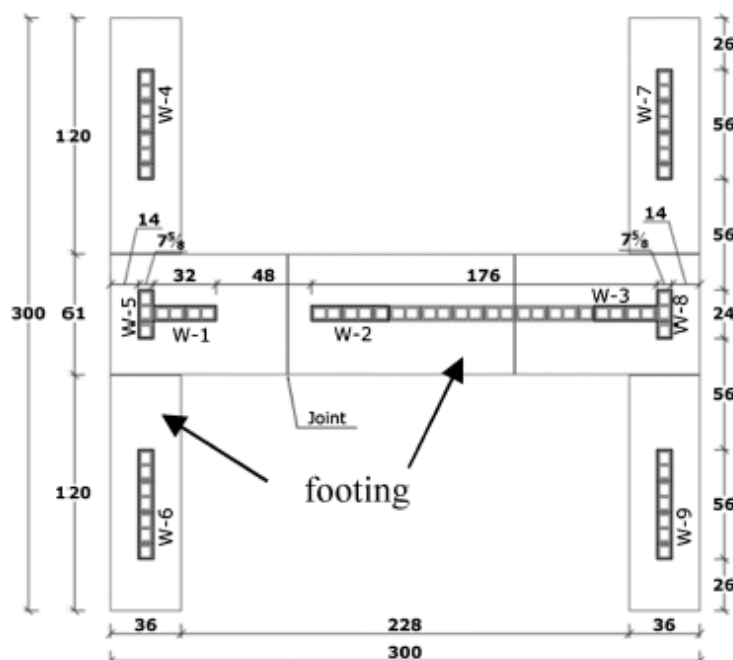
Initially, the seismic response was dominated by flexural behavior, transitioning to shear-dominated mechanisms as damage accumulated. This behavior was particularly evident in the in-plane walls, where diagonal shear cracks emerged at higher excitation levels. Despite

localized damage, including flexural cracking, toe crushing, and spalling at high seismic intensities, the building avoided collapse, demonstrating resilience and robustness.

2.2.3.6 Mavros et al. (2016)

Mavros et al. (2016) investigated the seismic performance of a full-scale, two-storey, FGFR building system, subjected to shake-table testing. The structure, designed to represent typical commercial or school buildings, featured one interior wall parallel to the shaking direction and two exterior walls perpendicular to it, incorporating rectangular and T-shaped wall sections, as illustrated in Figure 25.

Figure 25: Layout of the building tested by Mavros et al. (2016). Dimensions in cm.



Source: Mavros et al. (2016).

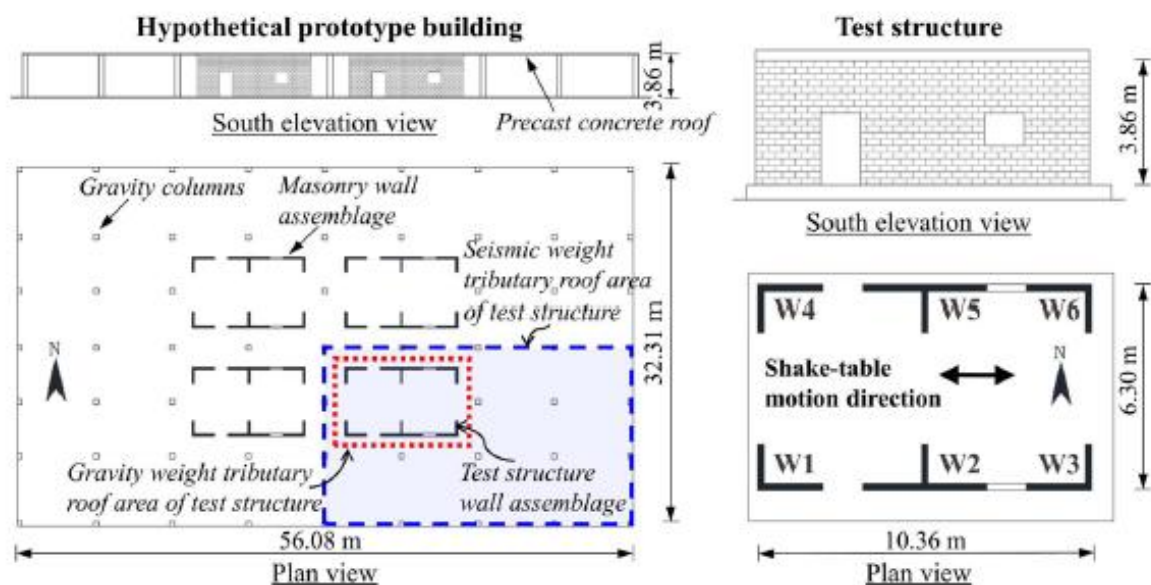
The building was subjected to a series of dynamic tests using scaled El Centro ground motion records, with intensities gradually increasing up to the maximum considered earthquake (MCE) level. Moderate ground motions (up to 86% of El Centro) caused minor cracking at wall openings, primarily flexural in nature. As seismic intensity escalated, diagonal cracking and masonry crushing emerged in the shear-dominated walls (W-2 and W-3), especially those with low aspect ratios. At the MCE level, the structure experienced maximum storey drifts of 1.9% in the first storey and 0.29% in the second, exceeding design displacement limits. Despite severe damage, the structure avoided collapse and maintained its stability.

The study emphasized the significant contribution of orthogonal walls to the lateral resistance of the system. Analysis indicated that their interaction with the in-plane walls, facilitated by horizontal diaphragms and lintels, increased base shear capacity by approximately 48%. This coupling action distributed lateral forces more effectively, enhancing energy dissipation and stabilizing the structure under extreme conditions.

2.2.3.7 Koutras and Shing (2020)

Koutras and Shing (2020) investigated the seismic behavior of a full-scale, one-storey PGRM building under shake-table testing. Designed for a moderate seismic zone in the U.S., the PGRM system measured 10360 mm and 6300 mm in plan with a wall height of 3860 mm. The building featured walls with door and window openings on both the north and south faces and incorporated four bond beams to enhance load transfer (Figure 26).

Figure 26: Layout of the building tested by Koutras and Shing (2020).



Source: Koutras and Shing (2020).

The building was subjected to a series of shake-table tests with increasing seismic intensities, including motions exceeding the Maximum Considered Earthquake (MCE) level. It achieved a base shear capacity 2.8 times the design base shear coefficient, showcasing significant overstrength. This overstrength was attributed to the coupling action of the bond beams with the walls, which helped distribute lateral forces more effectively across the structure and enhanced its overall seismic performance.

Cracking was initially concentrated in the mortar joints of ungrouted panels and around grouted cells, with bond beams yielding at extreme drifts. The centrally located T-walls experienced the highest seismic demands, playing a critical role in resisting lateral forces and maintaining structural stability. As seismic intensity increased, these walls exhibited the most significant cracking and damage, particularly at their intersections with bond beams. Out-of-plane walls contributed to overall stability by limiting excessive displacements.

Despite peak roof drift ratios reaching 2.25% under intense shaking, the structure avoided collapse, retaining 50% of its peak base shear capacity post-damage. This residual strength demonstrated the system's ability to sustain lateral loads even after significant damage, highlighting the robustness of properly detailed PGRM systems under severe seismic conditions.

2.3 NUMERICAL MODELING OF MASONRY WALLS

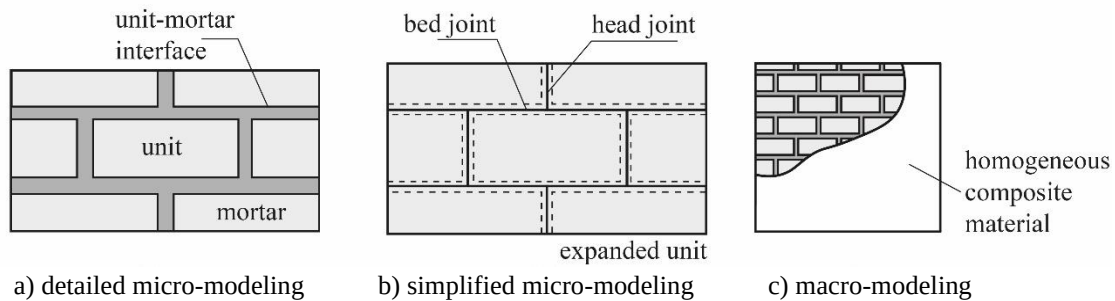
Numerical approaches for simulating the behavior of masonry structures are generally classified into micro- or macro-modeling based on the level of model detail. Micro-models focus on discretizing the masonry at the level of its constituent elements, enabling a more detailed understanding of localized behavior and potential damage mechanisms within the masonry components. In contrast, macro-models are designed to capture the overall (global) response of the structure by treating masonry as a homogeneous continuum. This approach simulates the behavior of masonry on a macroscopic scale, making it more suitable for analyzing the global performance of masonry structures (Haach; Vasconcelos; Lourenço, 2011; Lourenço, 1996).

Micro-modeling approaches can be further classified into detailed and simplified micro-modeling. In detailed micro-modeling, units, and mortar are represented as continuum elements, while the unit-mortar interface is modeled with discontinuous elements. Simplified micro-modeling, on the other hand, represents expanded units as continuum elements and lumps the behavior of mortar joints into discontinuous elements (unit-mortar interfaces). Essentially, the mortar joints and unit-mortar interfaces are replaced with zero-thickness interfaces while the units are expanded to preserve the geometry.

For macro-modeling, units, mortar, and unit-mortar interfaces are treated as a smeared continuum (Lourenço; Gaetani, 2022; Lourenço, 2002). The above concepts are shown schematically in Figure 27. Each modeling approach has its specific applications but simplified micro-modeling, and macro-modeling are often preferred due to their balance of accuracy and

efficiency, particularly in typical cases involving the seismic design of masonry buildings (Lourenço, 1996).

Figure 27: Modeling strategies for masonry structures.



Source: Lourenço and Gaetani (2022).

Simulating the nonlinear behavior of URM walls presents inherent challenges due to the intrinsic heterogeneity of the wall components. Researchers such as Shing and Cao (1997), Voon and Ingham (2006), Minaie et al. (2010), Bolhassani et al. (2016c), Shing et al. (1989; 1990) have identified mortar joints as inherent planes of weakness in ungrouted masonry regions. Consequently, to more accurately capture the behavior of masonry, numerical models must incorporate interface elements, as highlighted by Lourenço (1997). The complexity of simulating URM is further amplified for PGRM shear walls due to the non-constant cross-section, the non-homogeneous stress patterns generated by the unfilled hollow cores, and the presence of reinforcement that adds intricacy to an efficient simulation (Calderón et al., 2021b; Elmeligy; Aly; Galal, 2021).

In contrast, when all cells of a masonry wall are filled with grout, it mitigates the effects of masonry heterogeneity, resulting in a reduced influence of planes of weakness between the masonry units and mortar. In FGRM walls, diagonal tension cracks tend to propagate through the masonry blocks rather than along the mortar joints, demonstrating that the mortar joints have minimal impact on the shear strength of these walls. Therefore, the concepts employed in modeling RC structures can generally be applied to model FGRM shear walls (El-Dakhakhni; Banting; Miller, 2013; Elmeligy; Aly; Galal, 2021; Seif ElDin; Galal, 2017; Shedid; El-Dakhakhni; Drysdale, 2010a). By comparison, the behavior of PGRM walls is inherently more complex due to the combined effects of partial grouting, heterogeneous stress distributions, and the interaction between grouted and ungrouted regions (Koutras; Shing, 2021a; 2021b).

Several researchers (e.g., Arnau et al. (2015), Bolhassani et al.(2015), Calderón et al. (2021c), Koutras and Shing (2021b), and Shing et al. (1992)) have adopted different numerical modeling strategies to simulate the behavior of PGRM walls. These methods are mostly based

on developing numerical methods calibrated with experimental test results. The diversity in detail and accuracy observed in these numerical simulation approaches reflects the inherent complexity of numerically simulating PGRM walls.

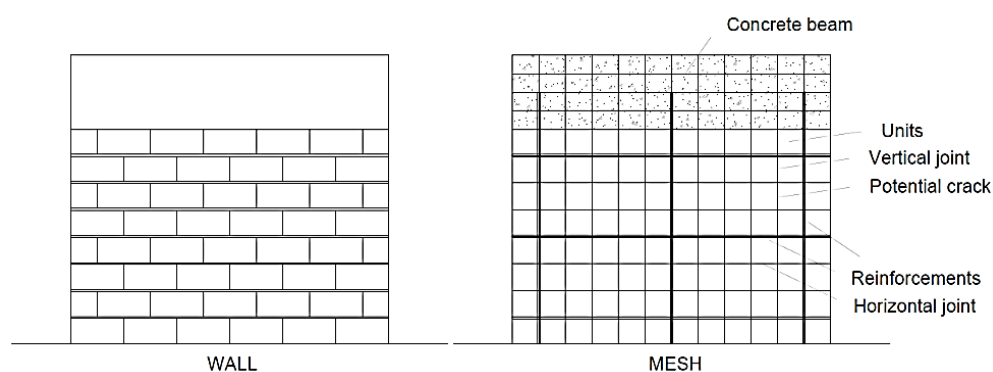
2.3.1 Shing and Cao (1997)

Shing and Cao (1997) employed two types of elements to model the behavior of masonry walls: smeared and interface elements. Smeared elements were used to model the behavior of masonry units, and an interface element with plastic behavior was used to model the tensile and shear behavior of the mortar joints. Results showed that the walls' lateral strength predicted numerically was higher than those obtained experimentally. However, the failure mode of the walls was well captured in this model. Moreover, the results showed that the behavior of PGRM walls is similar to that of infill-reinforced concrete frames. It has been shown that the behavior of a PGRM wall is very similar to that of a concrete-infilled frame. The grouted masonry provides the frame action, while the ungrouted masonry acts like an infill panel. According to the authors, one of the challenges of the study was the lack of available information on the tensile and shear behavior of mortar joints. Also, the difficulty of accurately simulating the complex behavior of the shear walls in the numerical models was emphasized.

2.3.2 Haach et al. (2011)

Haach et al. (2011) presented a detailed numerical micro-model based on Lourenço and Rots (1997) to simulate the in-plane behavior of reinforced masonry (RM) shear walls subjected to lateral loading, as represented in Figure 28. This approach, developed initially by Lourenço and Rots (1997), employs a plane stress interface cap model enhanced with modern plasticity concepts. The model integrates a Coulomb friction law, a tension cut-off, and an elliptical compression cap to represent the behavior of unit-mortar interfaces.

Figure 28: Representation of wall and mesh adopted by Haach et al. (2011).



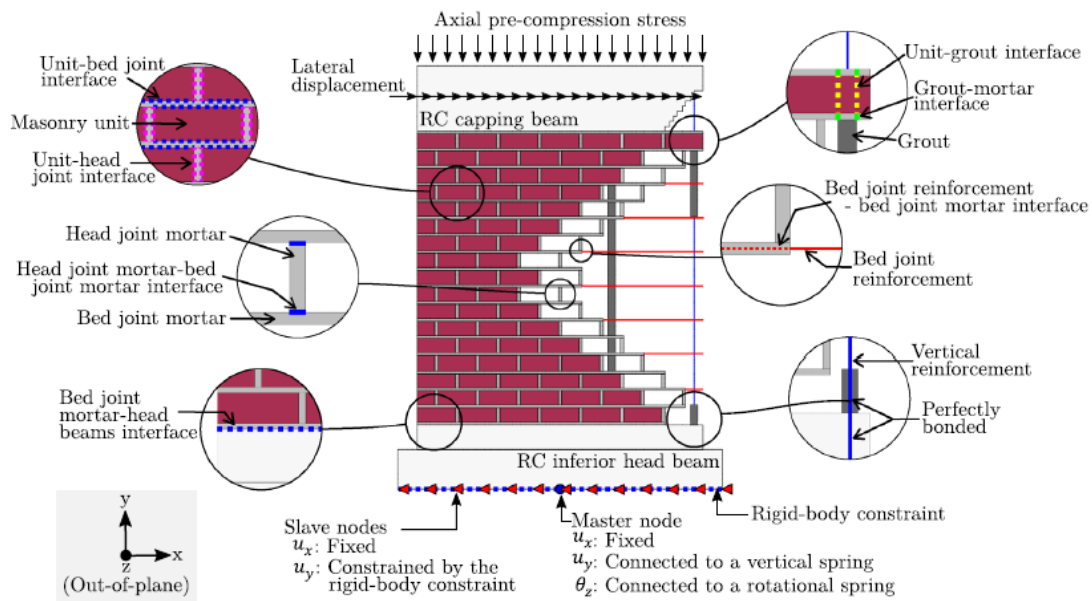
Source: Haach et al. (2011).

The goal was to accurately capture the local and global structural responses of masonry walls, including complex failure mechanisms such as flexural cracking, diagonal shear failure, sliding at mortar joints, and reinforcement interaction. Using the DIANA finite element (FE) software, the researchers adopted a micro-modeling approach, where the individual components of masonry walls — masonry units, mortar joints, and reinforcement — were explicitly modeled to reflect their distinct mechanical behaviors and interactions. The masonry units were represented using plane-stress elements to account for their elastic-plastic behavior under compression and tension. A total strain crack model was applied to simulate tensile cracking and compressive crushing, with a non-linear parabolic response governing compressive failure and an exponential softening curve for tensile behavior. Mortar joints, critical to masonry performance, were modeled using zero-thickness interface elements, which allowed for the simulation of shear slip and tensile separation. These interface elements utilized a Coulomb friction law with tension cut-off, capturing the reduction in strength as cracks developed and propagated through the wall. Truss elements were used for horizontal and vertical rebars embedded within the masonry to model reinforcement behavior. These reinforcement elements followed an elastic-perfectly plastic material model governed by the Von-Mises yield criterion, enabling the simulation of yielding and post-yield deformation under tensile loads. The interaction between masonry and reinforcement was assumed to be perfectly bonded, simplifying the modeling of load transfer between the materials. The models were validated with differences of less than 10% against the experimental data regarding the strength capacity of the walls.

2.3.3 Calderón et al. (2021c, 2021d)

Calderón et al. (2021c) presented a detailed micro-modeling approach (Figure 29) to simulate the in-plane lateral behavior of partially grouted reinforced masonry (PGRM) shear walls constructed with multi-perforated clay bricks (MPCLBs). Recognizing the complexity of PGRM walls, particularly due to their heterogeneous stress-strain distribution and localized failure mechanisms, this study developed and validated a refined finite element (FE) model that accurately captured the wall's global and local responses under seismic loading conditions.

Figure 29: Representation of detailed micro-modeling approach by Calderón et al. (2021c).



Source: Calderón et al. (2021c).

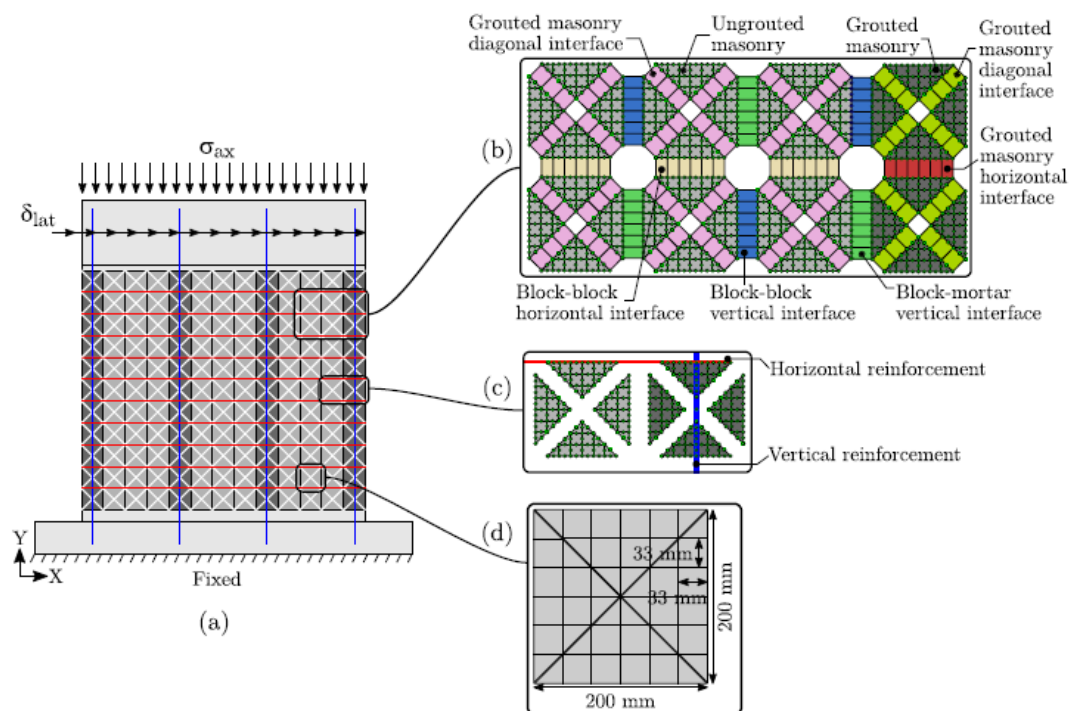
The proposed model adopted a 2D plane-stress framework implemented in the DIANA FE software. The geometry discretized bricks, mortar, and grout into eight-node quadrilateral elements, while the reinforcement was modeled using truss elements (horizontal bed-joint reinforcement) and beam elements (vertical rebars). Interface elements simulated critical contact zones between bricks and mortar, incorporating a Coulomb-based friction law with tension cut-off to reproduce shear sliding and crack propagation. This interface model also accounted for mortar spikes, representing the influence of mortar penetration into multi-perforated clay brick voids, significantly affecting shear behavior. The mortar spikes were incorporated by adjusting the shear and tensile strength of the interface to account for the additional shear transfer provided by the mortar penetrating into the brick voids. This ensured the model realistically simulated the enhanced shear capacity and improved crack resistance observed in partially grouted masonry walls constructed with multi-perforated clay bricks.

The model was validated using experimental data from seven full-scale PGRM shear walls tested under cyclic lateral loading. The walls featured variations in key parameters, including aspect ratio, mortar joint thickness, horizontal reinforcement ratio, and axial pre-compression stress. Validation results demonstrated the model's capability to reproduce load-displacement curves, including initial stiffness, peak strength, and post-peak degradation — crack patterns, such as diagonal shear cracking, mortar joint detachment, and sliding shear failures.

Recognizing the challenges and computational complexity involved in modeling PGRM walls, Calderón et al. (2021d) presented a simplified micro-modeling approach to accurately simulate the in-plane behavior of PGRM shear walls with bed-joint reinforcement under seismic loading aiming balances accuracy, computational efficiency, and practical implementation using the commercial finite element (FE) software, DIANA.

The masonry was discretized into 2D plane-stress triangular and quadrilateral elements, representing the grouted and ungrouted zones, with interface elements used to simulate cracks at mortar joints, vertical cracks within blocks, and diagonal shear cracks. Horizontal reinforcement, such as ladder-type bed-joint bars, was modeled with truss elements, while vertical reinforcement used beam elements with elastic-plastic behavior governed by the Von-Mises yield criterion. Grouted zones were idealized as grout columns with equivalent widths, reducing complexity while preserving the wall's heterogeneity. The equivalent width accounted for the actual width of the grout-filled hollow and the net contribution of the block's geometry to the wall's overall stiffness and strength. This adjustment ensured that the mechanical properties of the grouted regions, such as stiffness and strength, were accurately represented in a 2D model (Figure 30).

Figure 30: Representation of simplified micro-modeling approach by Calderón et al. (2021d). a) General overview; b) connectivity of nodes; c) connectivity of reinforcement; d) mesh size.



Source: Calderón et al. (2021d).

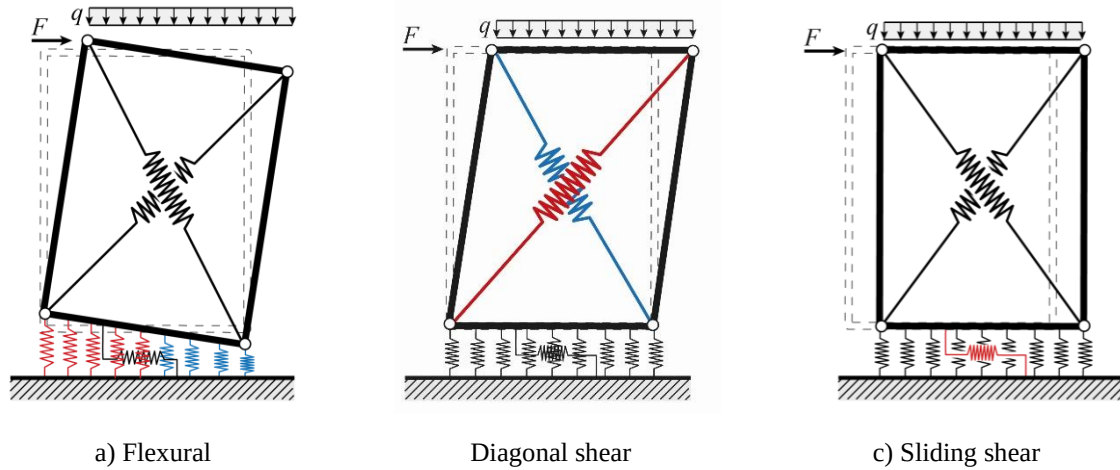
The study validated the model against experimental data from two full-scale PGRM shear walls tested under cyclic lateral loading. Despite being unable to reproduce the cyclic nature of loading fully, the model accurately predicted key system responses, including the lateral resistance and failure modes, such as diagonal shear cracking and sliding shear failure, crack patterns, highlighting the preferential propagation paths at mortar joints, and block interfaces.

2.3.4 Calìo et al. (2005)

As mentioned by Marques and Lourenço (2011), the need to consider the global response of masonry buildings has led many researchers to develop one-dimensional macro-elements to simulate a global response and then apply conventional structural mechanics methods (Magenes, 2006; Morandi et al., 2022; Morandi, 2006). Subsequently, Calìo et al. (2005) identified some limitations in using beam-type macro-elements, mainly due to the inaccurate simulation of the interaction between macro-elements and the poor modeling of the cracking state of the panels. These authors proposed returning to two-dimensional macro-elements, specifically using a set of nonlinear springs, an approach macro-elements incorporated into the 3DMacro computer code (Calìo et al., 2012). A schematization of how

the macro-element simulates typical failure modes of masonry walls is presented in Figure 31. It is worth noting that the model can simulate the combined development of such mechanisms.

Figure 31: Simulation of in-plane failure mechanisms of masonry walls using the macro-element



Source: Adapted from Calìo (2008)

The flexural mechanism can develop in two ways in the model, i.e., progressive tensile cracking at the heel leading to rocking and crushing of the masonry at the compressed edge (the toe). In the macro-model, rocking is associated with tension yielding of flexural springs (orthogonal to the interface), and masonry crushing is associated with progressive yielding by compression. Thus, the dependence on this mechanism's axial loading is implicitly considered.

2.4 RESPONSE MODIFICATION FACTOR FOR MASONRY STRUCTURES

2.4.1 Definition of the response modification factor

The nonlinear behavior of structural systems allows seismic forces, derived from an elastic response spectrum, to be reduced using a factor that accounts for nonlinearity. When explicit inelastic analysis is not performed, seismic verification can instead be carried out using linear elastic analysis based on a reduced response spectrum.

The literature reveals various terms for this same force reduction factor. For instance, EN 1998-1 (2004) defines it as the behavior factor, q , while ASCE SEI 7 (2022) and ABNT NBR 15421 (2023) refer to it as the response modification factor, R . In this text, for standardization purposes, this factor will be referred to as the response modification factor, R . Nonetheless, the behavior factor, q , is conceptually equivalent, as both represent the same underlying effect.

EN 1998-1 (2004) defines this factor as an approximation of the ratio between the seismic forces a structure would be subjected to if its response were entirely elastic and the minimum seismic forces that can be used in design based on a conventional elastic analysis model, while still ensuring satisfactory structural performance. Therefore, the application of the R factor should approximately encompass the effects of numerous factors contributing to the total nonlinear capacity of a structural system, such as deformation and energy dissipation capacity, overstrength, and even, as mentioned by NTC (2018), the increase in the natural period of the structure in the plastic zone, as outlined by Stavridis et al. (2016), and Mavros et al. (2016). In this context, the most recent drafts of Eurocode 8, such as the EN 1998-1 (2022) document, describe the R factor as the product of three components, as expressed in Equation 1.

$$R = R_d \cdot R_0 \cdot R_s \quad \text{Eq. 1}$$

Where:

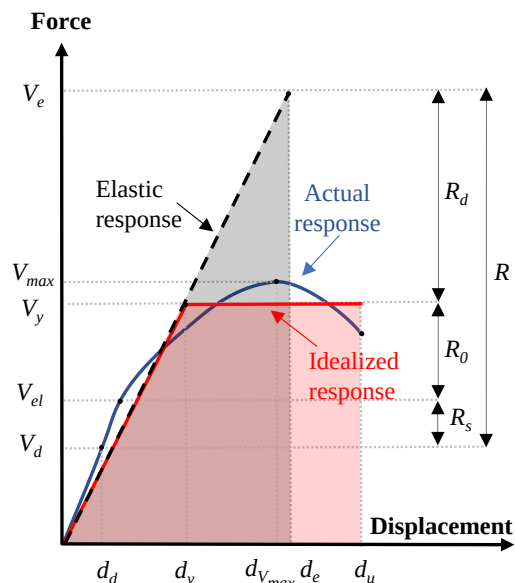
R_d is the component associated with deformation capacity and energy dissipation;

R_0 is the component associated with overstrength due to the redistribution of seismic action effects in redundant structures;

R_s is the component associated with overstrength due to all other sources.

Based on Tomazevic et al. (2004), Figure 32 illustrates the definition of these components using the example of an actual force-displacement curve, which could represent an experimental envelope curve or a pushover analysis. It also shows an equivalent idealization of the actual curve, in this case, a bilinear elastoplastic idealization, from which the three components of the response modification factor are derived. Additionally, it includes a perfectly elastic response with the same initial elastic stiffness as the idealized response. Further details about the components of the behavior factor are discussed in the following sections.

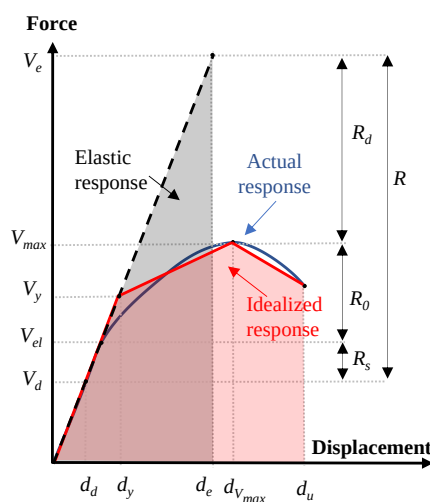
Figure 32: Definition of the components of the behavior factor.



Source: Adapted from Morandi et al. (2022).

It is worth noting that different strategies for idealizing the actual response have been used in scientific research (Banting; El-Dakhkhni, 2014; Bolhassani et al., 2016a; Calderón et al., 2021a; Morandi et al., 2022; Ramírez; Sandoval; Almazán, 2016; Sandoval; Calderón; Almazán, 2018; Seif Eldin; Ashour; Galal, 2019b). This is an important consideration, as different idealization strategies can lead to varying seismic parameter values for the same actual curve. In this context, Medeiros et al. (2022a) proposed a trilinear approach, as illustrated in Figure 33, which fits the actual response while preserving three essential characteristics: initial elastic stiffness, lateral force capacity, and the slope in the post-peak phase.

Figure 33: Trilinear idealization of the equivalent elastoplastic response.



Source: Adapted from Medeiros et al. (2022a).

2.4.1.1 Component due to ductility and energy dissipation capacity, R_d

As illustrated in Figure 32, the factor R_d is the ratio between the maximum seismic force the structure would be subjected to if its behavior were purely elastic, V_e , and the maximum force of the equivalent idealized curve, V_y . The typical criterion for evaluating the R_d component is given by Equation 2.

$$R_d = \frac{V_e}{V_y} \quad \text{Eq. 2}$$

The R_d factor can also be expressed in terms of the global ductility factor $\mu_d = d_u/d_y$, as shown by Tomazevic (2004), using Equation 3.

$$R_d = (2\mu_d - 1)^{1/2} \quad \text{Eq. 3}$$

Beyer and Magenes (2020) report that deformation capacity and energy dissipation depend on various characteristics of the masonry building, including the deformation capacity of each wall; the coupling of walls through slabs, bond beams, or beams; the distribution of wall lengths; and torsional effects.

2.4.1.2 Component due to overstrength, R_o

In linear elastic analyses, the "ultimate" limit state of the building is conditioned to the load level at which any wall in the building reaches its flexural or shear strength. This load level, defined here as V_{el} , generally does not correspond to the ultimate resistance capacity of the structural system, as it does not account for the redistribution of forces among walls, as illustrated in Figure 33.

Morandi et al. (2022) note that masonry walls possess a nonlinear deformation capacity that allows the building to withstand additional seismic loads beyond the "elastic" limit, V_{el} , by redistributing forces to other structural wall elements. Furthermore, as mentioned by Beyer and Magenes (2020), masonry buildings typically have many structural walls and are, therefore, highly redundant structures. As such, the overstrength resulting from structural redundancy, R_o , must be explicitly accounted for in masonry buildings, just as it is currently expressed for other structural typologies, such as reinforced concrete and structural steel, in EN 1998-1 (2004).

Magenes (2006) and Morandi (2006) initiated the discussion of these concepts applied to masonry buildings. Although it is a new aspect in Eurocode 8, the overstrength component

for masonry buildings is already employed in other regulatory recommendations, such as NBCC (2020) and NTC (2018).

For calculation purposes, the R_0 component, which accounts for redistributing seismic action effects in redundant structures, can be estimated from the bilinear elastoplastic idealization of the building's capacity curve, as expressed in Equation 4. In EN 1998-1 (2004) and NTC (2018), the relationship in Equation 4 is presented as α_u/α_1 , but it holds the same conceptual meaning.

$$R_0 = \frac{V_y}{V_{el}} \quad \text{Eq. 4}$$

It is worth noting that the redundancy of the structural system, and consequently the R_0 component, is sensitive to the building's structural configuration (in plan and elevation). Numerical studies have shown R_0 values ranging from slightly above 1.0 to approximately 4.0 (Magenes, 2006; Morandi, 2006). As expected, these analyses demonstrate that increasing the structural redundancy level of the system results in higher R_0 values. For example, buildings with walls of varying lengths in one direction are likely to exhibit elastic limit exceedance at different times, resulting in higher levels of overstrength than buildings where all walls in one direction have approximately the same length. Additionally, when there is a well-balanced distribution of walls with varying lengths, the greater the number of walls, the higher the potential for force redistribution and the greater the system's redundancy.

As reported by Magenes (2006), it seems appropriate for both R_d and R_0 to be defined as nationally determined parameters tailored to specific structural typologies. This underscores the importance of conducting analyses that reflect the diversity of masonry construction practices, as these can vary significantly between different countries and regions, potentially impacting seismic performance assessments.

When the level of overstrength is evaluated through nonlinear numerical analyses, it is expected that the assumptions made in the model will also influence the obtained response with varying degrees of sensitivity. In this regard, Morandi et al. (2022) and Magenes (2006) highlight the following aspects: modeling assumptions related to the coupling between shear walls (flexural coupling provided by slabs, beams, or masonry lintels); modeling assumptions related to the interaction between orthogonal walls (flange effects); and the resistance criteria adopted. The authors demonstrated that models considering the coupling action provided by slabs, beams, or lintels result in lower overstrength values than models where the coupling

action is neglected, such as cantilever wall models. This occurs because coupling actions lead to an increase or decrease in the axial force in the walls when the building is subjected to horizontal loading. Walls where the axial force decreases tend to reach their yield limit earlier than those where the axial force increases.

2.4.1.3 Component due to overstrength from other sources, R_s

According to Morandi et al. (2022), the R_s component can be understood based on the following aspects: overstrength due to the reduction of material strength from the mean value to design values, i.e., the contribution of material safety factors; effects that enhance resistance but are typically not considered in masonry building models, such as coupling and flange effects; and the potential underestimation of resistances when using analytical approaches in design.

Regarding material safety factors, Morandi et al. (2022) state that the reduction of material strength parameters does not result in a proportional reduction in the strength of a wall, as shear failures and flexural failures exhibit different sensitivities to material strength parameters. This difference is related to the design equations for M_d and V_d in the EN 1998-1 (2004), with flexural failure being less sensitive than shear failure. For this reason, if all the walls in a building fail due to flexure, the R_s component, resulting from overstrength due to material safety factors, should be approximately equal to one. However, assuming that, in a building, the system capacity is primarily affected by shear failures — typically associated with longer and stronger walls — a safe estimate for reducing material strength from the mean to the design value would range between 1.1 and 1.2.

Regarding the accuracy of analytical approaches, Morandi et al. (2018) demonstrated that the strength calculated using Eurocode 8 equations with mean material values shows good agreement compared to strengths obtained from experimental tests. However, as highlighted by Tomaževič (2007), shaking table tests suggest an overstrength of at least 30%, attributed to the underestimation of material strength. This overstrength explains discrepancies between design and experimental results, as analytical methods often use conservative safety factors.

Regarding the simplified modeling approach for masonry buildings, Morandi et al. (2022) report that the magnitude of these effects can vary significantly between buildings and models, but it can reasonably be assumed to range between 1.1 and 1.2. Lower values are applied when beam or floor coupling is explicitly modeled, and higher values when it is neglected. Based on these considerations, Morandi et al. (2018) suggest that R_s values in the

range of 1.35-1.5 are reasonable for unreinforced masonry buildings, depending on whether the flexural coupling provided by beams or floors is explicitly modeled. According to Beyer and Megenes (2020), a value of 1.35 is recommended for unreinforced masonry buildings, while, similar to other construction typologies, a value of 1.5 can be adopted for reinforced and confined masonry buildings. In the EN 1998-1 draft EN 1998-1 (2022), the R_s component is set to 1.5 for all construction configurations. However, the draft acknowledges unreinforced masonry walls as structural members without overstrength and, therefore, may warrant an exception to this rule.

As reported by Morandi (2006), a high level of structural irregularity can lead to a reduction of approximately 30% in the R factor. According to EN 1998-1 (2004), for buildings that are irregular in height, the R value is obtained by multiplying the reference value by 0.8.

2.4.2 Response modification factors for masonry structures in design standards

2.4.2.1 EN 1998-1 (2004)

The superior seismic performance of reinforced and confined masonry systems compared to unreinforced masonry systems is recognized in EN 1998-1 (2004). This standard provides a range of values for the component related to ductility and energy dissipation capacity, R_d , depending on the masonry typology. Note that this parameter is called behavior factor, q , in this design standard. The recommended values for masonry typologies are as follows:

- Unreinforced masonry: $R_d = 1.5 - 2.5$;
- Confined masonry: $R_d = 2.0 - 3.0$;
- Reinforced masonry: $R_d = 2.5 - 3.0$.

The variations reflect differences in the structural system's capacity to deform and dissipate energy under seismic loads, which are influenced by the assigned ductility class (low, medium, or high) and the level of construction detailing. Higher behavior factor values correspond to more stringent detailing requirements and superior construction practices, ensuring greater energy dissipation and deformation capacity.

This version of EN 1998-1 (2004) does not explicitly account for the overstrength resulting from structural redundancy, R_o . However, proposals by Bayer and Magenes (2020) suggest that future standard revisions may include updated seismic parameters for linear

analyses to better represent the contribution of redundancy and overstrength in masonry structures.

2.4.2.2 NTC (2018)

The Italian standard recommendations, NTC (2018), provides values for the ductility-related force modification factor, R_d , based on masonry typology:

- Unreinforced masonry: $R_d = 1.75$;
- Confined masonry: $R_d = 2.0$;
- Reinforced masonry: $R_d = 2.5$.

For both reinforced and confined masonry, the standard permits an R_d value of 3.0 if the design ensures flexural-controlled behavior (capacity design), where shear failure does not precede flexural failure. The overstrength component, R_0 , is also defined according to the masonry typology:

- Unreinforced masonry: $R_0 = 1.7$;
- Confined masonry: $R_0 = 1.6$;
- Reinforced masonry: $R_0 = 1.5$.

Additionally, the standard allows nonlinear static analysis to evaluate overstrength, though the maximum value of R_0 is capped at 2.5. Although it may seem counterintuitive for unreinforced masonry to have a higher R_0 , Magenes (2006) observed through numerical models that masonry buildings with minimal reinforcement (e.g., horizontal reinforcement ratio of 0.04%; vertical reinforcement ratio of 0.05%) exhibited higher levels of overstrength than fully reinforced masonry buildings. This is because reinforcement tends to increase V_{el} more significantly than V_y .

For regular masonry buildings with structural redundancy, the combined response modification factor, $R = R_d \cdot R_0$, can reach 3.75 or even 7.5 (when considering the capacity design and incorporating maximum overstrength through nonlinear static analysis) for reinforced masonry buildings. In comparison, unreinforced masonry buildings achieve approximately 3.0 or 4.4 under similar conditions for unreinforced masonry buildings.

2.4.2.3 ASCE SEI 7 (2022)

The ASCE SEI 7 (2022) categorizes five different masonry systems based on the level of detailing required. Each system is assigned a specific response modification factor, R , as follows:

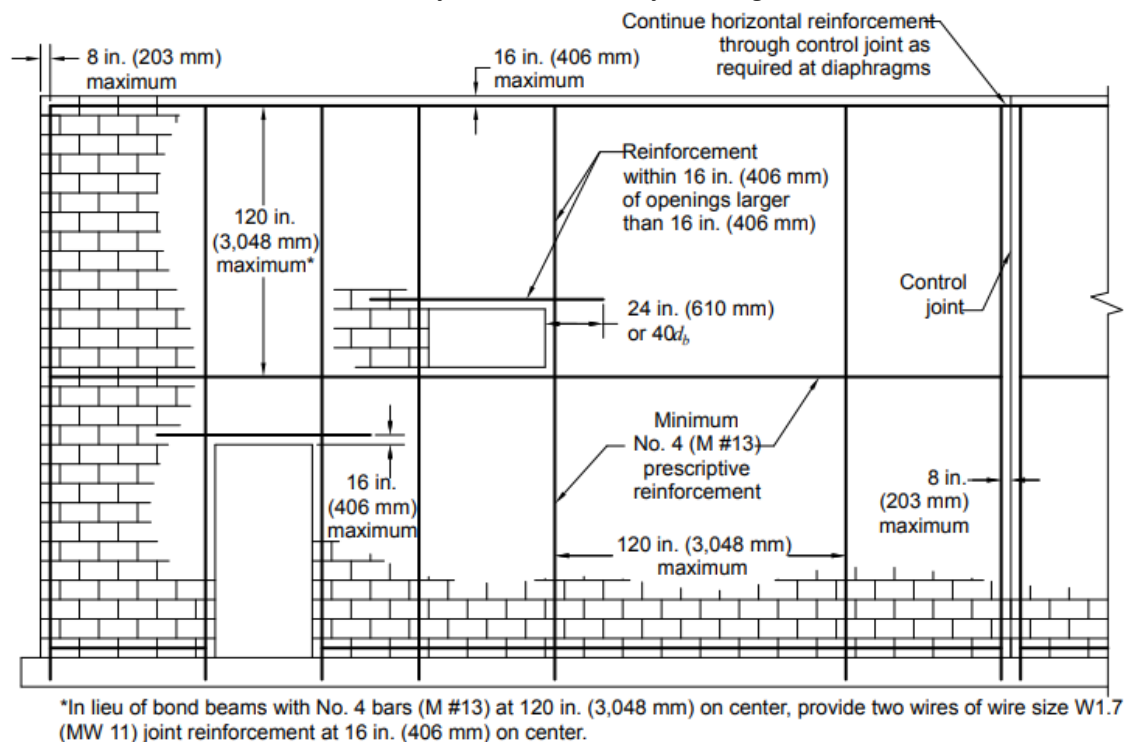
- Plain unreinforced masonry walls ($R = 1.5$);
- Detailed unreinforced masonry walls ($R = 2.0$);
- Reinforced masonry walls with ordinary detailing ($R = 2.0$);
- Reinforced masonry walls with intermediate detailing ($R = 3.5$);
- Reinforced masonry walls with special detailing ($R = 5.0$).

The detailing requirements for these systems, as outlined in TMS 402-602 (2022), are summarized below:

- Plain unreinforced masonry walls: Designed as unreinforced elements, and as such, rely entirely on the masonry to carry and distribute the anticipated loads. No prescriptive reinforcement is required;

- Detailed unreinforced masonry walls: Include minimum horizontal and vertical reinforcement to improve inelastic deformation and energy dissipation under seismic loads. At least one 12.5 mm diameter bar (1.29 cm²) must be placed vertically and horizontally at intervals not exceeding 3.0 m, with reinforcement at wall corners, ends, and openings. The requirements for this minimum reinforcement are outlined in Figure 34.

Figure 34: Prescriptive minimum reinforcement detailing for detailed unreinforced masonry walls and reinforced masonry walls with ordinary detailing.

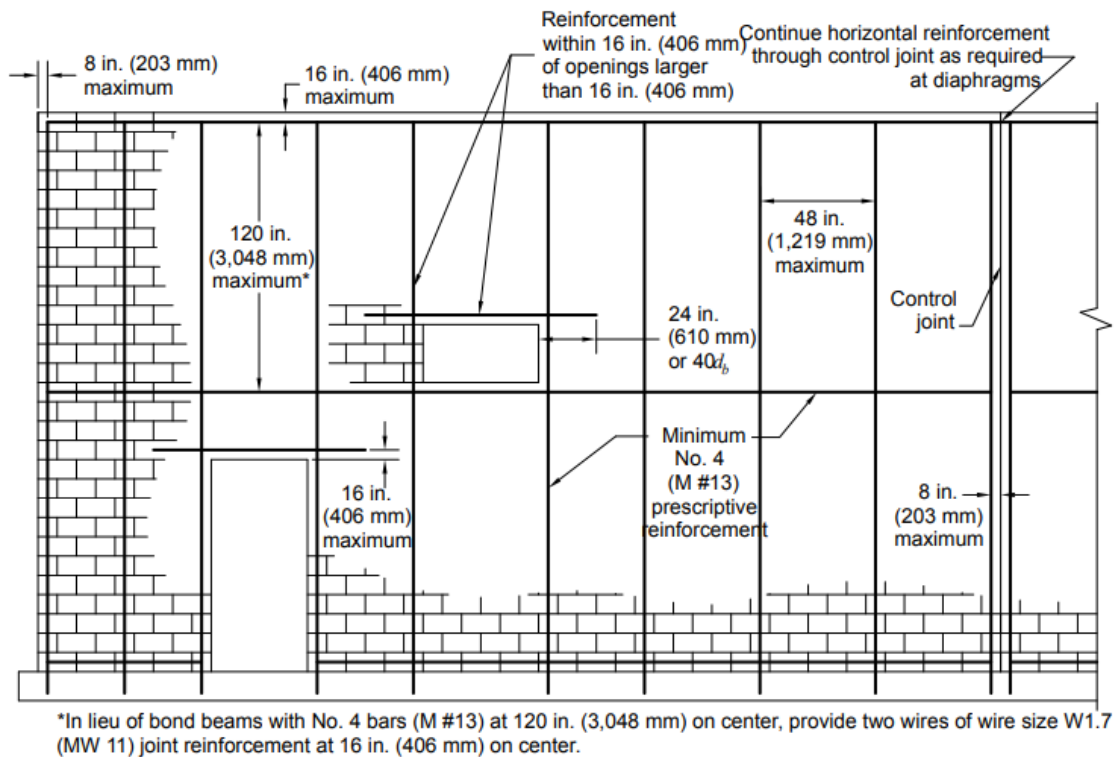


Source: CMHA (2025).

- Reinforced masonry walls with ordinary detailing: The minimum reinforcement requirements specified for detailed unreinforced masonry walls must be met. These requirements ensure safe resistance to design forces, as illustrated in Figure 34.
- Reinforced masonry walls with intermediate detailing: Maximum vertical reinforcement spacing is reduced from 3.0 m to 1.2 m, with additional specifications illustrated in Figure 35.
- Reinforced masonry walls with special detailing: For special detailing, the following prescriptions must be followed:
 - Reinforcement spacing must not exceed one-third of wall length, height, or 1.2 m.
 - Horizontal and vertical reinforcement must be uniformly distributed.
 - Combined horizontal and vertical reinforcement ratios must be at least 0.2%, based on the wall's gross cross-sectional area, with a minimum ratio of 0.07% in each direction.
 - Vertical reinforcement should be at least one-third of the horizontal reinforcement needed for in-plane shear resistance.
 - All horizontal reinforcement must be anchored with standard hooks around the vertical reinforcement.

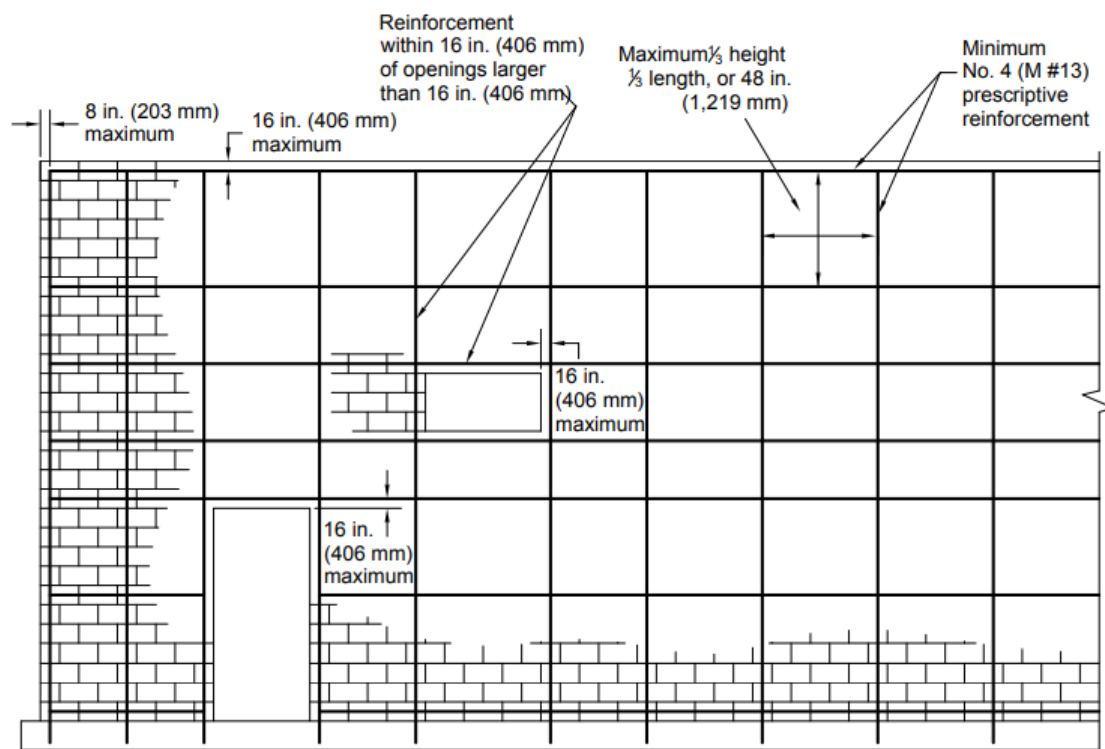
The special detailing prescriptions are illustrated in Figure 36.

Figure 35: Detailing for reinforced masonry walls with intermediate detailing.



Source: CMHA (2025).

Figure 36: Detailing for reinforced masonry walls with special detailing.



Source: CMHA (2025).

2.4.2.4 NBCC (2020)

The NBCC (2020) identifies four seismic-resistant systems for masonry construction, each with specific values for the ductility-related force modification factor, R_d , and overstrength-related force modification factor, R_o , as follows:

- Unreinforced masonry ($R_d=1.0$ and $R_o=1.0$);
- Conventionally reinforced masonry ($R_d=1.5$ and $R_o=1.5$);
- Moderately ductile reinforced masonry ($R_d=2.0$ and $R_o=1.5$);
- Ductile reinforced masonry ($R_d=3.0$ and $R_o=1.5$).

The primary difference between these systems lies in the required level of detailing for the walls, directly influencing the response modification factors. Notably, the ductility of unreinforced masonry systems is neglected, resulting in $R=1.0$. The detailing requirements are specified by CSA S304 (2024), and the main criteria are described below:

- **Unreinforced Masonry:** Walls are designed as unreinforced elements and are typically used in low-rise buildings. While prescriptive reinforcement is not required, their use is restricted to limited seismic zones, where the seismic demand parameter $I_e \cdot F_a \cdot S_a(0.2)$ — the product of importance factor, I_e , site amplification factor, F_a , and spectral acceleration at period, T , of 0.2 s, $S_a(0.2)$ — does not exceed 0.75. In these regions, considerations such as out-of-plane stability, minimum wall thickness, and proper construction practices are necessary to ensure safety under minor seismic forces.

- **Conventionally Reinforced Masonry:** Walls must comply with minimum seismic reinforcement requirements, illustrated in Figure 37 and outlined below:

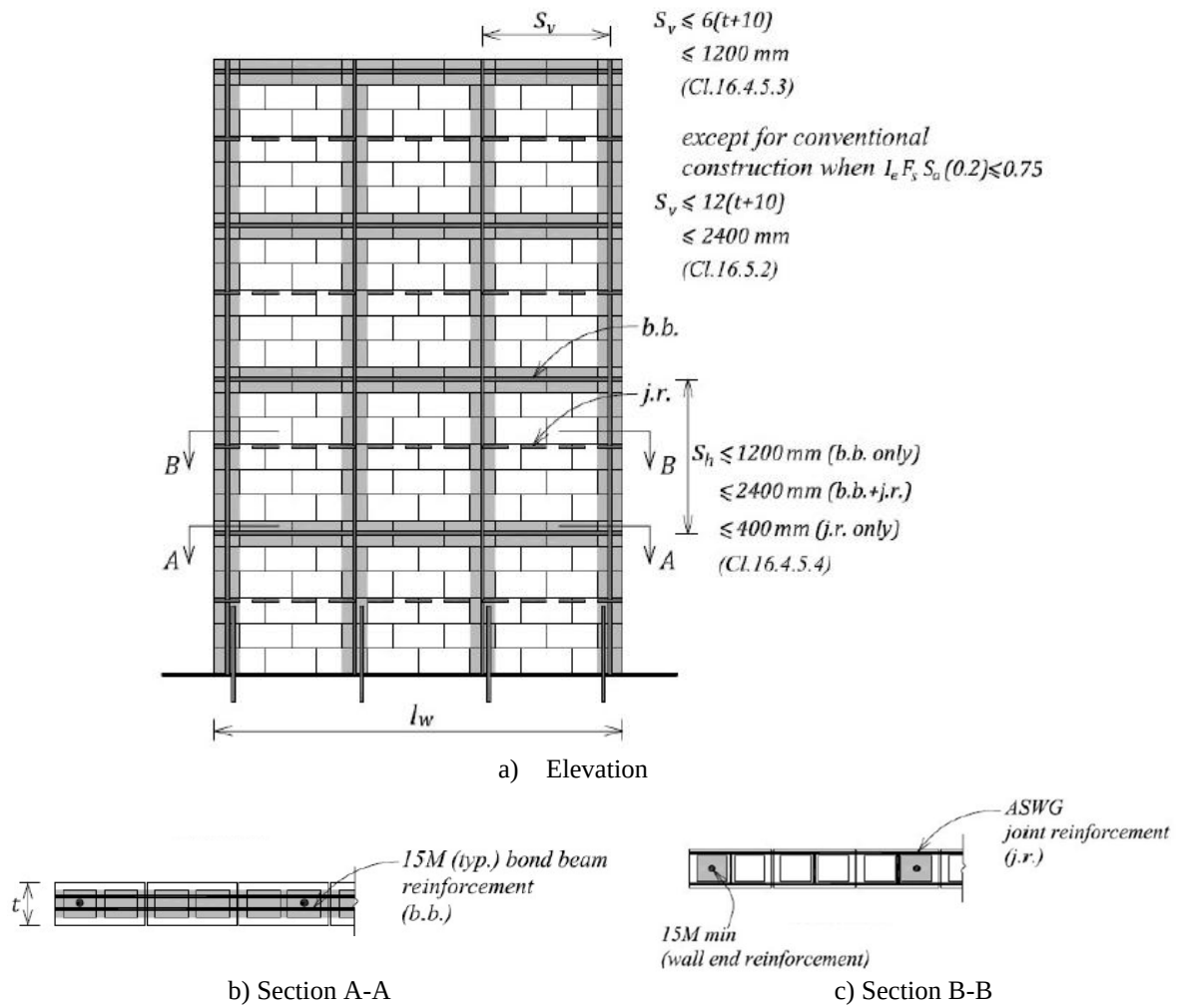
- A minimum reinforcement equivalent to one 16 mm bar must be provided at critical locations, including the perimeter of masonry panels and around openings exceeding 1000 mm in width or height;

- Vertical and horizontal reinforcement must be uniformly distributed across the wall's length;

- The vertical reinforcement spacing should not exceed the lesser of $6(t+10)$ mm or 1200 mm. In regions with seismic activity characterized by $I_e \cdot F_a \cdot S_a(0.2) \leq 0.75$, the vertical reinforcement spacing must not exceed the lesser of $12(t+10)$ mm or 2400 mm;

- Horizontal reinforcement spacing should not exceed 400 mm when using ASWG-type reinforcements in mortar joints or 1200 mm when bond beams are utilized as horizontal reinforcements. All reinforcements should be continuous between lateral supports.

Figure 37: Minimum seismic reinforcement requirements according to CSA S304.



Source: Adapted from Brzev and Anderson (2018).

- **Moderately Ductile Reinforced Masonry:** CSA S304 (2024) prescribes additional reinforcement requirements for moderately ductile shear walls to enhance seismic performance. These requirements are illustrated in Figure 38 and described below:

- In regions of inelastic flexural curvature, known as plastic hinge regions, supplemental seismic detailing is required. The height defining this region, h_p , is calculated as the greater of $l_w/2$ or $h_w/6$, but not exceeding $1.5l_w$. In this region, the wall should be fully grouted. An exception for partial grouting is allowed when $1 \leq h_w/l_w < 2$, but the compressive stress due to the factored axial load should be less than $0.1f_m$.

- Vertical reinforcement spacing must not exceed the lesser of $l_w/4$; $6(t+10) \text{ mm}$, or 1200 mm , with a minimum spacing of 600 mm .

- Lap splices for vertical reinforcement must be at least 1.5 times the development length, l_d , within the plastic hinge region;

- Horizontal reinforcement spacing in the plastic hinge region must not exceed $l_w/2$ or 1200 mm;
- Horizontal bars must include at least 90° hooks at the wall ends.
- Ductile Reinforced Masonry: The most stringent detailing requirements apply to ductile shear walls. These requirements are illustrated in Figure 40 and described below:
 - Full grouting is required in the plastic hinge region, with a height h_p calculated as $0,5l_w + 0,1h_w$, within the range of $0,8 l_w$ to $1,5l_w$;
 - Vertical reinforcement spacing must not exceed the lesser of $l_w/4$; $6(t+10)$ mm, or 1200 mm, with a minimum spacing of 400 mm;
 - No more than 50% of the vertical reinforcement area may be spliced within any section of the plastic hinge region;
 - Horizontal reinforcement spacing in the plastic hinge region must not exceed the lesser of 600 mm or $l_w/2$;
 - Horizontal bars must include at least 180° hooks at the wall ends.

Figure 38: Detailing requirements for moderately ductile masonry walls according to CSA S304 (elevation).

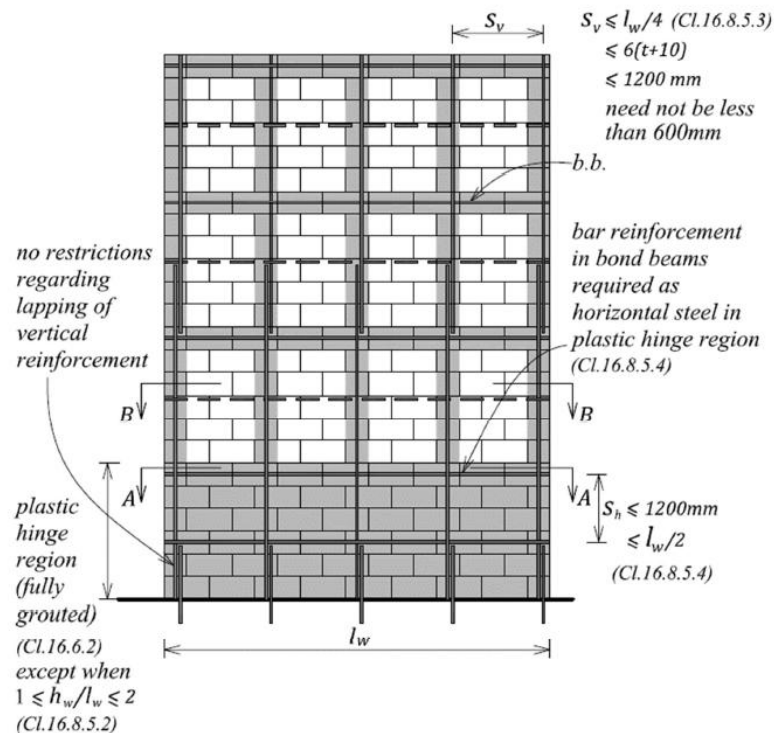


Figure 39: Detailing requirements for moderately ductile masonry walls according to CSA S304 (Section A-A and Section B-B).

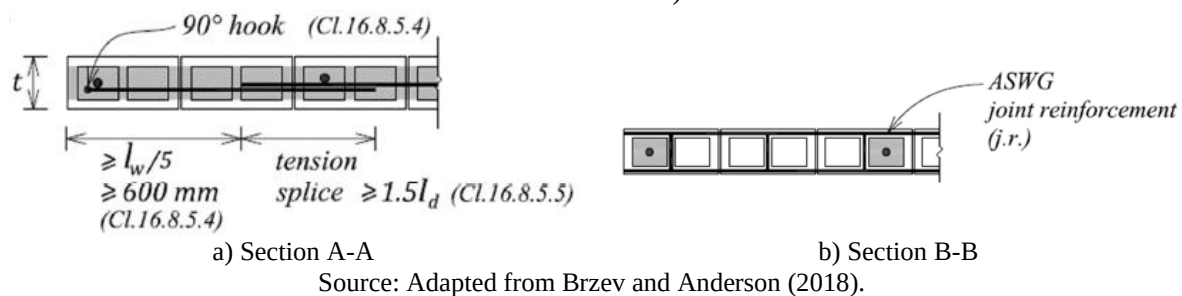
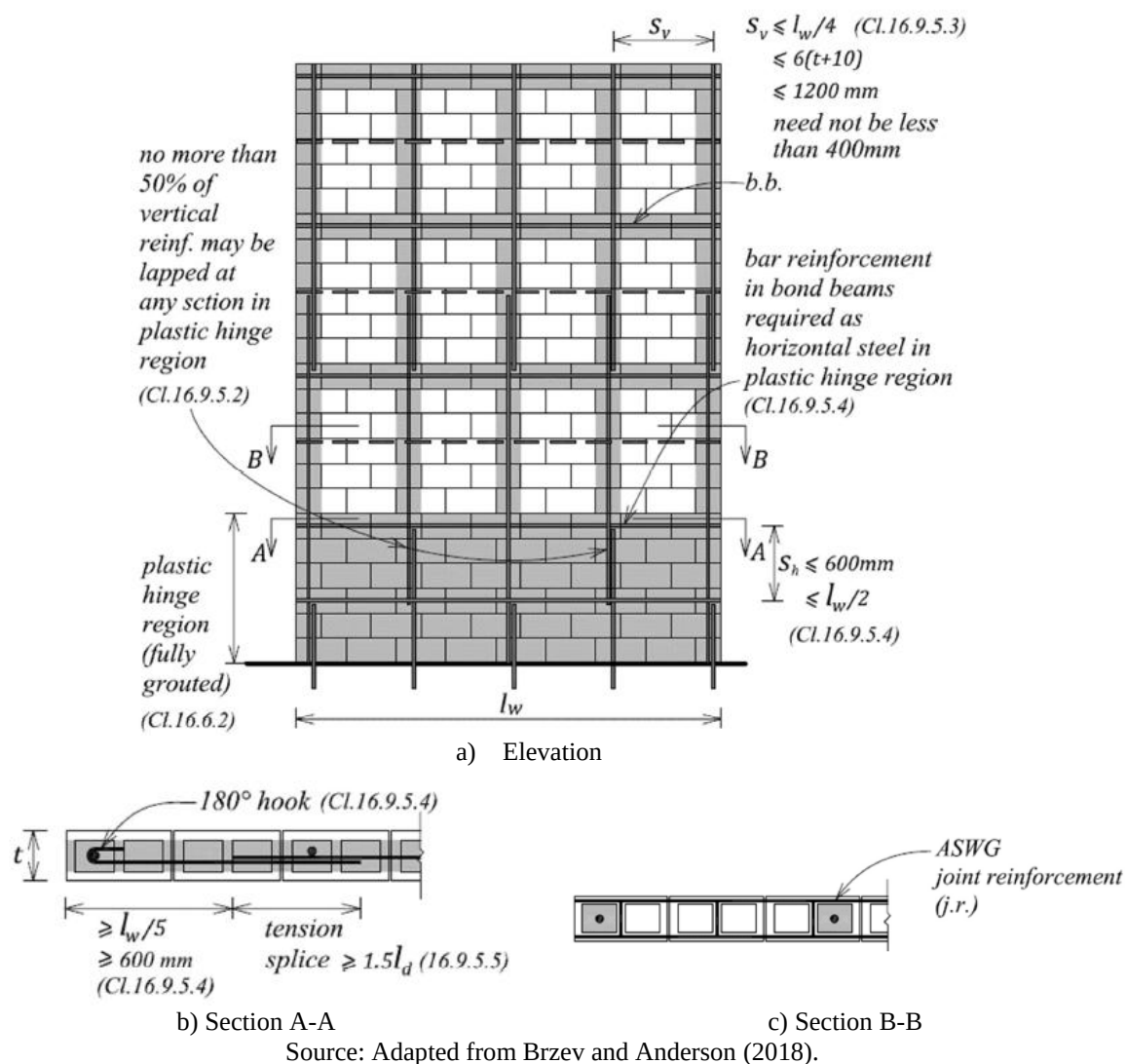


Figure 40: Detailing requirements for ductile masonry walls according to CSA S304.



2.4.2.5 ABNT NBR 15421 (2023)

The Brazilian seismic design standard, ABNT NBR 15421 (2023), specifies an R value of 1.5 for structural masonry systems with "usual detailing", referring to the level of detailing required by the respective Brazilian material design standards. However, the masonry design standard, ABNT NBR 16868 (2020), does not explicitly define categories of seismic detailing

(e.g., usual, special) or establish minimum requirements specifically for seismic performance. This lack of clarity underscores a significant gap in the Brazilian seismic design framework, particularly regarding masonry structures. To improve the precision and reliability of seismic design in Brazil, further research is necessary to develop and formalize detailed guidelines and criteria for masonry systems.

To summarize and provide a clearer comparison of the seismic parameters for masonry structures across different design standards, the values of R , R_d , and R_0 for each standard discussed here are summarized in Table 1.

Table 1: Summary of response modification factors (R , R_d , and R_0) for masonry systems in design standards.

Design standard	Masonry Typology	R	R_d	R_0
EN 1998-1 (2004)	Unreinforced	1.5 - 2.5	1.5 - 2.5	-
	Confined	2.0 - 3.0	2.0 - 3.0	-
	Reinforced	2.5 - 3.0	2.5 - 3.0	-
NTC (2018)	Unreinforced	3.0 - 4.4	1.75	1.7 - 2.5
	Confined	3.2 - 7.5	2 - 3.0	1.6 - 2.5
	Reinforced	3.75 - 7.5	2.5 - 3.0	1.5 - 2.5
ASCE SEI 7 (2022)	Plain unreinforced	1.5	-	-
	Detailed unreinforced	2.0	-	-
	Ordinary detailing	2.0	-	-
	Intermediate detailing	3.5	-	-
	Special detailing	5.0	-	-
NBCC (2020)	Unreinforced	1.0	1.0	1.0
	Conventionally reinforced	2.25	1.5	1.5
	Moderately ductile	3.0	2.0	1.5
	Ductile reinforced	4.5	3.0	1.5
NBR 15421 (2023)	Usual detailing	1.5	-	-

Source: Author.

2.4.3 Research on response modification factors

The response modification factor of a building system can be derived through laboratory testing or numerical modeling. While laboratory testing provides valuable insights, it is often costly and limited to specific configurations. In contrast, numerical modeling offers cost-efficient analysis across various building configurations and seismic ground motions, making it a practical choice for comprehensive evaluations. The following section briefly overviews key research contributions in this area.

2.4.3.1 Morandi (2006) and Morandi et al. (2022)

Several studies highlight inconsistencies in linear seismic analysis methods, particularly when applied to masonry buildings. For example, linear methods using R values of 1.5 or 2.0 — suggested by seismic standards like EN 1998-1 (2004) for URM — fail to satisfy safety checks prescribed by EN 1996-1 (2005) for typical URM buildings under seismic intensities of $a_g S > 0.10g$ and sometimes even for $a_g S = 0.05g$ (Lagomarsino; Marino; Cattari, 2020; Magenes, 2006; Morandi, 2006; Morandi; Manzini; Magenes, 2020). These issues stem not from the seismic actions from spectral acceleration but from the omission of overstrength, R_0 , in response modification factor definitions (Beyer; Magenes, 2020; Magenes, 2006; Morandi et al., 2022). In Brazil, where ground motions often fall within these ranges, this underscores the need for more localized studies to refine seismic design parameters, particularly for soil types D and E, common in regions like Fortaleza-CE (Miranda, 2021).

In this regard, Morandi (2006) conducted nonlinear analyses on 12 configurations of unreinforced and reinforced masonry buildings. The buildings analyzed had simple and regular layouts, with wall areas varying between 3.5% and 7.5% of the total floor area in each primary direction. Wall thicknesses of 250 mm, 300 mm, and 350 mm were considered. The elevations of the buildings varied, with URM buildings modeled for one to three stories and RM buildings for two to four stories. For reinforced masonry buildings, a minimum reinforcement was provided according to EN 1998-1 (2004). The numerical analyses utilized a simplified equivalent frame modeling approach. Elastic-plastic behavior with limited deformation was assumed for all structural elements, with failure mechanisms defined for ultimate flexure and shear. Ultimate deformation capacities for URM were set at 0.8% for flexure and 0.4% for shear, while RM values were 1.2% and 0.6%, respectively. The overstrength factor, R_0 , was derived from pushover analyses. For URM buildings, R_0 values ranged from 1.21 to 3.91, with an average of 2.39 and a standard deviation of 0.60. For RM buildings, R_0 values ranged from 1.19 to 3.30, with an average of 1.86 and a standard deviation of 0.44.

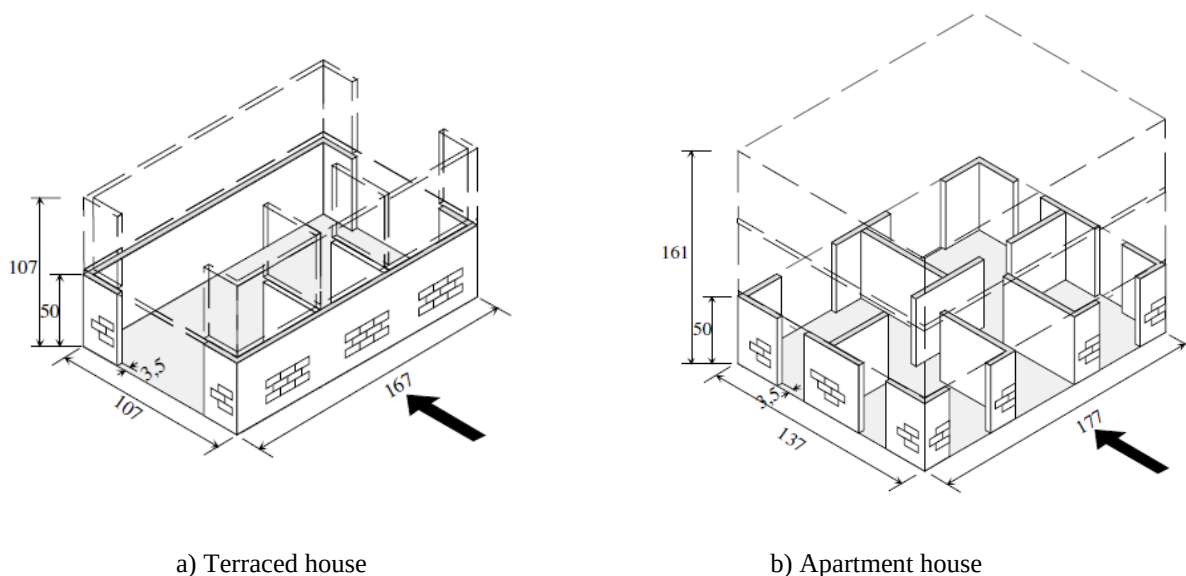
Morandi et al. (2022) extended this work to Italian and German low-rise URM buildings. It focuses on low-rise buildings, up to four stories, with rigid floors located in regions with moderate seismic activity, similar to those analyzed in Morandi (2006). Using the N2 methods (Fajfar, 1999), the study reported behavior factors, R_d , ranging from 1.35 to 1.70 and R_0 values between 1.50 and 1.60, yielding combined R factors of 2.40 to 2.90, with the results being applicable only to regular buildings in both elevation and plan. Irregular configurations required reductions in R values (0.80 for elevation irregularities, 0.64 for plan irregularities).

The study highlighted the importance of carefully considering layout and redundancy in masonry buildings and determining R values.

2.4.3.2 Tomažević et al. (2004, 2010)

Tomažević et al. (2004; 2010) conducted shaking table tests on scaled masonry buildings to evaluate the structural behavior factor, q , by Eurocode 8 for unreinforced and confined masonry systems. The research focused on two structural configurations — a two-storey terraced house and a three-storey apartment house (Figure 41).

Figure 41: Arrangement of scaled masonry buildings tested by Tomažević et al. (2010). Dimensions in cm.



Source: Tomažević et al. (2010).

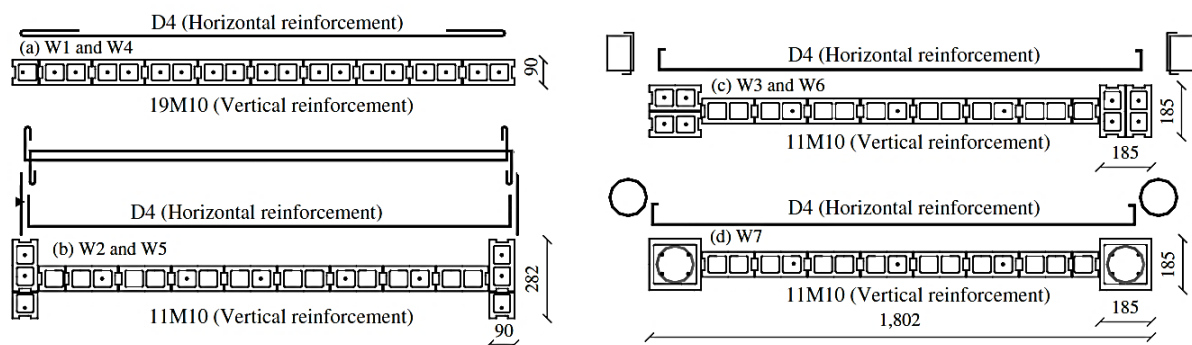
The study quantified q -values by analyzing the relationship between the elastic response and the observed inelastic behavior of the models. For URM, q -values ranged from 1.56 to 2.05, reflecting the brittle nature of these systems. Confined masonry exhibited higher q -values, between 2.43 and 4.50, due to its improved energy dissipation and deformation capacity. Overall, the study concludes that q -values cannot be accurately assessed through isolated wall tests alone but require full-system analyses to capture the effects of structural configuration, material variability, and damage progression.

2.4.3.3 Shedid et al. (2010b, 2011)

Shedid et al. (2010b, 2011) investigated the seismic performance of RM shear walls with different cross-sectional geometries, including rectangular, flanged, and end-confined

walls, representing two- and three-storey high RCM structural walls (Figure 42). Shedid et al. (2010b) tested six FGRM rectangular walls under cyclic loading. The total response modification factor ($R_d \cdot R_o$) for the tested walls varied between 3.5 and 6.6, influenced by reinforcement ratios and axial stress levels. Shedid et al. (2011) expanded the analysis to include flanged and end-confined RMSWs, comparing their seismic behavior to rectangular walls. For rectangular walls, the R value was reported as 8.4. For flanged walls, which exhibited a 30% increase, the factor was approximately 10.9. With a 72% increase, end-confined walls achieved a factor of approximately 14.45. These findings highlight the superior ductility and energy dissipation capacity of flanged and end-confined walls, demonstrating their potential as significantly more effective alternatives to rectangular walls in seismic performance. It is also evident that different aspect ratios and, consequently, different natural periods caused significant changes in the seismic coefficients.

Figure 42: Arrangement of the walls studied by Shedid et al. (2011).



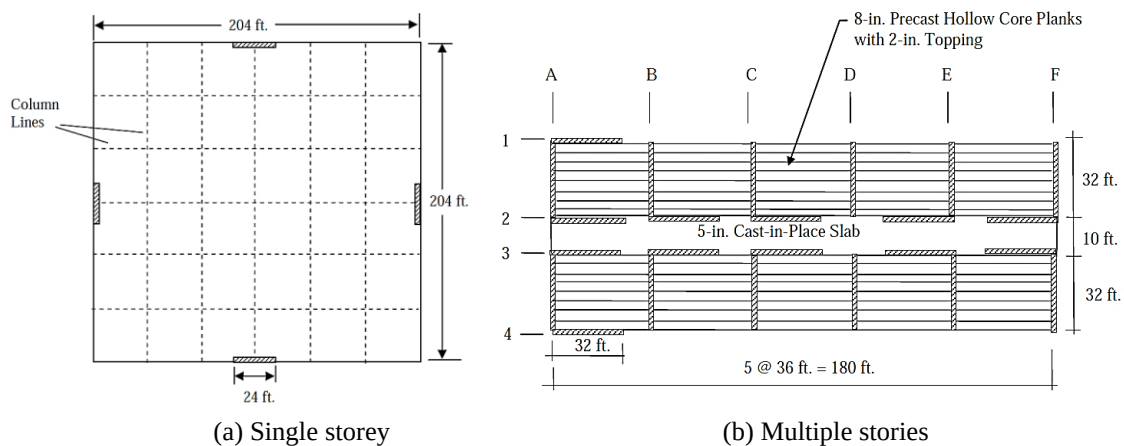
Source: Shedid et al. (2010b, 2011).

2.4.3.4 NIST (2010)

The FEMA P695 (2009) methodology provides a structured framework for quantifying seismic performance factors, including R , of a newly developed system. This has been performed using incremental dynamic analysis (IDA) while the system is subjected to maximum considered earthquake (MCE) ground motion.

The National Institute of Standards and Technology (NIST) (2010), evaluated the FEMA P-695 methodology by applying it to a variety of structural systems, including RM walls systems ranging in the number of stories (2, 4, 8, and 12), wall aspect ratio, axial loading, seismic category, total or partial grouting, and reinforcement spacing. The archetype adopted was distinguished for buildings with a single storey and multiple stories (Figure 43). The design and verification did not consider flanges and coupling between piers in walls with openings.

Figure 43: Plan view of the typical archetypes used in the NIST (2010) study.



Source: NIST (2010).

The study found that the R factor for special RM walls was consistent with the standard value of 5.0, as these systems demonstrated superior energy dissipation and overstrength. In contrast, ordinary RM walls, which exhibited reduced ductility, warranted a lower recommended R factor to account for their increased collapse risk. The evaluation also highlighted the importance of archetype selection in determining R -factors. By analyzing a range of structural configurations, including single-storey and multistorey buildings, the study confirmed that short-period systems often experience greater seismic demands, requiring careful calibration of their R -factors.

2.4.3.5 Ezzeldin et al. (2017)

Ezzeldin et al. (2017) investigate the seismic performance and collapse risk of RM shear walls with boundary elements. This study adopted the same archetype, Figure 43, and the same number of stories as the NIST (2010). The configuration of the walls was similar to those with ends confined by masonry columns analyzed by Shedid et al. (2011), shown in Figure 42. Ezzeldin et al. (2017) found the R -factor of 5.0 prescribed for RM walls to be conservative for walls with boundary elements, as such elements enhance ductility by providing additional confinement, improving the seismic performance of the walls.

2.4.3.6 Beyer and Magenes (2020)

Beyer and Magenes (2020) presented a comprehensive proposal for revising the behavior factors, q , for modern masonry buildings, specifically for inclusion in the new version of the Eurocode 8 (EC8) Part 1. Their work addressed limitations in the current standard, which often fails to account for the diversity of masonry construction practices and the seismic

performance of newer masonry typologies. The proposal integrates findings from extensive numerical and experimental studies, encompassing various configurations of URM and RM buildings.

These authors suggest that the q factor be decomposed into three components: q_D (representing ductility and energy dissipation), q_R (accounting for structural redundancy and force redistribution), and q_S (reflecting overstrength due to material properties and construction practices). For modern URM buildings, the proposed q -factors were derived through extensive pushover analyses of 1- to 4-storey buildings with varying wall thicknesses, axial load ratios, and plan configurations. The studies revealed that current q -values in EC8 underestimate the seismic capacity of well-designed URM buildings with stiff slabs and robust connections.

The research highlights that q_D , related to the deformation capacity of masonry walls, is primarily influenced by drift limits associated with failure modes. For walls failing in shear, ultimate drift capacities were conservatively estimated based on experimental data, leading to a proposed $q_D = 2.0$ for URM buildings with a drift limit of 0.3%. Reinforced and confined masonry walls, with greater deformation capacities, were assigned higher q_D -factors of 1.8 and 1.65, respectively.

Structural redundancy, reflected in q_R , is strongly tied to the number and configuration of walls. Buildings with more than six walls per direction, varying wall lengths, and significant coupling effects (e.g., stiff slabs or ring beams) achieved q_R values of 1.6. In contrast, configurations with fewer walls or poor coupling were assigned lower q_R -values, often near 1.0, reflecting their limited capacity for force redistribution. The coupling effect, a critical factor in redundant structures, influences axial force redistribution among walls and enhances overall seismic performance.

The overstrength component q_S takes into account material properties and unavoidable overdesign. For URM, $q_S = 1.35$ was proposed, while RM and confined masonry systems, benefiting from distributed reinforcement and better inelastic performance, were assigned $q_S = 1.5$.

Based on their extensive research, Bayer and Magenes (2020) proposed the maximum q -factors: $q_{max} = 2.8$ for unreinforced masonry, $q_{max} = 3.4$ for confined masonry, and $q_{max} = 3.8$ for reinforced masonry). These values reflect the varying ductility and energy dissipation capacities of different masonry systems, with unreinforced masonry offering limited ductility, confined masonry benefiting from enhanced deformation capacity due to reinforced concrete

elements, and reinforced masonry achieving superior seismic performance through greater ductility and overstrength.

2.4.3.7 Medeiros et al. (2022a)

Medeiros et al. (2022a) conducted a numerical investigation on the seismic performance of three-storey perforated masonry walls, evaluating the influence of reinforcement detailing strategies (concentrated at the wall piers vs. distributed along the wall, maintaining the same total reinforcement/grout) and axial load levels on the response modification factor (R values). The study adopted a trilinear idealization approach, incorporating post-peak behavior to better capture the nonlinear response of the walls. The findings revealed significant variability in R values, ranging from 4.0 to 9.7, with axial load emerging as the most influential parameter.

Walls subjected to lower axial loads exhibited higher R values, benefiting from increased ductility and improved seismic performance, with average R values reaching 9.6 for concentrated reinforcement detailing and 9.5 for distributed reinforcement detailing. In contrast, under higher axial loads, the walls experienced a notable reduction in R values, indicating a more brittle response, with concentrated reinforcement detailing reaching as low as 4.0 and distributed reinforcement detailing reaching 4.9. While reinforcement detailing distribution had minimal impact on R values, the dominant effect of axial load underscores its critical role in the seismic performance of masonry walls.

2.4.3.8 Elmeligy et al. (2023)

Elmeligy et al. (2023) investigated the seismic performance of flexural-dominated PGRM walls, with a focus on evaluating the force modification factors, R , specified in CSA S304 (2014) and TMS 402/602 (2016). These walls were designed following the seismic requirements for moderately ductile walls in CSA S304 (2014) and intermediate reinforced masonry walls in TMS 402/602 (2016), specifically focusing on flexural-dominated behavior.

The results revealed distinct differences in the adequacy of R -factors prescribed by the two design standards: The calculated R -factors for walls designed under CSA S304 (2014) consistently exceeded the standard-prescribed value of $R = 3$, with minimum R -values greater than 4.8. This suggests that the CSA S304 (2014) design guidelines provide an adequate margin of safety and can be extended to cover such systems without significant revisions. In contrast, walls designed under TMS 402/602 (2016) showed greater variability, with some walls

achieving R -values below the standard minimum of 3.5, highlighting potential deficiencies in the design provisions for intermediate reinforced masonry walls.

In addition to the studies detailed here, other research on seismic performance coefficients of masonry systems is available in the literature (Benedetti; Carydis; Pezzoli, 1998; Bolhassani et al., 2016a; Frumento et al., 2009; Hassanli; ElGawady; Mills, 2015; Tomažević; Weiss, 1994; Zhang et al., 2021).

2.5 FINAL REMARKS

The review presented in this chapter offers comprehensive insights from experimental studies, numerical modeling efforts, and standard-based prescriptions concerning the behavior and seismic performance of masonry structures.

Experimental studies have provided valuable data on the inelastic behavior of masonry walls, identifying key failure mechanisms such as flexural, shear, and sliding modes. These studies have highlighted the influence of parameters such as aspect ratio, axial load, and reinforcement detailing on wall response. However, most experimental programs have focused on individual wall components under specific boundary conditions, limiting the ability to generalize findings to more complex building systems. Interaction effects within multi-storey, three-dimensional configurations — particularly the influence of orthogonal walls — remain underexplored, especially in the context of real-world layout configurations and construction practices across diverse seismic regions.

Advancements in numerical modeling have improved the ability to simulate the nonlinear behavior of masonry structures. Despite this progress, challenges persist in achieving a balance between model accuracy and computational efficiency, particularly for PGRM systems. The heterogeneity introduced by ungrouted regions, variable reinforcement distributions, and complex stress distribution patterns poses significant modeling difficulties. Furthermore, the applicability of existing models across a wide range of concrete masonry scenarios — including URM, PGRM, and FGRM walls — has not been fully validated.

The response modification factors, R , prescribed in seismic design standards are primarily based on empirical data, which may not adequately reflect the variability in masonry construction practices worldwide. In particular, a deeper understanding of how reinforcement detailing and layout configurations influence R is crucial for the potential review and updating of the values currently adopted in seismic design standards. In conclusion, while substantial

progress has been made in advancing the seismic design of masonry structures, addressing the identified gaps is essential to ensure continued progress in the field. To address these gaps, the research in this thesis aims first to develop and validate a numerical modeling approach based on extensive experimental wall tests covering diverse configurations and design scenarios. Once validated, the model framework will be applied to investigate the nonlinear behavior of multi-storey, three-dimensional concrete masonry systems, focusing on assessing the influence of orthogonal walls on the response of the system. Subsequently, the study evaluates real-world buildings to emphasize how structural layout influences seismic performance. Insights gained from these numerical simulations are expected to contribute to the refinement of building seismic design provisions, supporting the development of more robust and context-specific design guidelines.

3. A MACRO-MODELING APPROACH FOR NON-LINEAR ANALYSIS OF MULTI-STOREY PERFORATED MASONRY WALLS WITH GROUT AND REINFORCEMENT CONCENTRATED AT THEIR PIER ENDS

3.1 INTRODUCTION

Current numerical approaches applied to the simulation of the behavior of masonry structures are classified as micro- or macro-modeling, depending on the model detail. Micro-models are intended for a detailed discretization of the masonry at the level of the constituent elements. Therefore, a better understanding of localized behavior and possible damage mechanisms involving the masonry constituents can be achieved. On the other hand, in macro-models the masonry is considered as a homogeneous continuum, with the aim of simulating the behavior of the masonry on a macroscopic scale. Each modeling approach has different fields of application. However, macro-modeling is a more advantageous alternative when it is necessary to combine accuracy and efficiency, as in typical cases of the seismic design of masonry buildings (Lourenço, 2002).

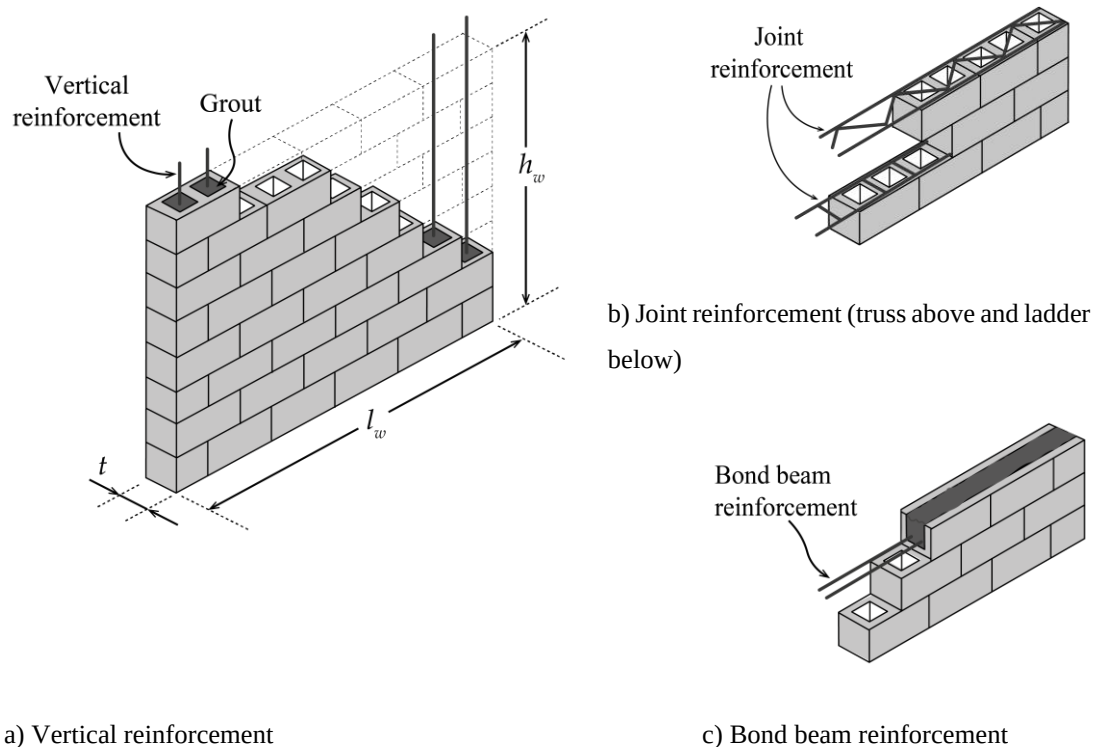
The possibility of exploring the non-linear behavior of buildings in practical seismic design applications, such as in performance-based pushover analysis implemented in design standards (i.e., Eurocode 8 (2004), FEMA 440 (2013), and NTC (2018)), has driven the development of several macro-element modeling approaches, specially oriented to unreinforced masonry (URM) structures (Caliò; Marletta; Pantò, 2012; Lagomarsino et al., 2013; Magenes et al., 2006; Raka et al., 2015). Comparative analyses between these approaches were performed by Marques and Lourenço (2011, 2013, 2014) and Pantò et al. (2015). However, further work is needed to validate the available approaches comprehensively, to allow extension of their application to other building typologies, particularly partially grouted masonry (PGM) structures, as well as to provide guidance on the use of the available tools in design practice.

Experimental work has shown that the combination of masonry, grout, and reinforcement – reinforced masonry (RM) – creates better structural performance in terms of lateral load capacity, ductility, crack opening control, and energy dissipation when compared to URM; e.g., Alves (2011), Haach (2009), Magenes et al. (1996), and Voon and Ingham

(2006). Consequently, RM structures have been widely used, especially in North America. In this structural system, the walls can be fully or partially grouted. However, as Koutras and Shing (2020) reported, in regions of low to moderate seismicity in North America, RM structures are mostly partially grouted due to their practical and economic advantages.

In PGM walls, only vertical cells and courses containing reinforcement are grouted. Vertical reinforcement is commonly in the form of standard steel bars, while horizontal reinforcement can be applied as steel bars (to form a bond beam) or a ladder or truss-type wire reinforcement embedded in the mortar joints (joint reinforcement). These typical detailing aspects of PGM walls are illustrated in Figure 44.

Figure 44: Detailing aspects of PGM walls.



Source: Author.

Despite the better performance of PGM walls compared to URM walls, the numerical simulation of their inelastic behavior represents a complex issue. This is due to the interaction between the grouted/reinforced and ungrouted portions of the masonry and the difference in stiffness between these two parts. In addition, partial grouting generates a non-uniform cross-section, resulting in heterogeneous stress-strain patterns and complex damage distribution (Calderón et al., 2021c). Difficulties in conceiving and implementing simplified macro-models for PGM walls have led to the use of 2D and 3D finite element models with different levels of

detail and accuracy (Arnau; Sandoval; Murià-Vila, 2015; Bolhassani; Hamid; Moon, 2016c; Calderón et al., 2021d; Calderón; Milani; Sandoval, 2021; Calderón; Sandoval; Arnau, 2017; Elmeligy; Aly; Galal, 2021; Koutras; Shing, 2020, 2021b; Medeiros et al., 2022a). Although these modeling strategies are valuable for the advancement of research and the understanding of the behavior of PGM walls, they demand higher computational costs and are difficult to apply in engineering designs. For these reasons, establishing numerical approaches capable of combining accuracy and efficiency is highly relevant in the computational simulation of building structures.

The research described here was aimed at evaluating a macro-modeling approach in simulating the global in-plane behavior of masonry walls with grout and vertical reinforcement concentrated at the pier ends. In this approach, the grouted/reinforced masonry portions are modeled using frame elements with lumped plasticity and the ungrouted masonry panels employed 2D macro-elements idealized by Calìo et al. (2005, 2012). The interaction between these two parts is simulated using non-linear interface elements. Similar approaches have been validated in numerous applications of composite reinforced concrete-masonry structures, like confined masonry and infilled masonry (Caddemi et al., 2013a; Caddemi et al., 2013b; Calìo et al., 2008; Calìo; Pantò, 2014; Marques; Lourenço, 2013; Nucera et al., 2012a; 2012b; Pantò; Calìo; Lourenço, 2017). In this work, the approach is applied in an unprecedented way on PGM walls with vertical reinforcement concentrated at the ends.

The proposed approach was evaluated based on the in-plane loading simulation of four multi-storey perforated walls tested by Fortes et al. (2017; 2016). Additionally, results of two-dimensional finite element models for the same walls, obtained by Medeiros et al. (2021; 2022a; 2022b), were considered in the assessment. Finally, a sensitivity analysis was performed to investigate the influence of different modeling parameters on the results.

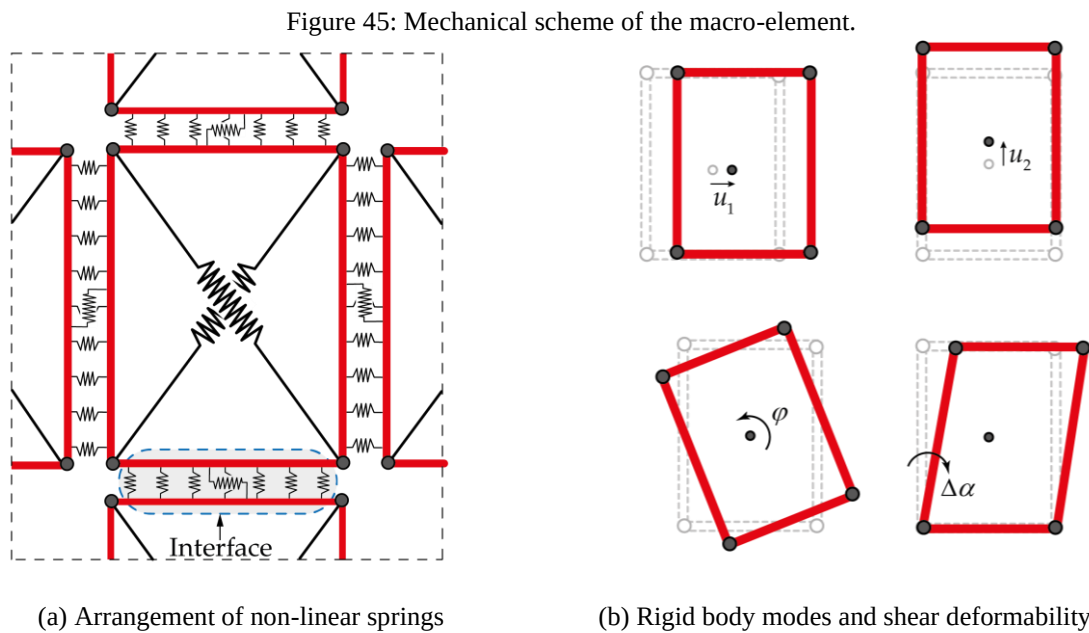
3.2 ESSENTIAL CHARACTERISTICS OF THE EVALUATED MACRO-MODELING APPROACH

The modeling approach adopted here is based on the two-dimensional macro-element conceived by Calìo et al. (2005, 2012) for the simulation of the non-linear behavior of URM walls. This macro-element is implemented in the 3DMacro[®] software (Calìo et al., 2012) and has applications in scientific research and seismic design. More recent studies are presented in Maio et al. (2020), Celano et al. (2021a), Castellazzi et al. (2022), D'altri et al. (2022), and

Occhipinti et al. (2022). The essential characteristics of the modelling method are presented here, while the specific approach for simulating PGM walls will be described in Section 4.

3.2.1 Macro-element

The macro-element consists of an articulated quadrilateral with rigid edges, in which two diagonal springs are connected to the corners to simulate the diagonal shear behavior. Discrete distributions of springs orthogonal to the sides of the macro-element (interfaces) simulate the axial-flexural behavior. A spring parallel to each side of the macro-element can also be included to simulate the potential sliding mechanism. The mechanical scheme of the macro-element is illustrated in Figure 45a. Note that, for convenience of representation, the interfaces are represented with a thickness, even though they have zero thickness (Caliò; Pantò, 2014). Each macro-element has the rigid body modes and one more for describing the shear deformability as shown in Figure 45b.



Source: Occhipinti et al. (2022).

Studies have shown that the proposed model is capable of simulating the main failure mechanisms of masonry walls loaded in their plane (Caliò; Marletta; Pantò, 2008; Celano et al., 2021a; Marques; Lourenço, 2011; Occhipinti et al., 2022; Pantò et al., 2015). In general, three main failure modes can be distinguished: flexural, diagonal shear, and sliding. In the case of diagonal shear failure, as reported by Marques and Lourenço (2011), and Celano et al. (2021a; 2021b), two different mechanisms can be identified: diagonal sliding and diagonal cracking.

According to Calì et al. (2012), considering the typical mechanical properties of masonry and the geometry of structural walls, diagonal shear failure is the most common type of failure for in-plane loaded URM walls. Therefore, diagonal springs have greater importance in the model. Besides, Caddemi et al. (2013b) report that the sliding shear mechanism is irrelevant in macro-model applications. Nevertheless, its consideration can be suitable in some specific situations, for example, when there are weak joints and the walls are discretized into several elements (Calì et al., 2012). For the present study, as in other studies (D’Altri et al., 2022; Pantò; Calì; Lourenço, 2017), the sliding shear mechanism was inhibited in the models.

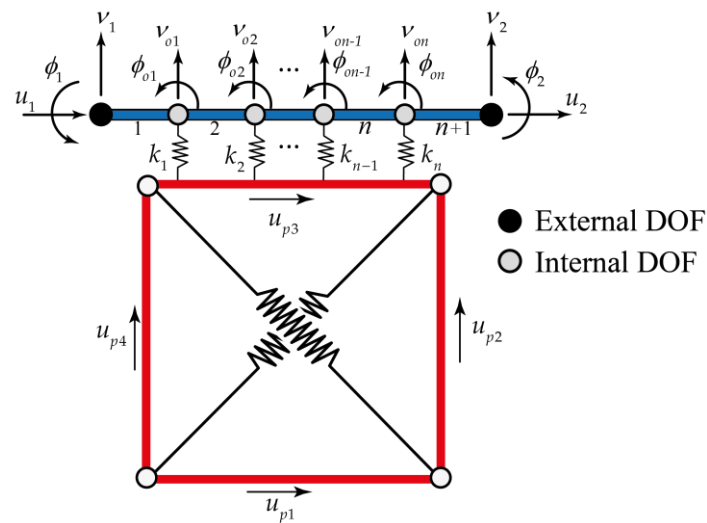
3.2.2 Interaction between the macro-element and frame elements

An original aspect of the macro-element is that it can interact with other elements through non-linear springs distributed along the sides (interfaces) of the element. This feature allows simulation of the interaction between the masonry panels (macro-elements) and frame elements, extending the applications to structures consisting of masonry walls enclosed by reinforced concrete or steel frames (Caddemi et al., 2013a).

When the macro-element interacts with a frame element, the macro-element interface has, in addition to the standard degrees of freedom (DOFs), internal DOFs that enforce correspondence with the frame element along its full length. The DOFs that govern the interaction between a panel (disregarding the sliding shear spring) and an adjacent beam element are schematized in Figure 46, where: u_{pk} ($k = 1 \dots 4$) are the four DOFs that describe the kinematics of the macro-element; $u_1, v_1, \varphi_1, u_2, v_2$, and φ_2 are the DOFs at the beam ends, named external DOFs; while v_{oj}, φ_{oj} ($j = 1 \dots n$) are the internal DOFs corresponding to the non-linear springs at the ends of the beam sub-elements (Calì; Pantò, 2014).

The frame element flexural behavior is characterized by external DOFs represented by the displacements and rotations of the frame ends and by the internal DOFs represented by the displacements and rotations of the frame sub-element ends. The axial behavior is governed only by the two axial displacements at the frame ends and by an internal displacement, u_m , only considered when the spring parallel to the interface (sliding shear spring) is enabled. In the case of a frame element considering the axial-flexural interaction, the axial behavior is enriched by including the axial displacements of all sub-elements to allow for the coupling with the flexural behavior (Gruppo Sismica s.r.l, 2020).

Figure 46: Illustration of an interface between a beam element and an adjacent macro-element.



Source: Calìo and Pantò (2014).

According to Calìo and Pantò (2014), the frame elements are modeled using beam-column type non-linear finite elements with plasticity concentrated at hinges, which are assumed to occur in each sub-element bounded between two consecutive non-linear springs. This representation provides a model capable of simulating the occurrence of plastic hinges in different positions of the frame element. In addition, it is consistent with the discretization level of the interface springs adopted for the macro-element.

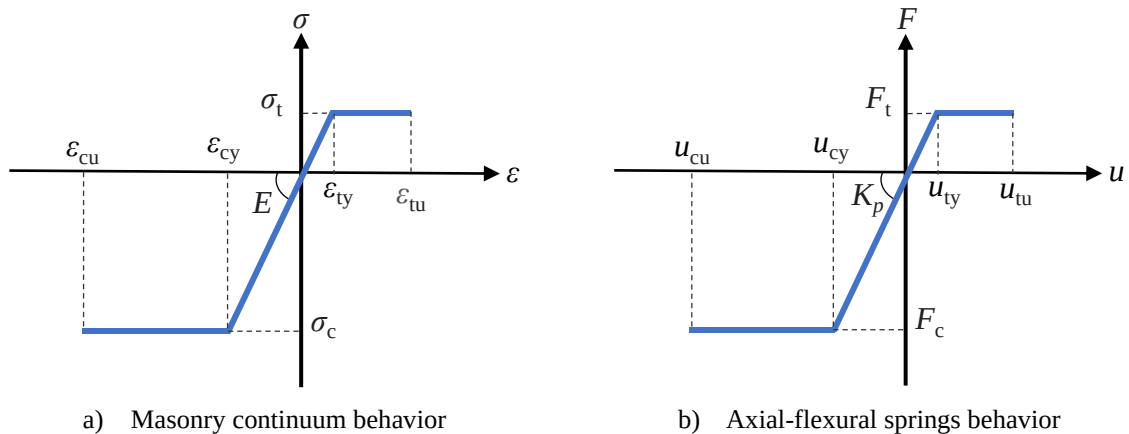
In the 3DMacro[®] software (2012), the frame elements are classified according to the following types: 1D (Tie-Rod), which represents exclusively axial behavior (suitable for modeling tie-rod elements); 2D Beam, which simulates the flexural behavior in the element plane (suitable for modeling beam elements); and 3D Column, which simulates the interaction between axial and flexural behavior in both principal planes (suitable for modeling column elements). The mechanical behavior of these elements is determined by the cross-section geometry and the parameters of the materials of which the element is composed, considering non-linear constitutive laws. Strength domains are computed according to conventional formulations for reinforced concrete frame elements in design standards, e.g., NTC (2018). More details about these frame elements types can be consulted in Gruppo sismica (2020, 2022).

3.2.3 Non-linear constitutive laws

The characterization of the constitutive properties of the non-linear springs is carried out by imposing an equivalence between the macro-model and a corresponding homogeneous continuous model in order to dissociate each mechanism (flexural, diagonal shear, and sliding).

For the axial and flexural behaviors of the panel, a uniaxial elastoplastic law with limited deformability is assigned to the orthogonal springs to the interfaces, derived from the following masonry properties: elastic modulus, E ; compressive strength, σ_c ; tension strength, σ_t ; and strain limits of compression, ε_{cu} , and tension, ε_{tu} . The continuum behavior of the masonry in terms of stress-strain is grouped into the orthogonal springs to the interfaces through a procedure consistent with a fiber approach. In the calibration procedure, each axial/flexural spring is considered to correspond to a masonry strip equivalent to the spring's influence area and the panel's half-dimension in the direction perpendicular to the interface. This procedure originates the force-displacement constitutive laws inherited by the axial/flexural springs, described by: the initial stiffness, K_p ; the compressive and tensile yielding forces, F_{cy} and F_{ty} ; and the corresponding ultimate displacements, u_{cu} and u_{tu} . Further details can be found in Calì et al. (2012). The elastoplastic constitutive laws to define the axial-flexural behavior of the panel are portrayed in Figure 47.

Figure 47: Elastoplastic constitutive laws to define the axial-flexural behavior of the panel.



Source: Calì et al. (2012).

For simulating diagonal shear behavior, a symmetrical tensile-compressive elastoplastic law with given deformation limits is assigned to each of the two diagonal springs. The mechanical parameters of the diagonal springs are derived from a calibration process that imposes equivalence in terms of displacements between the macro-element and a pure shear deformable homogeneous panel, which means that the force-displacement relationship of each

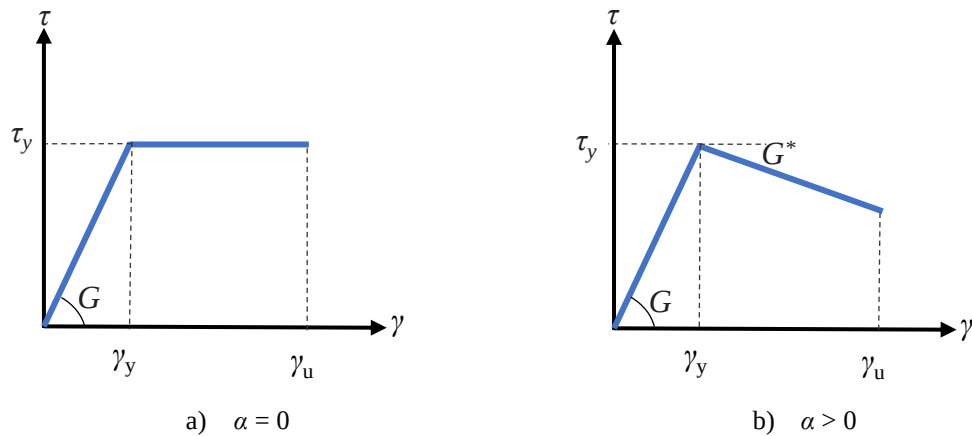
diagonal link is based on the shear modulus, G ; cross-sectional area, A_t ; and height, h , of the corresponding panel. The ultimate shear load and the corresponding displacement of each diagonal spring are therefore derived from the mechanical material properties of the masonry according to the adopted shear resistance criterion. More details are available in Calì et al. (2012).

Two possible resistance criteria can be adopted: Mohr-Coulomb and Turnsek and Čačovič (1970). According to D'Altri et al. (2022), the Mohr-Coulomb criterion is generally adopted for new masonry buildings, while the Turnsek and Čačovič criterion is usually used for existing masonry buildings. NTC (2018) suggests the Mohr-Coulomb criterion be used for masonry walls with regular bond pattern (“regular” masonry). This recommendation is based on the physical observation that the failure mechanism in these walls occurs by cracking with a “staircase” diagonal path along the bed and head mortar joints. In this study, the Mohr-Coulomb criterion was adopted, in which the yielding criteria is established according to Equation 5.

$$|\tau_y| = f_{v0} + \mu \cdot \sigma_n \quad \text{Eq. 5}$$

where: τ_y is yield shear stress; f_{v0} is shear strength in the absence of normal stress; μ is the coefficient of friction; σ_n is the compressive stress calculated as the average of the resultant forces of the orthogonal springs on the four sides of the element, divided by the cross-sectional area, A_t . The post-yield behavior is controlled by the kinematic hardening/softening parameter, α . This parameter is defined as the ratio between the post elastic shear modulus, G^* , and the initial shear modulus, G , and allows imposing the following post-yield behaviors: softening ($\alpha > 0$); hardening ($\alpha < 0$); and perfectly plastic ($\alpha = 0$). It is worth mentioning that the shear strength is updated step by step during nonlinear analysis as a function of the level of confinement, σ_n (D'altri et al., 2022). The constitutive laws for the calibration of diagonal springs, considering σ_n is constant, are exemplified in Figure 48.

Figure 48: Elastoplastic constitutive law for the calibration of diagonal springs.

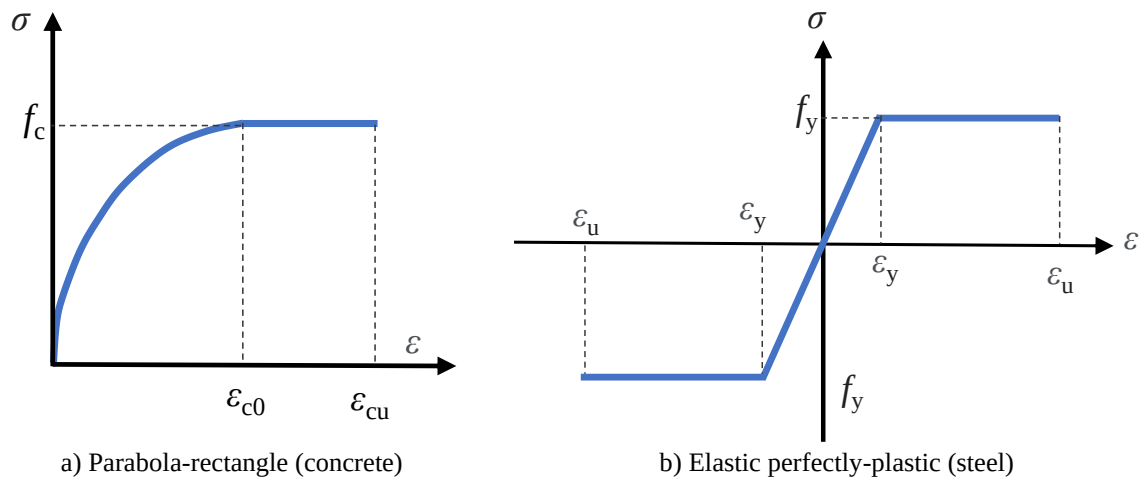


Source: Calì et al. (2012).

The deformation limit is given by the ultimate displacement of the panel, δ_u , expressed in terms of the ultimate angular deformation ($\gamma_u = \delta_u/h$), and its value can be evaluated experimentally or obtained from technical standards (EN 1998-3, 2005; NTC, 2018). Morandi et al. (2022) present values ranging between 0.2% and 0.4%, with the strictest limits applied to cases of higher axial loads (compressive stress level higher than 15% of compressive strength masonry).

Concrete and steel material models are defined for modeling composite-section frame elements of reinforced concrete. The constitutive laws are required for the generation of moment–curvature diagrams. A parabola–rectangle constitutive law with compressive strain limit and neglected tensile strength is adopted for concrete, as illustrated in Figure 48a. An elastic perfectly-plastic constitutive law with limited deformations and symmetrical behavior in tension and compression is adopted for steel, as shown in Figure 49.

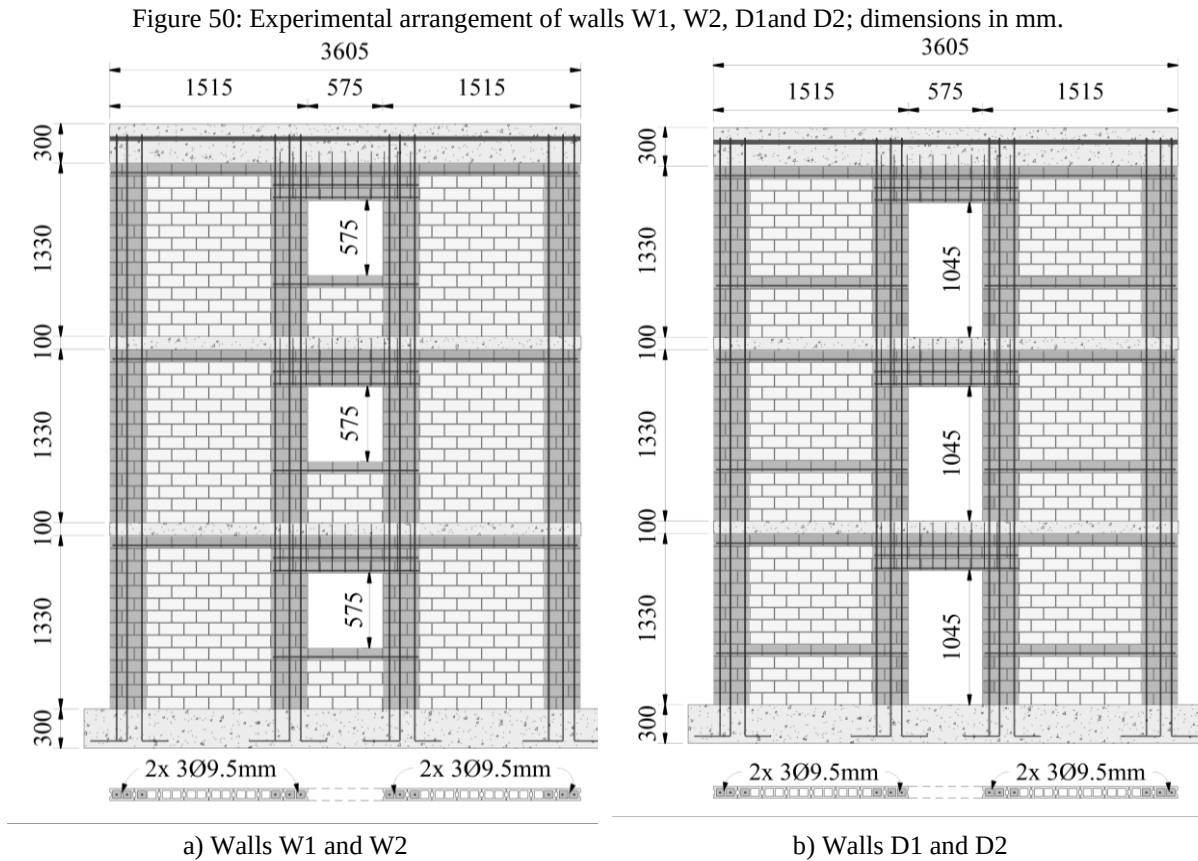
Figure 49: Constitutive laws for concrete and steel materials.



Source: Calì et al. (2012).

3.3 BRIEF DESCRIPTION OF THE EXPERIMENTAL TESTS

Four three-storey PGM walls with window and door openings are used herein as an experimental dataset Fortes et al. (2017; 2016). The experimental walls were built in running bond with half-scale hollow concrete blocks ($185 \times 90 \times 90 \text{ mm}^3$), being mortared with joints 5 mm deep only in the face shells, which had a uniform thickness of 15 mm each. Therefore, block webs have no significant effect on the in-plane stiffening of the ungrouted panels. Two walls, herein called W1 and W2, were built to be the same and had a horizontally centered window opening with dimensions of $575 \times 575 \text{ mm}^2$ on each storey. The other two walls, designated as D1 and D2, also the same, had a horizontally centered door opening with dimensions of $575 \times 1045 \text{ mm}^2$ on each storey. Two specimens were tested in each design case for representativeness purposes and to check for eventual deviations in each case/test. The structural details of these walls are presented in Figure 50.



Source: Adapted from Fortes (2017).

The walls had inter-storey reinforced concrete slab flanges with a width of 330 mm and reinforced masonry beams above the openings coupling the wall piers. In addition, on the top floor, there was a reinforced concrete beam where vertical and lateral forces were applied.

Regarding the vertical reinforcement detailing, a steel bar with a diameter of 9.5 mm was placed in each of the first three cells of blocks adjacent to the ends along all stories. For the horizontal reinforcement detailing, the walls had a complete course grouted and reinforced (bound beam) containing a steel bar with a diameter of 9.5 mm at the top of each storey. Furthermore, while walls D1 and D2 had their fifth course completely grouted and reinforced, walls W1 and W2 had their fifth course grouted and reinforced just below the window opening extending through one and a half blocks on each side of the opening. Two horizontal steel bars with a diameter of 9.5 mm were used in the course immediately above the opening, and one bar of the same type in the subsequent course. The two grouted and reinforced courses above the openings were the length of the opening plus one and a half blocks on each side. Simple stirrups with a diameter of 4.2 mm were placed every 100 mm connecting the courses above the openings with the reinforced concrete slab wing, thus forming the masonry beam.

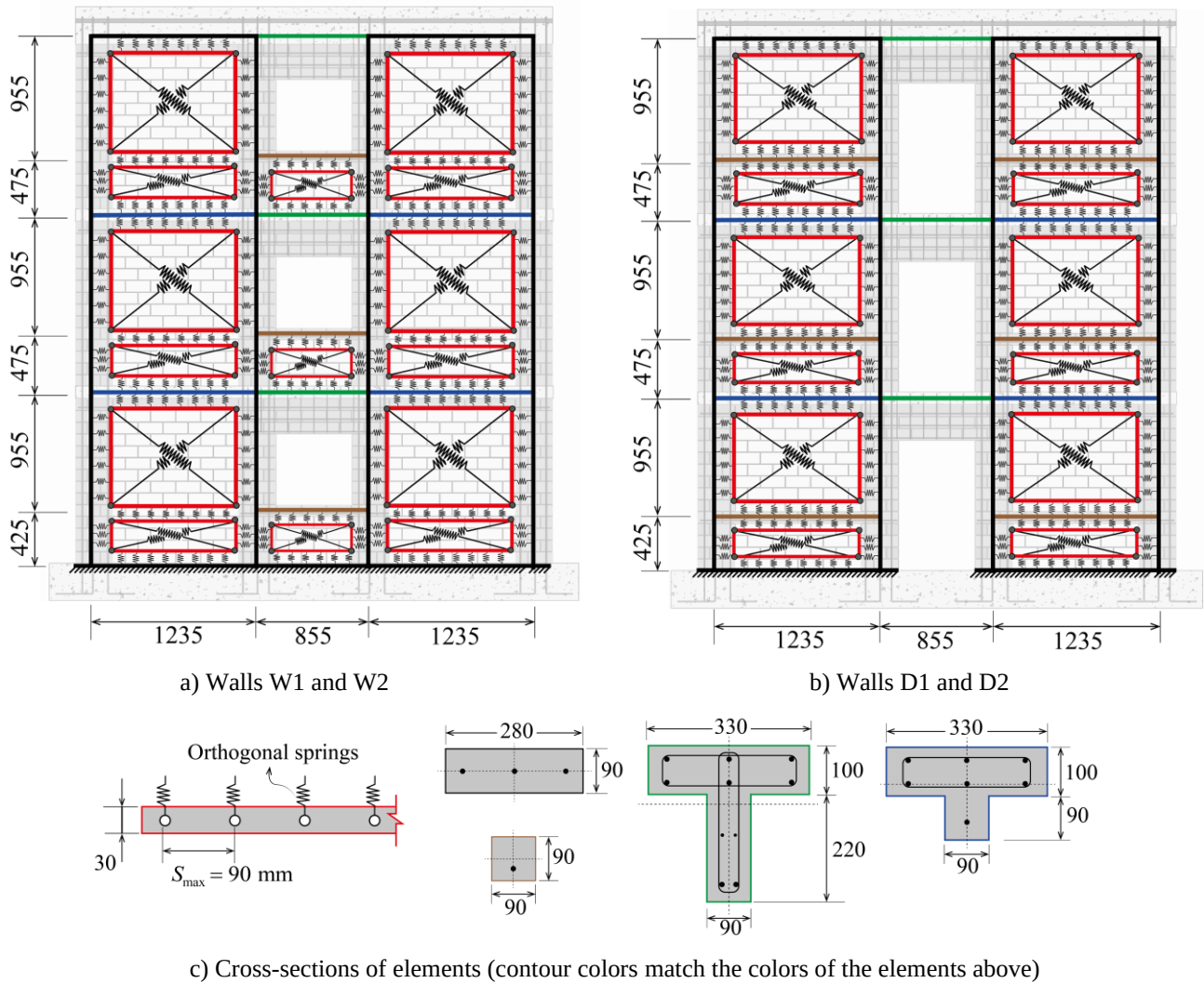
The walls were subjected to reversed cyclic in-plane loads applied to the upper reinforced concrete beam with displacement control. In addition, a distributed axial load of approximately 50 kN was applied evenly along its top, corresponding to 14 kN/m. Further details regarding the experimental tests can be found in Fortes et al. (2017; 2016).

3.4 DESCRIPTION OF THE NUMERICAL MODEL

The experimental walls described above were simulated using the macro-modeling approach introduced in Section 2 with the aid of 3DMacro[®] software (Caliò et al., 2012). Taking advantage of the possibility of simulating the interaction between the macro-elements and the frame elements, the ungrouted masonry portions were modeled with macro-elements, while the vertical grouted masonry parts and the lintel/slab inter-storey assemblages were modeled with frame elements. The configurations of numerical models concerning the geometry of the experimental walls are presented in Figure 51. A simplified representation of the macro-element is reported, although the element interfaces have zero thickness, as mentioned in Section 2.

This modeling strategy is inspired by the work by Medeiros et al. (2022a) who demonstrated, based on the same experimental walls evaluated here, that the concentration of grout and reinforcement at the ends seems to have induced the structure to behave as a continuous frame with confined masonry. Other research also shows that the mechanical behavior of PGM walls with large spacing between grout/reinforcement points may resemble that of confined masonry (Bolhassani et al., 2016b; Calderón et al., 2020; Dhanasekar; Haider, 2011; Koutras; Shing, 2021b; Minaie et al., 2010).

Figure 51: Configuration of the numerical models; dimensions in mm.



Source: Author.

All elements in the model have the net effective cross-section of the corresponding part of the wall. Therefore, the grouted masonry portions have a thickness of 90 mm, corresponding to the total thickness of the blocks, while the ungrouted parts have a thickness of 30 mm, equal to the sum of the thicknesses of the two face shells of the hollow concrete blocks.

The properties used for modeling the ungrouted masonry portions, associated with each resistant mechanism of the macro-element, are summarized in Table 2. The ultimate angular deformation was assumed to equal 0.4%, which constitutively governs the diagonal springs. This limit is in agreement with other studies studies (D'Altri et al., 2022; Morandi et al., 2022; Pantò et al., 2015). The compressive and tensile ductility values of the flexural springs, given by the corresponding ultimate to yield strain ratio (ϵ_u/ϵ_y), were adopted equal to 2.7 and 42.5, respectively. These values were defined to match the ultimate strains of 0.3% for compression

and 0.1% for tension, as adopted by Pirsahab et al. (2021). The self-weight was considered equal to 22.5 kN/m³.

Table 2: Material properties adopted in macro-elements.

Mechanism	Parameter	Value	Reference
Flexural	Elastic modulus (E)	10,620 MPa ($=900f_m$)	TMS 402-602 (2022)
	Compressive strength (f_m)	11.8 MPa	Test (2017)
	Tensile strength (f_{tm})	0.25 MPa	NBR 16868 (ABNT, 2020)
	Ductility in compression	2.7	Pirsahab et al. (2021)
	Ductility in tension	42.5	Pirsahab et al. (2021)
Diagonal Shear	Shear modulus (G)	4,248 MPa ($=0.4E$)	TMS 402-602 (2022)
	Yield criterion	-	Mohr-Coulomb
	Initial shear resistance (f_{v0})	0.20 MPa	EN 1996-1 (2005)
	Coefficient of friction (μ)	0.50	NBR 16868 (2020)
	Ultimate deformation (γ_u)	0.40%	(D'altri et al., 2022; Morandi et al., 2022; Pantò et al., 2015)
	Kinematic hardening/softening parameter (α)	0.03	Calibrated value

Source: Author.

The properties assigned to the frame elements are listed in Table 3. The concrete properties of the elements that simulate the slab in the experimental walls were disregarded in the model, considering them, therefore, with the same properties of grouted masonry.

Table 3: Material properties adopted in frame elements.

Material	Parameter	Value	Reference
Concrete/Grouted masonry	Elastic modulus (E_c)	10,980 MPa ($=900 f_m$)	TMS 402-602 (, 2022)
	Compressive strength (f_c)	12.2 MPa	Test, Fortes (2017)
	Ultimate comp. strain (ϵ_{cu})	0.3%	NBR 16868 (2020)
	Transition comp. strain (ϵ_{c0})	0.2%	Medeiros et al. (2022a)
	Self-weight	22.5 kN/m ³	Medeiros et al. (2022a)
Steel	Elastic modulus (E_s)	203,512 MPa	Test, Fortes (2017)
	Yield strength (f_y)	540 MPa	Test, Fortes (2017)
	Ultimate strain (ϵ_{yu})	8.0%	Test, Fortes (2017)
	Self-weight	78.5 kN/m ³	Usual value

Source: Author.

Different types of frame elements were used. The vertical elements were modeled as a 3D column type considering the interaction between axial and flexural behavior. The horizontal elements at the top of each floor were modeled as 2D beam elements, simulating the in-plane flexural behavior. The horizontal elements equivalent to the intermediate bond beam were considered with only axial behavior (1D type).

In addition to the self-weight of the structure, a vertical loading uniformly distributed along the horizontal elements on the top floor was considered to account for the axial load applied in the test. The upper beam of reinforced concrete used in the experimental test (Figure 49) was not explicitly modeled; however, its self-weight was considered in addition to the existing vertical load, totaling a vertical load of 15.52 kN/m.

In the first stage of the analysis, vertical loading was applied. Then, pushover analysis was performed to simulate the horizontal loading. The horizontal forces were applied according to a distribution proportional to the fundamental vibration mode of the walls in their plane. This consideration agrees with the observation by Medeiros et al. (2022a) that the experimental walls behaved like a solid composite with shear deformations and an approximately linear lateral load distribution along the height, up to the maximum lateral load level.

The pushover horizontal loading was performed in two successive phases: the first, controlled by forces, lasts until the point where the stiffness matrix becomes singular (an increment in force is no longer possible); the second, with displacement control, proceeds with displacement increments until conventional collapse. A force–displacement relationship is obtained, allowing evaluation of the softening phase after the peak lateral force and investigation of the ductility of the models.

3.5 COMPARISON BETWEEN THE NUMERICAL AND EXPERIMENTAL RESULTS

The comparison between the numerical and experimental results was based on the similarity of the force–displacement curves, the lateral displacement profiles along the height of the wall, and the damage mechanisms. In addition, results of robust two-dimensional finite element models (FEM) developed by Medeiros et al. (2021, 2022a, 2022b) using these same experimental tests were included in the analysis. Briefly, the FEM were developed using the program VecTor2 (VecTor Analysis Group, 2019; Wong; Vecchio; Tamm, 2013), which is based on the Modified Compression Field Theory (MCFT) (Vecchio; Collins, 1986) and the Disturbed Stress Field Model (DSFM) (Vecchio, 2000). A comprehensive description of the FEM is provided in Medeiros et al. (2022a).

3.5.1 Force–displacement response

The results were initially evaluated by comparing force–displacement curves with the displacement monitored at the wall top. The force–displacement curves obtained from the experimental tests and FEM were defined as the average of the envelopes extracted from the hysteresis curves in the push and pull directions. The ultimate displacement of the wall, d_{Vu} , was determined for a lateral resistance degradation of 20% of the peak resistance, V_{max} .

The force-displacement graphs of all responses are presented in Figure 52. The top drift values (displacement/height ratio) relative to the base of the wall are also indicated on the upper horizontal axis. Note that the complete responses of the walls (W1 and W2) and (D1 and D2) are presented in Figure 52a and Figure 52c, respectively; while a zoom of the initial phase of the responses is provided in Figure 52b and Figure 52d, respectively. The light red area in the graphs indicates the variability between the experimental responses.

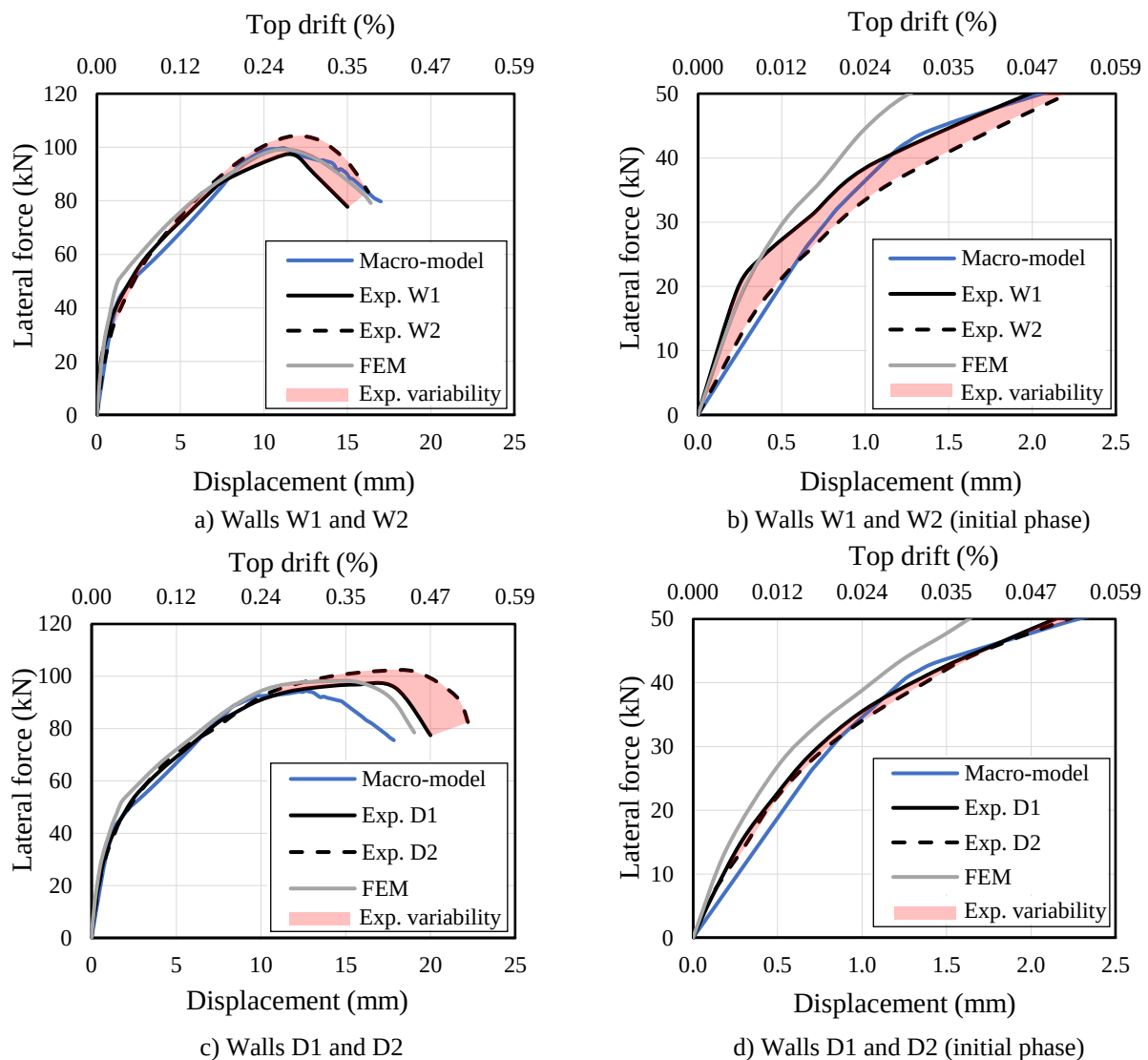
The values of maximum lateral force, V_{max} ; displacement at the level of the maximum lateral force, d_{Vmax} ; ultimate displacement, d_{Vu} ; and secant stiffness, K_{sec} , of all models are summarized in Table 4. The secant stiffness, K_{sec} , was evaluated by considering two approaches. In the first one, K_{sec1} was defined as the stiffness that intercepts the force–displacement curve at a force level of 25 kN. In the second one, K_{sec2} was determined at a level ranging from 5% to 33% of the maximum lateral force. The last approach follows a common procedure in design standards, e.g., TMS 402/602 (2022) and CSA S304-14 (2014), for the modulus of elasticity calculation and may be more appropriate for design purposes.

Table 4: Summary of the main response measures of force–displacement curves.

Wall	Model	Force (kN)	Displacements (mm)		Stiffness (kN/mm)	
		V_{max}	d_{Vmax}	d_{Vu}	K_{sec1}	K_{sec2}
W1 and W2	Macro-model	99.7	11.2	17.0	40.4	37.9
	Exp. W1	97.2	11.8	15.0	62.1	40.8
	Exp. W2	104.1	11.7	16.3	38.8	30.1
	Avg. Exp.	100.7	11.8	15.6	50.4	35.4
	FEM	98.9	11.7	16.4	62.6	51.4
D1 and D2	Macro-model	94.4	12.6	17.8	37.5	36.0
	Exp. D1	96.8	15.5	20.0	43.4	35.9
	Exp. D2	102.0	17.0	22.3	41.5	31.9
	Avg. Exp.	99.4	16.3	21.1	42.4	33.9
	FEM	98.1	13.9	19.1	54.7	43.5

Source: Author.

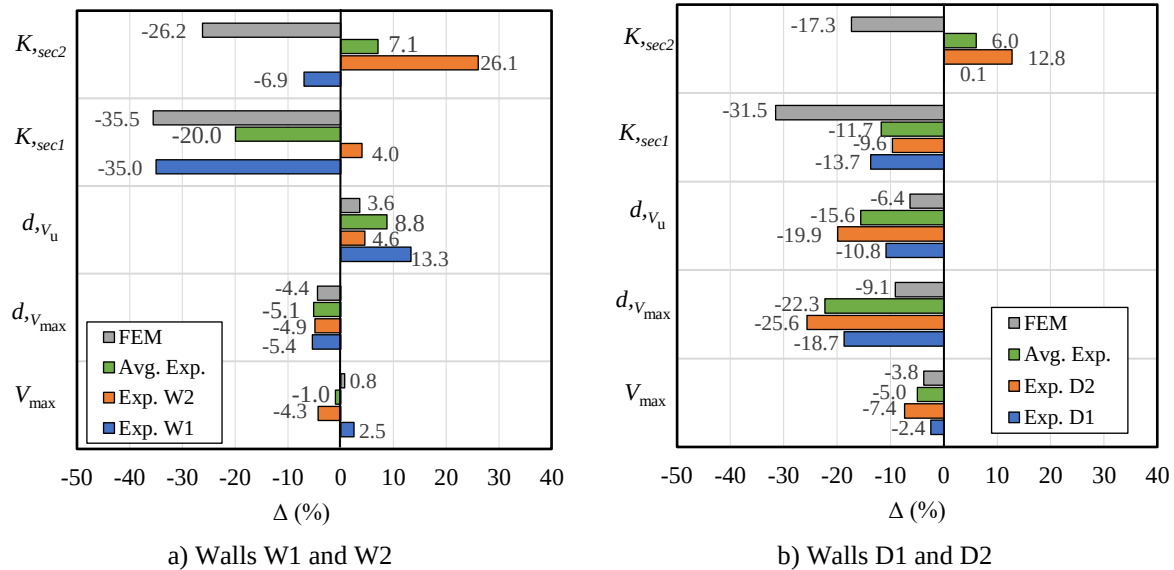
Figure 52: Force–displacement curves of all models.



Source: Author.

The percentage differences between the results of the macro-models and the ones corresponding to the other responses are shown in Figure 53. Examination of Figure 52 and Figure 53, and Table 4 reveals that the force-displacement curves of all models are very similar in the initial phase, in terms of maximum lateral force, ultimate displacements, and post-peak behavior. The variability between the experimental results themselves when comparing the responses of the supposedly identically constructed walls (W1 with W2 and D1 with D2) should be noted. This variability indicates the complexity involved in the experimental tests and their numerical representation through simplified models.

Figure 53: Percentage differences between the results of macro-models and the other responses.



Source: Author.

The results of maximum lateral force, V_{max} , were estimated by the macro-model with good accuracy. Regarding the experimental walls W1 and W2, the results indicate relative differences (compared to the macro-model) of 2.5% and -4.3%, respectively, for the two experimental tests and a difference of -1.0% for the experimental average. Similarly, for walls D1 and D2, a difference of -5.0% is observed for the average experimental result. In absolute values, the most significant difference in relation to the estimated maximum lateral force was 7.6 kN for the experimental wall D2.

Regarding the d_{vmax} and d_{vu} displacements, the largest percentage differences in relation to the macro-model for walls W1 and W2 were -5.4% and 13.3% concerning wall W1. In absolute values, these differences correspond to 0.6 mm and 2.0 mm, respectively. For walls D1 and D2, the macro-model could not reproduce these displacement phases with the same level of accuracy. Considering the macro-models results compared to the average experimental values, conservative relative differences of -22.3% and -15.6% were obtained for d_{vmax} and d_{vu} , respectively. In absolute values, these differences correspond to 3.7 mm and 3.3 mm. Comparing the macro-model results with the respective FEM, better agreement is found for d_{vmax} and d_{vu} for all walls.

For the initial phase of the responses, as shown in Figure 52b and Figure 52d, a similar trend may be seen. However, differences in the initial stiffness are observed. In general, the macro-models tend to be less stiff than the experimental walls, while the FE models tend to be stiffer than the experimental walls. For the secant stiffness, K_{sec1} , the difference between the

macro-models and average experimental results of the W1 and W2 walls was -20.0%, while in relation to D1 and D2 walls, this difference was -11.7%. Regarding the secant stiffness, K_{sec2} , the values predicted by macro-models and the average experimental results were quite similar for both wall types. Differences of 7.1% for the walls with window openings and 6.0% for the walls with door openings show good agreement in the initial stiffness estimated by macro-models in a design context.

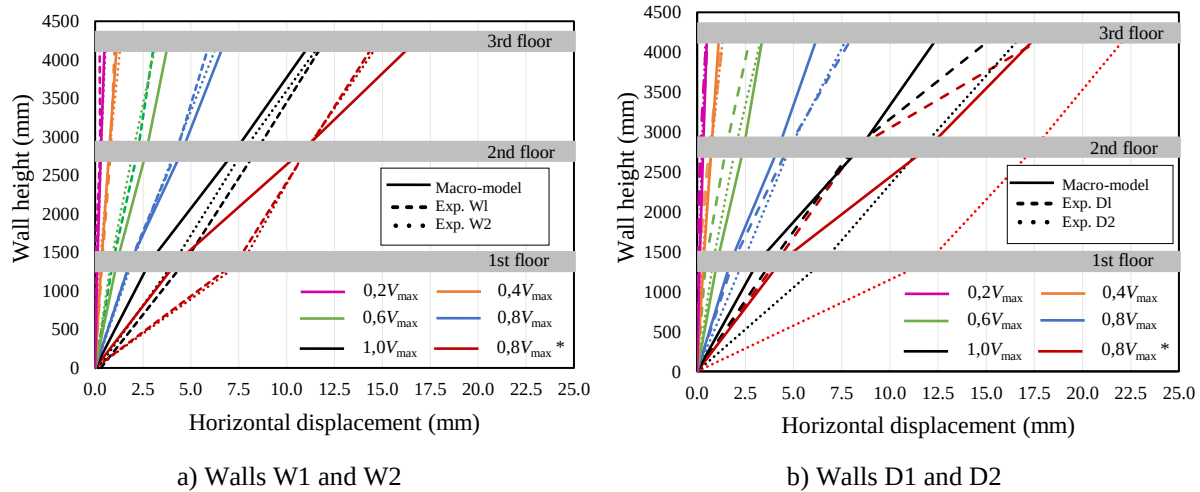
3.5.2 Lateral displacement profile along the height

To evaluate the evolution of displacement along the height of the walls, the monitored displacements at the levels of each floor of the experimental walls and macro-models are shown in Figure 53, considering the force levels of $0.2V_{max}$, $0.4V_{max}$, $0.6V_{max}$, $0.8V_{max}$, V_{max} and $0.8V_{max}^*$. The latter corresponds to a post peak degradation of 20% of the maximum lateral force capacity. The displacement profiles of the experimental walls result from calculating the average values of the push and pull stages of the cyclic test.

It is possible to observe in Figure 54, again, a good agreement between the macro-models and the experimental walls, both for walls W1 and W2 and walls D1 and D2, especially when comparing the displacement profiles from the initial phases to the peak force. The displacement profiles denote an approximately linear response from the base to the top of the wall, up to the maximum lateral force phase, implying that inter-storey drifts are about equal.

In the post-peak stage, a non-linear trend of the horizontal displacement profiles due to different inter-storey drifts is apparent. For the experimental walls W1 and W2, higher inter-storey drifts were obtained regarding the first floor, i.e., 0.54% and 0.56%, respectively, against drifts of approximately 0.25% for the other floors. This trend was not perfectly predicted by the macro-model, which, in turn, estimated the highest inter-storey to occur on the second floor, with a value of 0.45%, against drifts of 0.31% and 0.42% for the first and third floors, respectively.

Figure 54: Experimental and numerical horizontal displacement profiles.



Source: Author.

For the experimental walls D1 and D2, different trends from each other in the post-peak stage may be observed, demonstrating the variability of the behavior of masonry walls, especially in this stage of severe damage (post-peak). While wall D1 showed the highest inter-storey drift on the third floor (0.71%), for wall D2, the highest inter-storey drift was achieved on the first floor (0.87%). With the macro-model, the highest estimated inter-storey drift for these walls was 0.45% on the second floor. This may explain in part the less accurate estimate of the force–displacement curve for walls D1 and D2 relative to walls W1 and W2 at this stage of behavior (Figure 52a and Figure 52c).

3.5.3 Damage mechanisms

In all experimental walls, a shear failure mode was observed characterized by diagonal cracks following the path of the bed and head mortar joints (Fortes, 2017; Medeiros et al., 2022b). Fortes (2017) describes that failure began with the development of horizontal flexural cracks, which were later replaced by wide and open diagonal cracks that extended through the panels of the walls in all stories. There is no indication of a specific storey where diagonal cracking developed predominantly. However, Fortes (2017) reports that, in some walls, the main cracks occurred in the third storey, while other walls had a predominance of cracking in the first storey. The diagonal shear crack pattern observed in the tests is shown in Figure 55.

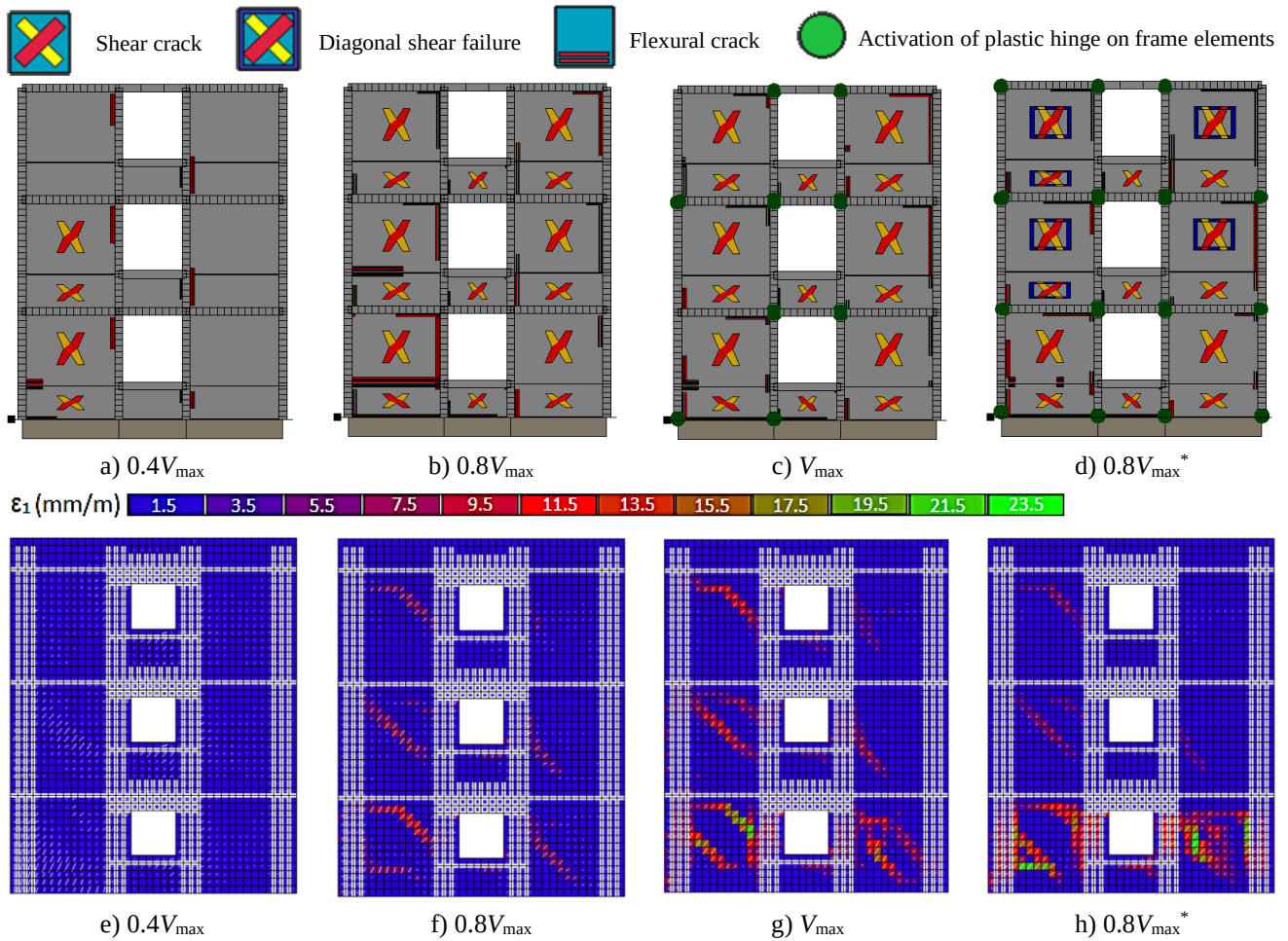
Figure 55: Damage mechanisms observed in experimental tests.



Source: Author.

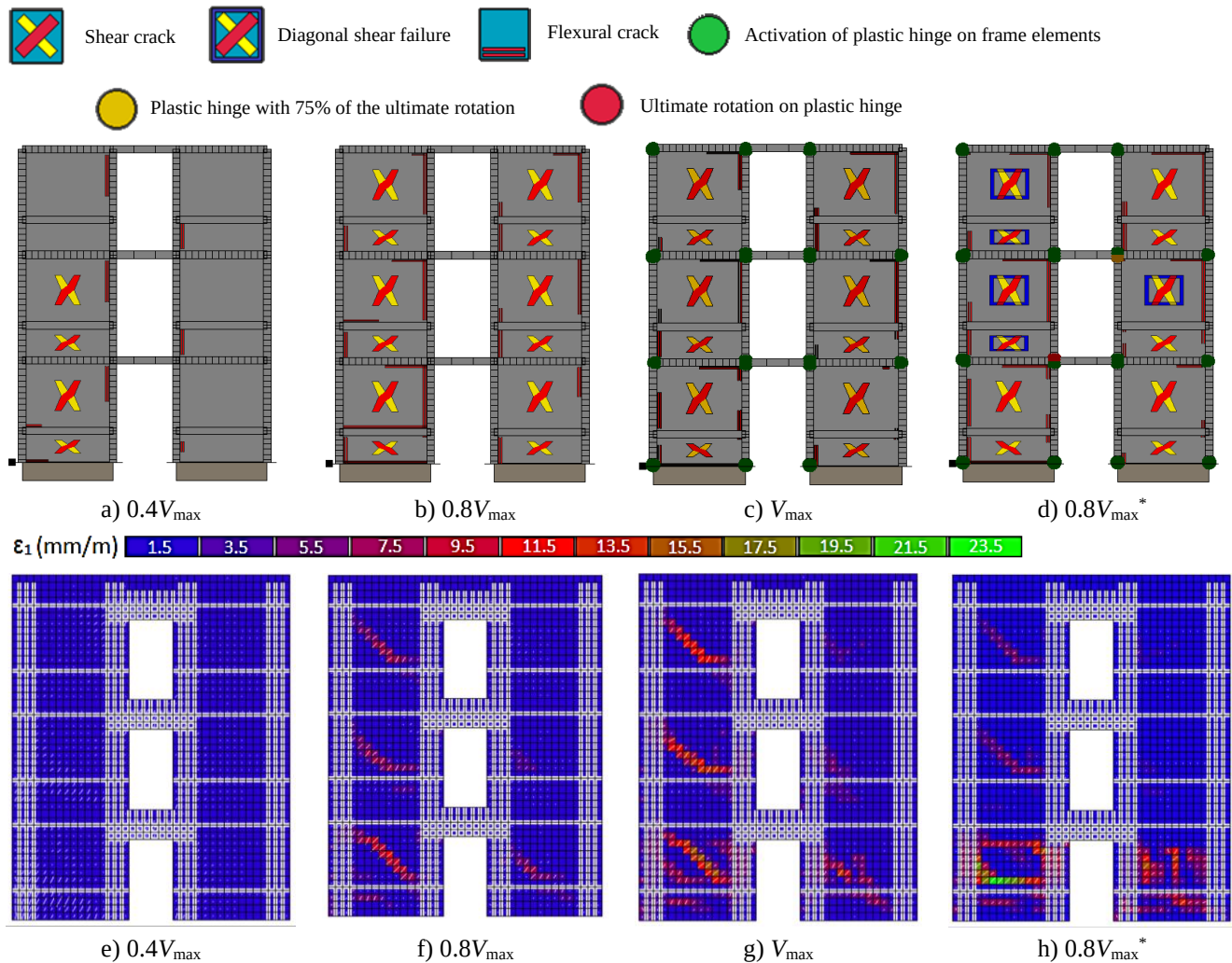
The evolution of damage mechanisms predicted by the macro-models and the FE models are illustrated in Figure 56 and Figure 57 for the walls with window and door openings, respectively. The damage sequence is evaluated considering the force steps of $0.4V_{\max}$, $0.8V_{\max}$, $V_{\max}$, and $0.8V_{\max}^*$. In addition to pictures of the post-peak stage from the experimental benchmarks (Figure 55), the FEM scheme of damage provides a visual idea of what occurred in these experimental walls.

Figure 56: Evolution of damage mechanisms of walls W1 and W2 for the macro-model, from (a) to (d) and FEM, from (e) to (h).



Source: Author.

Figure 57: Evolution of damage mechanisms of D1 and D2 walls for the macro-model, from (a) to (d) and FEM, from (e) to (h).



Source: Author.

It can be seen in Figure 56 and Figure 57 that the failure mode predicted by both models is governed by the diagonal shear mechanism, which agrees with the experimental observations.

For the macro-models, the presence of frame elements simulating the grouted/reinforced masonry portions seem to inhibit the flexural mechanism by partially restricting the rotation of ungrouted masonry panels. In the initial stages of loading, flexural cracks developed at the base of the wall together with vertical cracks arising from the interaction of the macro-elements with the frame elements: diagonal shear cracks in some panels (macro-elements) also appeared. These cracks indicate that such macro-elements are in a non-linear regime. As the lateral force increases, diagonal shear cracks develop in all panels of all stories. Plastic hinges on frame elements are activated at the peak lateral force (Figure 56c and Figure 57c). All these mechanisms acting simultaneously mobilize the maximum resistance of the structural model.

In the post-peak phase, some panels of the second and third stories reach their deformation limits and, therefore, they fail in diagonal shear (Figure 56d and Figure 57d). When a panel reaches its deformation limit, the corresponding diagonal springs lose their ability to resist shear force, and load redistribution occurs locally in the structure.

For the FE models, noticeable cracks developed diagonally in the central ungrouted region on each pier of both wall types, increasing in width gradually until failure. Comparing the damage evolution in the FE and the macro-models, the overall crack patterns were similar, indicating diagonal cracks in the ungrouted masonry regions of the three stories. Differences were noted in the post-peak phase. In the FE models, major cracks were mainly noted in the first storey, while in the macro-models, the ultimate shear deformation was reached in the panels (macro-elements) of the upper stories. The failure of these macro-elements agrees with the analysis of inter-storey drifts mentioned earlier since the inter-storey drifts estimated by the macro-model in this phase were higher for the second and third stories, respectively. It may be a limitation of the macro-models in assuming the same ultimate shear deformation for all panels at all stories. Since the bottom panels are subjected to a higher axial load level, their ultimate shear deformation may be smaller. In contrast, the FE model takes into account the effect of the actual axial stress on the ultimate shear deformation. However, this difference in the post-peak damage scenario did not affect the global load-displacement result.

3.6 SENSITIVITY ANALYSIS OF THE NUMERICAL MODEL

A sensitivity study of the macro-model was performed to demonstrate the influence of specific parameters on the force–displacement response. For objectivity, the analyses were performed using walls W1 and W2 as a reference, except for the analysis of the axial load level, in which the influence on the models of walls D1 and D2 was also evaluated. When a specific parameter was changed, the other parameters were kept the same as those listed in Table 2 and Table 3.

3.6.1 Axial load level

Studies show that the level of axial pre-compression significantly affects the structural behavior of masonry walls, especially in terms of lateral force capacity and ductility: increasing axial load increases lateral force capacity while decreasing ductility (Haach, 2009; Medeiros et al., 2022a; Voon; Ingham, 2006). Therefore, it becomes relevant to evaluate the influence of the axial load level on the results estimated with the macro-models.

The experimental walls were tested only for axial compressive stress at the base of the wall (including self-weight) corresponding to about 4% of the masonry compressive strength, f_m . However, to simulate axial load conditions closer to an actual design situation for mid and/or high-rise masonry buildings, numerical results obtained through the FEM, as well as the estimation of the maximum lateral force capacity by the analytical equation of Medeiros et al. (2022b), were used to check the consistency of the macro-models in this aspect. For this, an axial compressive stress level of approximately $0.2f_m$ was considered.

The analytical equation was proposed for unperforated masonry walls and then adapted for perforated walls using approaches based on the strength of the wall piers with dimensions limited by different possibilities for the diagonal shear cracks. The distinction between these approaches is where the diagonal shear crack is assumed to start and finish in the panels, which implies different dimensions for the panels and, consequently, different shear capacities. The expression is presented in Equation 6; further details can be found in Medeiros et al. (2022b).

$$V_n = k_{gv}k_{gh}\beta A_{ch}\sqrt{f'_m} + 0.4P\tan\theta + 0.02A_v f_{yv}\sqrt{f'_m} + 0.02\rho_h A_{ev} f_{yv}\sqrt{f'_m} \quad \text{Eq. 6}$$

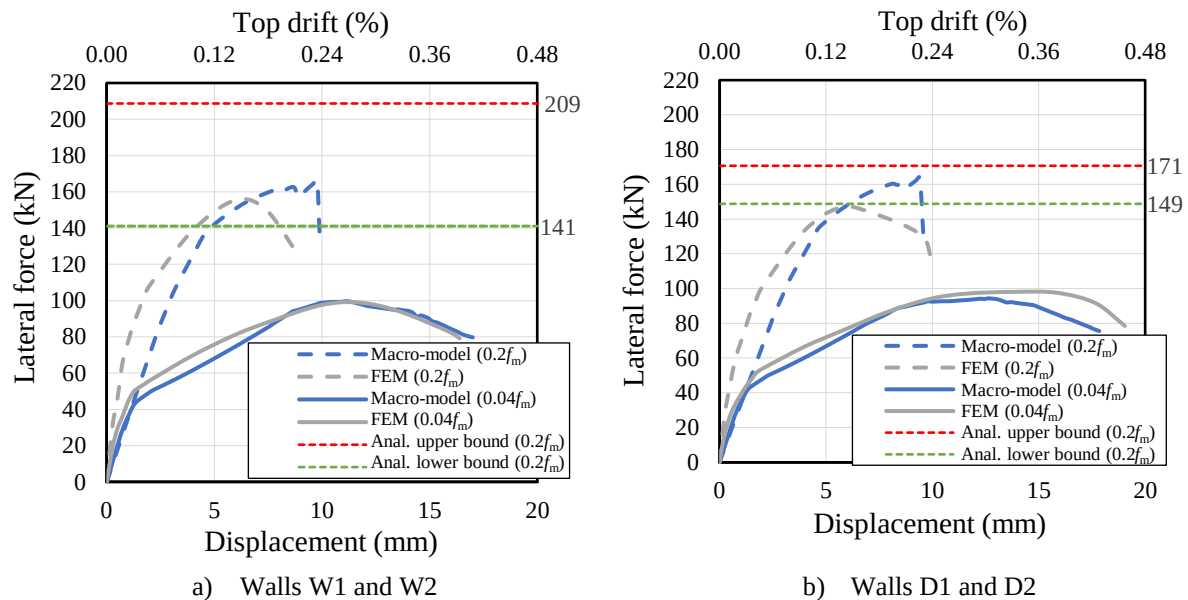
where: V_n is the nominal shear load capacity; k_{gv} and k_{gh} are the coefficients to account for vertical and horizontal grouting, respectively; β is the coefficient to account for the shear span ratio; A_{ch} is the effective horizontal cross-sectional area of masonry; f'_m is the net compressive strength of masonry; P is the axial load; θ is the angle formed between the vertical wall axis and the strut; A_v is the total cross-section of vertical reinforcement; f_{yv} is the yield strength of vertical reinforcement; ρ_h is the net horizontal reinforcement ratio; A_{ev} is the effective vertical cross-sectional area of masonry.

For this analysis, the value of the ultimate shear deformation for the macro-elements was reduced from 0.4% to 0.2% to assess the influence of the higher axial load on both lateral force capacity and ductility. These values are in agreement with the work by Morandi et al. (2022), who considered a range of values from 0.2% to 0.4% for the ultimate shear deformation capacities of URM walls, depending on the pre-compression level. However, it is noteworthy that further investigation into how the ultimate shear deformation varies as a function of axial load is still necessary, especially in partially grouted masonry walls.

The force–displacement curves obtained from the macro-model and FEM, both for the axial compressive stress of $0.04f_m$ and $0.2f_m$, are shown in Figure 58. In addition, values of the lower and upper bounds of the maximum lateral force predicted by the analytical equation of

Medeiros et al. (2022b), assuming the more and less conservative approaches, are also plotted for cases of axial compressive stress of $0.2f_m$. The axial compressive stress level corresponding to each response is specified in the graph legend. The top drifts are also shown on the top horizontal axis.

Figure 58: Force–displacement curves for different values of axial compressive stress.



Source: Author.

In Figure 58, it is possible to see that the increase in lateral force capacity and the reduction in ductility due to the increase in axial load level, following literature observations, were captured by the macro-model. In fact, according to the Mohr-Coulomb yield criterion adopted for the macro-elements, the shear strength increased proportionally to the compressive stress. Furthermore, the responses obtained with the macro-models were consistent with those of the FEM and with the maximum lateral force range estimated from the analytical equation.

For walls W1 and W2, the maximum lateral force values estimated with the macro-model and the FEM were 166.2 kN and 155.3 kN, respectively, which corresponds to a relative difference of 7.1%. Both values are within the range of the analytical equation considering the most and least conservative approaches, respectively, 141.0 kN and 208.6 kN. The ultimate displacement values, d_{Vu} , were 9.9 mm for the macro-model and 9.0 mm for the FEM and, therefore, with a relative difference of 10.7%.

For walls D1 and D2, the maximum lateral force values estimated with the macro-model and the FEM were 163.8 kN and 146.7 kN, respectively, corresponding to a relative difference of 11.7%. The value predicted by the macro-model is within the range of the analytical equation

considering the most and least conservative approaches, respectively, 148.8 kN and 170.6 kN. Despite not falling into this range, the maximum lateral force estimated with the FEM is very close (-1.41%) to the value using the most conservative approach of the analytical equation. The ultimate displacement values, d_{vu} , were 9.1 mm for the macro-model and 10.0 mm for the FEM and, therefore, with a relative difference of -9.0%.

The main difference observed between the models was that, as previously noted, the macro-models tended to be less stiff than the FEM. For a secant stiffness, K_{sec} , defined at the force level of 25 kN, the relative difference between the macro-model and the FEM was about -70.0% for the two types of walls.

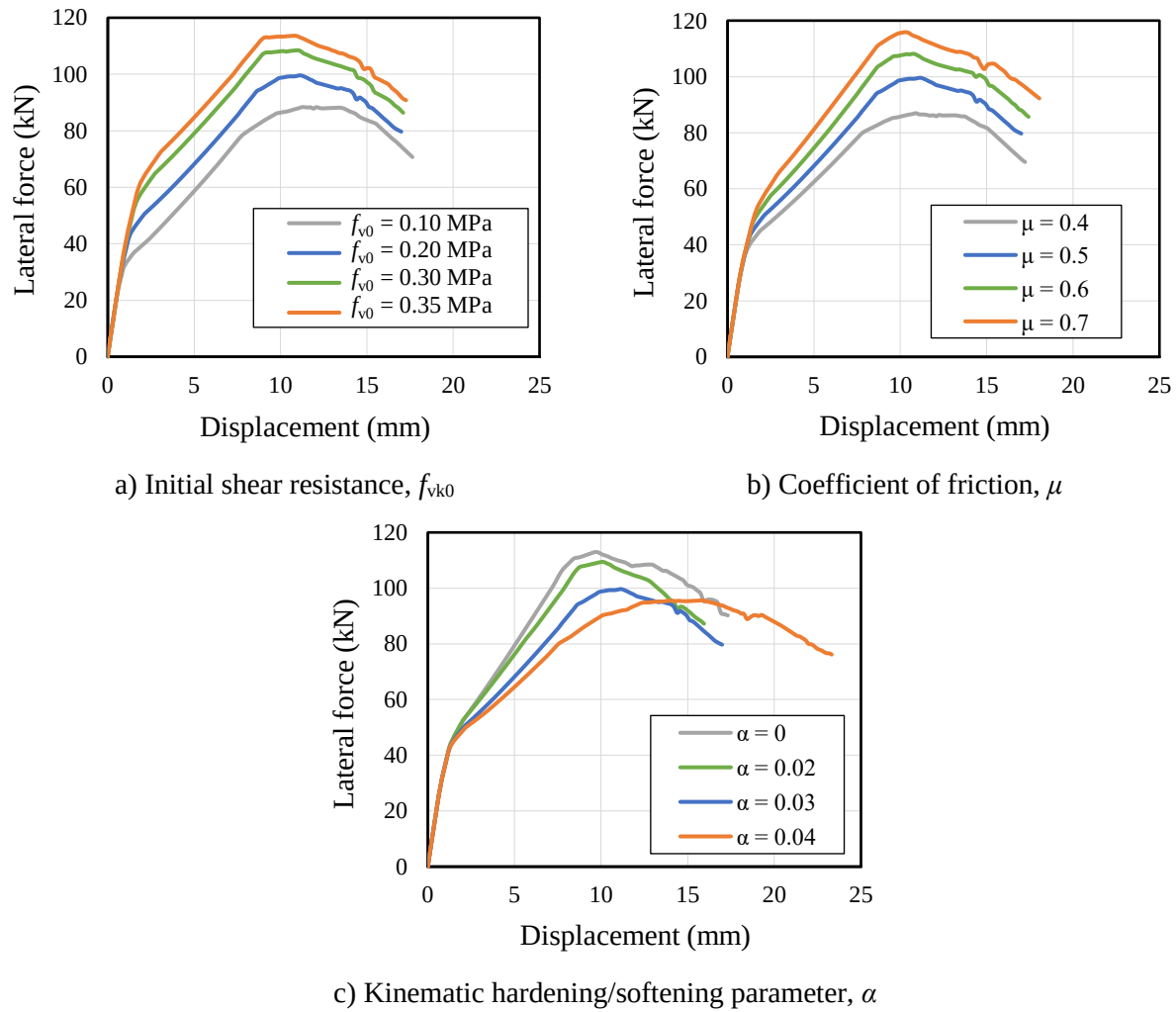
It is important to note that no upper limit of shear strength due to the level of the axial load was considered in the macro-model and the analytical equation, as is commonly recommended in design standards (EN 1996-1, 2005; NTC, 2018; TMS, 2016). This may be an issue to be considered in future studies.

3.6.2 Diagonal shear mechanism parameters

The diagonal shear mechanism is predominant in the non-linear behavior of the model for the evaluated walls. Thus, a sensitivity analysis of the parameters of the shear strength, i.e., initial shear strength, f_{v0} , and coefficient of friction, μ , as well as the kinematic hardening/softening parameter (α), was performed to assess the influence of each of these parameters on the model's global behavior. The force-displacement curves for all cases considered are shown in Figure 59. The variation range with the respective values of each parameter evaluated is indicated in the legends of the graphs.

As may be seen in Figure 59 the parameters analyzed have a significant influence on the response of the model. Within the range of values considered, increasing the values of the strength parameters f_{v0} and μ increased the level of lateral load after which an evident non-linear behavior begins, characterized by a loss of stiffness of the wall. In addition, increasing these parameters caused an increase in maximum lateral force capacity (Figure 59a and Figure 59b). Regarding the base model used in the evaluation of the experimental walls W1 and W2, the following percentage variations of V_{max} were observed: -11.3%, 8.8%, and 13.9% when changing f_{v0} from 0.20 MPa to 0.10 MPa, 0.30 MPa, and 0.35 MPa, respectively; and -12.7%, 8.6%, and 16.3% when changing μ from 0.5 to 0.4, 0.6, and 0.7, respectively.

Figure 59: Force–displacement curves for different parameter values of the diagonal shear mechanism (Walls W1/W2).



Source: Author.

These results demonstrate that increasing the values of the shear strength parameters increases the yield resistance, due to the extension of the elastic regime of the diagonal springs of the macro-elements that govern the behavior of the wall. The considerable influence of μ is worth noting, even though the model corresponds to the case of walls with a relatively low pre-compression level ($0.04f_m$). Probably, this influence would be even more expressive in walls with higher levels of axial compressive stress.

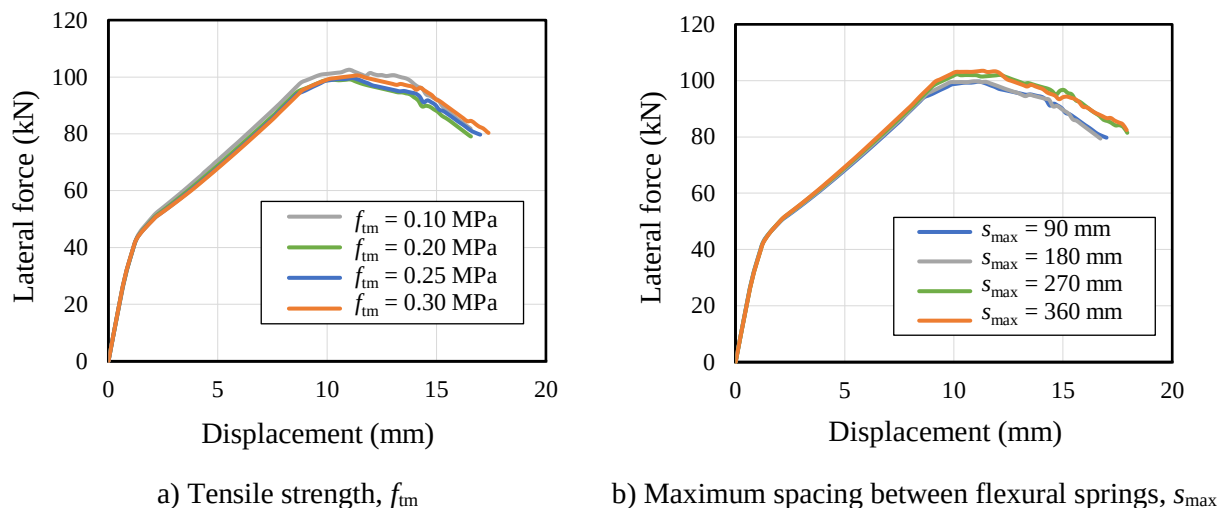
Regarding the kinematic hardening/softening parameter, α , considered in the constitutive behavior of the diagonal springs (Figure 59c), it was noticed that this parameter controls the stiffness of the model after the diagonal springs first yield. Increasing the value of this parameter caused a reduction in the model's stiffness after a force level of about 45 kN and in the maximum lateral force capacity. Compared to the case with $\alpha = 0$, the maximum lateral force capacity of the model with the highest α value ($\alpha = 0.04$) was reduced by 15.4%. The

introduction of the kinematic hardening/softening parameter appears adequate for simulating the cyclic degradation of the walls against the experimental results. However, it is emphasized that relatively high values of this parameter can result in an exaggerated ductility, as occurred when considering a value equal to 0.04 in the specific model here.

3.6.3 Flexural mechanism parameters

The force–displacement responses obtained by varying the tensile strength, f_{tm} , and the maximum spacing between the flexural springs, s_{max} , assigned to the macro-elements, are presented in Figure 60. It may be seen that these parameters do not have significant influence on the response of the model of these walls. The results show that the model behavior is mainly governed by the diagonal shear mechanism of the ungrouted masonry panels. Although the variation in f_{tm} causes almost no change in the global response of the models, it does result in different load distributions among the elements. Consequently, slight variations are observed in Figure 60a, mainly in the post-peak stage.

Figure 60: Force–displacement curves for different values of the flexural mechanism parameters (Walls W1/W2).



Source: Author.

Regarding s_{max} , a value must be chosen from which the number of orthogonal springs is sufficient to obtain an accurate solution. Decreasing this spacing from 360 mm to 90 mm caused a slight change in the response, mainly in the post-peak stage. The responses were practically the same when s_{max} was equal to 180 mm and 90 mm, indicating that the number of springs assigned would already be sufficient. As the walls were half-scale, 90 mm is the spacing between each cell of the hollow concrete blocks used in the construction of the walls.

Finally, changes in the masonry compressive strength, f_m , close to the experimental value, have no influence on the force–displacement response of the model, as the compression springs remain in a linear regime throughout loading. Despite this, as the stiffness properties (E and G) are known to be somewhat related to the compressive strength, f_m , the influence of these specific parameters will be demonstrated in the next subsection.

3.6.4 Stiffness parameters

In some standards, the stiffness of cracked masonry is considered (EN 1998-1, 2004; NTC, 2018). According to Castellazzi et al. (2022), the value of the elastic modulus, E , of the macro-element does not necessarily need to be reduced to consider the cracked condition, as is often done in equivalent frame models (Morandi et al., 2022). This is because the reduction in flexural stiffness is associated with the progressive yielding of springs orthogonal to the interfaces, even in an elastoplastic law.

Different analytical approaches for estimating E can be found in design standards such as (ABNT, 2020; EN 1996-1, 2005; TMS, 2022). To demonstrate the influence of E on the model, the force–displacement responses when applying the multiplier coefficients of 0.8, 0.9, and 1.1 in the stiffness parameters, E and G , of the base model ($E = 900f_m$ and $G = 0.4E$) are shown in Figure 61a and Figure 61b.

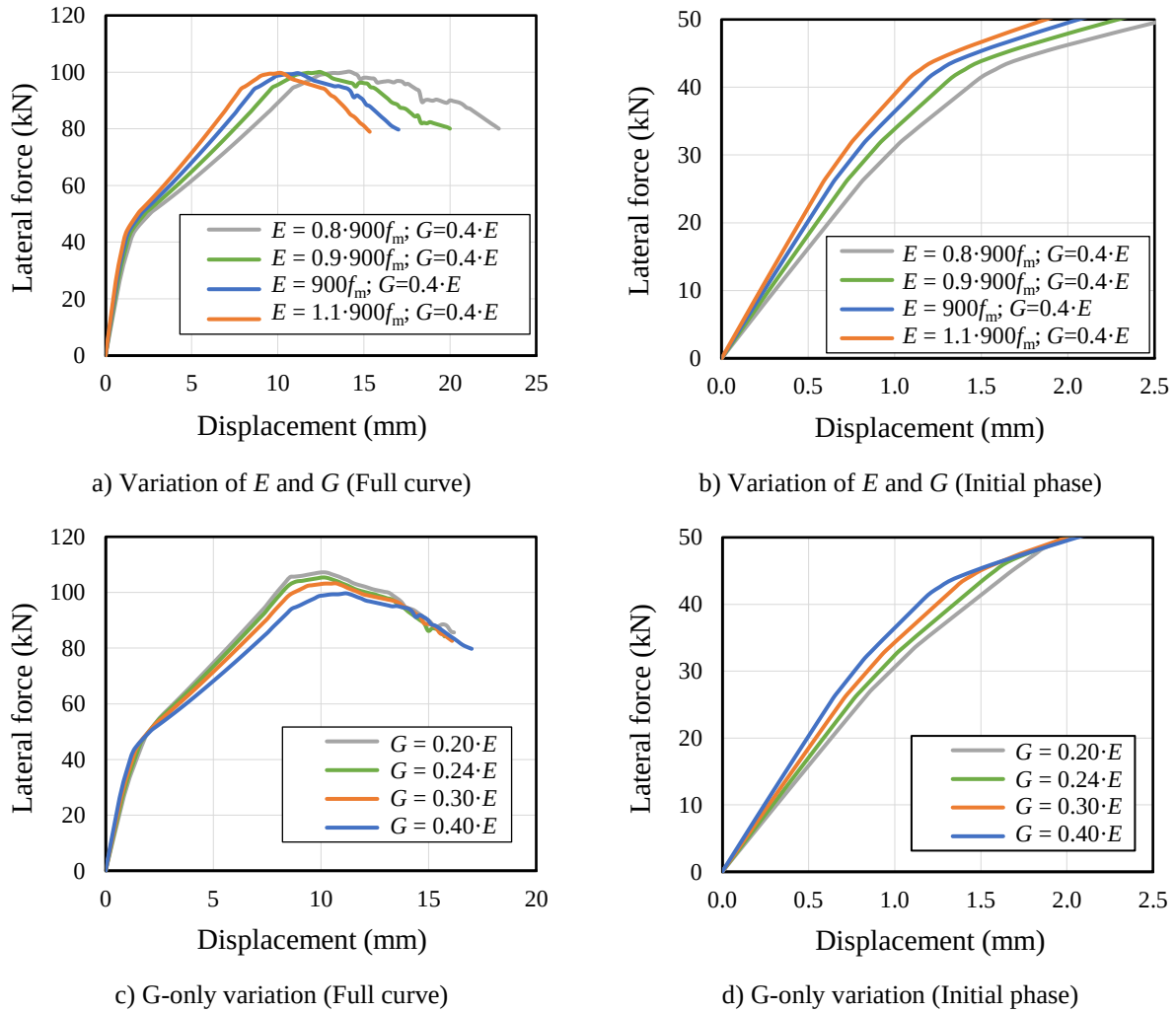
Castellazzi et al. (2022) mention that a cracked state can be represented exclusively by a reduction in the shear modulus, G , the parameter governing the stiffness of the diagonal springs. In the absence of more precise values, Eurocode 8 - Part 1 (EN 1998-1, 2004) recommends a 50% reduction in the uncracked elastic modulus for non-linear analyses. Therefore, reductions of 50%, 40%, and 25% in G of the reference model were evaluated: results are presented in Figure 61c and Figure 61d.

Complete force–displacement curves are presented in Figure 61a and Figure 61c, while zooms of the initial stage of the corresponding curves are provided in Figure 61b and Figure 61d. Modification of E and G did not change the maximum lateral force of the model, but did change the initial stiffness, the displacement corresponding to the maximum lateral force, and the ultimate displacement, as may be seen in Figure 61a and Figure 61b. In general, increasing the values of E and G increased the stiffness and decreased the displacements d_{vmax} and d_u . The percentage variation of the initial stiffness was proportional to the multiplier coefficients

applied to the values of E and G . For d_{Vmax} , percentage variations of 26.6%, 11.5%, and -9.4% were obtained by applying the 0.8, 0.9, and 1.1 multiplier coefficients to E and G , respectively.

For d_u , the percentage variation was higher, i.e., 34.1%, 17.6%, and -9.8%, respectively. This last displacement reflects changes in the post-peak behavior of the model and is in a different proportion when compared to d_{Vmax} .

Figure 61: Force–displacement curves for different values of the stiffness parameters (Walls W1/W2).



Source: Author.

From the results shown in Figure 61c and Figure 61d, one may see that although the initial stiffness was reduced by the decrease in G , higher stiffness values occurred from a force level of 50 kN. This indicates the softening stiffness in the constitutive law of the diagonal springs, calculated as a function of the kinematic hardening/softening parameter, on this model. Thus, as the initial stiffness was reduced, the softening stiffness was also reduced, that is, the influence of the kinematic hardening/softening parameter on the model response also became smaller.

3.7 CONCLUSIONS

A macro-model approach to simulate the in-plane horizontal behavior of partially grouted masonry walls with reinforcement concentrated at their ends was evaluated in this study. Numerical results were compared against those of experimental walls representative of the studied structural masonry building typology. Aspects related to the force–displacement curves, lateral displacement profiles along the wall height, and damage mechanisms were evaluated.

In general, the results indicated that the macro-model could simulate the global behavior of the analyzed experimental walls with low computational cost and good accuracy. When comparing numerical and experimental results, the main conclusions are:

- The macro-models were able to predict the maximum lateral force of the experimental walls with excellent accuracy. The most significant percentage difference, compared to the average experimental values, was -5.0%.
- The $d_{V_{max}}$ and d_{V_u} displacements were satisfactorily predicted by the macro-models, especially for the walls W1 and W2. For walls D1 and D2, conservative values were estimated by macro-models.
- The K_{sec1} evaluation (early stiffness) indicates that, in general, the macro-models tended to be less stiff than the experimental walls at the initial phase. The most significant percentage difference, compared to the average experimental values, was -20.0%. The K_{sec2} evaluation (chord stiffness from 5% to 33% V_{max}) shows good agreement between the macro-models and the average experimental values, with the most significant difference being 7.1%.
- The analysis of displacement profiles has shown that the macro-models could satisfactorily predict the evolution of horizontal displacements at the storey levels up to the peak load. In the post-peak stage, a non-linear trend of the displacement profiles was observed in both the macro-model and experimental walls due to different inter-storey drifts. However, the macro-models could not precisely match the variations observed on the experimental walls.
- The damage mechanisms simulated by the macro-models were consistent with experimental observations that indicate failure was predominantly by diagonal shear of the ungrouted panels in the three stories.

From the sensitivity analysis of the macro-model, it can be concluded that:

- The influence of the axial stress level on the wall lateral force capacity was satisfactorily captured by the macro-model when compared with the FEM results and the analytical equation

for an axial stress level higher than that considered in the experimental tests ($0.2f_m$).

- Increasing the values of the shear strength parameters (f_{v0} and μ) that control the response of the diagonal springs of the macro-elements extended the elastic regime of the global response and allowed higher maximum lateral forces to be attained.
- The introduction of the kinematic hardening/softening parameter appears to be adequate for simulating the cyclic degradation of the walls against the experimental results: however, it is emphasized that inadequately high values can result in excessive levels of ductility.
- The parameters that control the flexural mechanism of the macro-elements, specifically for the walls modeled herein, did not have significant influence on the structural response. The model behavior was mainly governed by the diagonal shear mechanism of the ungrouted masonry panels.
- Increasing the values of the stiffness parameters E and G , although without influence on the maximum lateral force of the model, increased the initial stiffness and reduced the displacements d_{Vmax} and d_u .
- A reduction in the value of only G indicated that the softening stiffness in the constitutive law of the diagonal springs was also reduced and, therefore, the influence of the kinematic hardening/softening parameter on the global behavior of the model was smaller.

Finally, despite the consistency of the macro-modeling approach for the experimental tests presented here, for its generalization, especially in buildings, it is necessary to understand possible application limitations in certain design situations. For instance, when the vertical reinforcements (grouted portions) are not continuous along all stories. Additionally, the hypothesis of assuming the grouted vertical parts as frame elements interacting with the ungrouted panels may not be completely suitable as the amount of reinforcement and, consequently, the dimensions of these parts increase or decrease. As the results here are only applicable to the cases examined (partially grouted masonry walls with the dimensions and details specified), further studies are still needed to validate this macro-modeling approach for different detailing and loading conditions before applying it to building scale models of partially grouted masonry.

4. A UNIFIED MODELING APPROACH FOR CONCRETE MASONRY SHEAR WALLS: CONCEPTION, VALIDATION AND INSIGHTS

4.1 INTRODUCTION

Masonry structures are commonly classified as either reinforced masonry (RM) or unreinforced masonry (URM), with RM further categorized as fully grouted (FGRM) or partially grouted (PGRM). RM allows to significantly improve the seismic performance of masonry structures since the reinforcement overcomes the inherent limitations of masonry, mainly associated with low tensile strength (Tomaževič; Weiss, 1994).

Contemporary FGRM walls are commonly built using hollow concrete masonry units, bed and head joint mortar, steel reinforcing bars placed in continuous vertical cells and horizontal bond beams, and grout filling the cavities inside the units. In PGRM walls, only the cells that contain reinforcement or other specific zones are grouted. It is also essential to note that even in buildings designated as URM, there is usually a minimum level of reinforcement associated with proper construction practices, aligning them with characteristics observed in PGRM.

Particularly in low- and medium-hazard seismic zones, PGRM walls may be an efficient solution for composing a seismic force-resisting system (Koutras; Shing, 2020; 2021a). However, numerical prediction and assessment of their behavior, especially in the nonlinear domain, poses challenges due to the complexity of the system.

Simulating the nonlinear behavior of URM walls presents inherent challenges due to the intrinsic heterogeneity of the wall components. Recent research has addressed this by applying simplified micro-modeling strategies to accurately simulate the behavior of different types of URM walls, including those composed of clay brick or stone units, while considering variability in material properties, through sensitivity and stochastic analyses (Abdulla; Cunningham; Gillie, 2017; Gonen et al., 2021, 2022, 2024; Howlader; Masia; Griffith, 2020). The complexity of simulating URM is amplified for PGRM shear walls due to the non-constant cross-section, the non-homogeneous stress patterns due to the unfilled hollow cores, and the presence of reinforcement that adds intricacy to an efficient simulation (Calderón et al., 2021a; Elmeligy; Aly; Galal, 2021). When all cells of a masonry wall are filled with grout, the effects of masonry

heterogeneity are mitigated, resulting in a reduced likelihood of planes of weakness between the masonry units and mortar. Therefore, the concepts employed in modeling RC structures can generally be applied to model FGRM shear walls (El-Dakhakhni; Banting; Miller, 2013; Elmeligy; Aly; Galal, 2021; Seif ElDin; Galal, 2017; Shedid; El-Dakhakhni; Drysdale, 2010a). Indeed, the behavior of PGRM walls is more complex than that of FGRM walls (Koutras; Shing, 2021a; 2021b).

Several researchers (e.g.: Arnau et al. (2015), Bolhassani et al. (2015), Calderón et al. (2021b), Koutras and Shing (2021b), and Palhares et al. (2023)) have adopted different numerical modeling strategies to simulate the behavior of PGRM walls. The diversity in detail and accuracy observed in these numerical simulation approaches reflects the inherent complexity of numerically simulating PGRM walls.

Calderón et al. (2019b; 2017) presented a detailed micro-modeling approach for analyzing three experimental PGRM walls constructed with multi-perforated clay bricks and horizontally reinforced with embedded ladder-type steel elements. The model was the result of successive improvements of previous studies (Arnau, O.; Sandoval; Murià-Vila, 2015; Sandoval; Arnau, 2017) and was developed using the DIANA software. The strategy considered the total strain crack model (TSCM) associated with the Mohr–Coulomb friction model (DIANA FEA, 2022). Subsequently, Calderón et al. (2021b) extended the validation of this modeling approach using experimental results from seven other PGRM shear walls, also constructed with multi-perforated clay bricks and featuring bed-joint reinforcement. Pettit (2023) developed a detailed micro-model using the finite element software ABAQUS to capture the in-plane shear behavior of PGRM walls. The model incorporated contact interfaces using a combination of a Mohr-Coulomb friction law and a surface-based cohesion model.

Despite providing a more realistic representation of masonry behavior at the component scale, these finite element-based micro-modeling approaches have significant time-related challenges in development and computational processing demands. To address this challenge, significant contributions have been made with simplified micro-modeling (Calderón; Milani; Sandoval, 2021d; Haach; Vasconcelos; Lourenço, 2011; Lotfi; Shing, 1994; Lourenço; Rots, 1997; Shing; Cao, 1997). These studies have demonstrated the ability of simplified micro-modeling to capture lateral response and failure modes of masonry walls with a good degree of accuracy.

Haach et al. (2011; 2009) adopted a simplified micro-modeling approach to study the behavior of concrete PGRM walls under in-plane loading. Employing the DIANA software, the TSCM was adopted to represent the nonlinear behavior of concrete masonry units, relying on a fixed stress-strain law concept (DIANA FEA, 2022). Interface elements describing the masonry joints were included and characterized by an interface cap model based on multi-surface plasticity, featuring a Coulomb friction model with a tension cut-off and an elliptical compression cap. The validation of this model was based on comparing the results of monotonic analyses with the response of in-plane cyclic tests. Subsequently, Haach et al. (2013), using the same approach, conducted a numerical analysis to investigate the behavior of URM with flanges under horizontal loading.

Bolhassani et al. (2015) developed a simplified 3D micro-model to simulate tests conducted on PGRM assemblies. To capture the nonlinear response of masonry, the concrete damage plasticity (CDP) model available in ABAQUS was used (Dassault Systèmes, 2014). The introduction of interface elements, for which a cohesive surface-based behavior was assumed, played a crucial role in representing the mortar joint discontinuity, especially in hollow masonry panels. Bolhassani et al. (2016a, 2016b) expanded this study to numerically simulate the in-plane behavior of PGRM walls.

More recently, Calderón et al. (2021d) proposed a 2D simplified micro-modeling approach to simulate the in-plane behavior of PGRM shear walls. The tensile fracture of masonry and the detachment of blocks from mortar were considered through considering an interface failure in tension. The compression failure of GM and UGM was simulated using the TSCM implemented in DIANA software (DIANA FEA, 2022). After performing monotonic analysis of the walls, the authors compared the numerical results with the backbone responses from cyclic tests.

Elmeligy et al. (2021) and Medeiros et al. (2022a) evaluated a simplified micro-model implemented in VecTor 2 software (VecTor Analysis Group, 2019) to simulate the cyclic behavior of PGRM walls. It should be noted that in the VecTor 2 model, the joint's mortar interfaces are not explicitly modeled; instead, the software automatically attributes the joint's effects based on input properties like cohesion and angle of internal friction. The results demonstrated the effectiveness of this approach in simulating the cyclic experimental behavior of the investigated walls. This software is limited to 2D analyses; therefore, simulation of 3D effects prevalent in building systems, such as those in flanged walls and slab connections, is currently not possible.

Macro-modeling approaches have been used satisfactorily to simulate the in-plane behavior of URM (Berto et al., 2004; Calìo; Marletta; Pantò, 2012; Karapitta; Mouzakis; Carydis, 2011; Lourenço, 2002; Morandi et al., 2022; Occhipinti et al., 2022; Parisse et al., 2022) and FGRM (Abdellatif et al., 2019; Aly; Galal, 2019, 2020; Hosseinzadeh; Galal, 2020) shear walls. However, due to the complexities associated with PGRM walls, most studies to simulate this typology have focused on refined approaches. Even so, Palhares et al. (2023) introduced a macro-modeling approach, which is particularly intended to simulate the behavior of PGRM walls where the vertical reinforcement is concentrated at the wall ends and remains constant along the wall height. In this configuration, PGRM walls exhibit similar behavior to confined masonry walls or reinforced concrete (RC) infilled frames (Bolhassani et al., 2016a; Calderón et al., 2020; Dhanasekar; Haider, 2011; Koutras; Shing, 2021b; Minaie et al., 2010; Shing; Cao, 1997).

Although several studies have been developed to simulate masonry shear walls, each is validated using a limited number of specimens tested in the same laboratory conditions. Besides, those studies cover only specific scenarios of detailing, so that a unified and more comprehensive modeling approach is needed. Validation studies of numerical models against experimental tests are still needed to obtain more insights about the behavior of diverse configurations of masonry structures and their well-known variability. The most challenging configurations involve PGRM walls, given the complex interaction between the grouted/reinforced masonry and the ungrouted regions. These two regions exhibit differences in stiffness, and partial grouting introduces non-uniformity in the cross-section. To address these challenges, this research aims to conceive and validate a unified modeling approach integrating macro-modeling concepts for grouted masonry (GM) regions and simplified micro-modeling for ungrouted masonry (UGM) regions using an experimental dataset covering different masonry shear wall typologies (URM, FGRM, and PGRM) and considering key variables, such as wall aspect ratio, presence of openings, axial stress level, and loading protocols. The validation process consists of two levels: the first concentrated on URM and FGRM walls, with a primary emphasis on analyzing lateral force–displacement curves; the second level entails validating PGRM walls, involving an assessment of lateral force–displacement curves, damage patterns, and reinforcement strains.

4.2 MODELING APPROACH

Different numerical methods can be used to simulate the response of composite materials, such as the Finite Difference Method (Wang; Gu; Liu, 2020) and the Bezier Multi-Step Method (Kabir; Aghdam, 2021). For specific problems, such as stress analysis in layered media, these numerical methods can be very robust and stable, but are difficult to apply to the scale of civil engineering structures, particularly for nonlinear analysis, since most software packages available for this purpose are based on the Finite Element (FE) Method.

In FE modeling of masonry, cracking can be modeled either with a smeared-crack or discrete-crack approaches. The smeared-crack method treats cracking as a continuum within solid or shell elements and suits quasi-brittle materials with densely distributed cracks, such as large-scale RC structures (Lourenço; Gaetani, 2022). However, this method faces limitations in accurately representing brittle shear behavior, characterized by localized diagonal cracks, due to the stress-locking phenomenon (Rots; Blaauwendraad, 1989). Although computationally more intensive, the discrete-crack approach offers a more sensible representation of cracks, explicitly utilizing interface elements to model the displacement discontinuity across a crack.

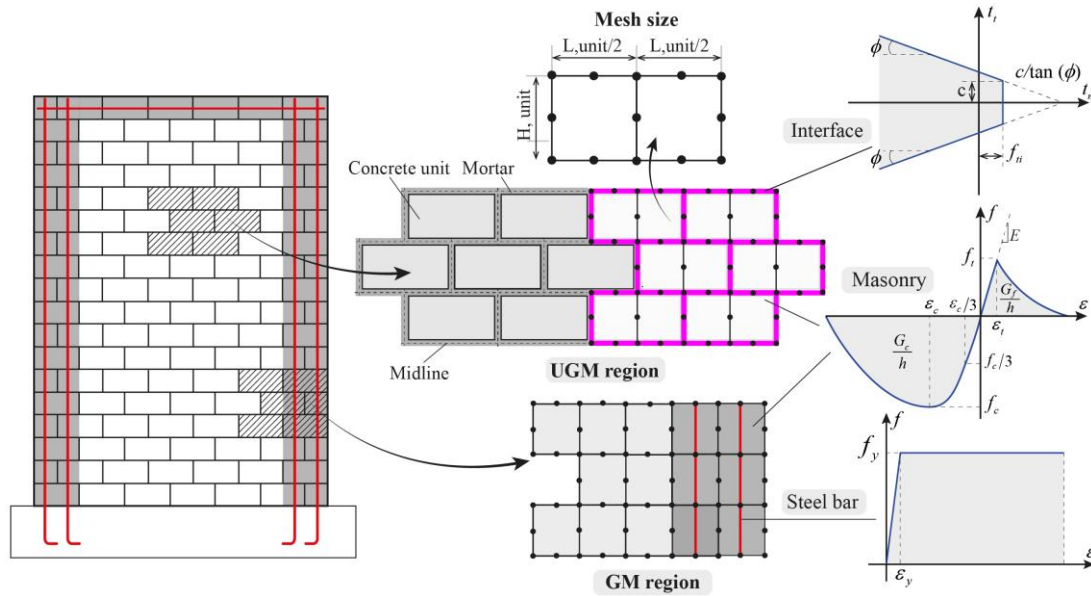
The smeared approach is suitable for modeling FGRM walls due to their similarity to concrete structures. However, for hollow masonry, the smeared method may be less appropriate (Koutras; Shing, 2021b). Therefore, coupling the smeared and discrete approaches appears to be a practical and valuable means of modeling the flexural and shear behavior of PGRM walls.

The modeling approach evaluated herein, conceived using the DIANA[®] software (DIANA FEA, 2022), considers a smeared-crack approach, i.e., the total strain crack model (TSCM), to simulate the compressive behavior, as well as to capture diffuse cracking throughout both grouted (GM) and ungrouted masonry (UGM) regions, and a discrete-crack approach to model mortar bed and head joints in UGM regions as interfaces. FG regions were modeled without considering the influence of the mortar joints between elements, while embedded bond truss elements simulate the reinforcement bars.

The smeared and discrete modeling states are defined independently a priori, i.e., the continuum and interface elements, respectively, are modeled explicitly, but within the simulation, there is connectivity and interaction between the two types of elements. These are connected by adjacent nodes, ensuring interaction and transition without any compatibility issues. The discrete state is due to the opening of interfaces. A comprehensive model

discretization is illustrated in Figure 62. A brief overview of the main characteristics of the FE types employed in the analysis is provided in Sections 2.1 to 2.4.

Figure 62: Scheme of the FE modeling approach adopted.



Source: Author.

4.2.1 UngROUTED and grouted masonry regions

GM and UGM regions were simulated employing the TSCM. This model has been widely used for simulating the failure of brittle materials like masonry and concrete (e.g., Calderón et al. (2021c; 2019b; 2021d), Lourenço et al. (2011), Ponte et al. (2019), Sanabria et al. (2018), Schreppers et al. (2017), and Zizi et al. (2022)). The TSCM is derived from the principles of the modified compression field theory for concrete, initially introduced by Vecchio and Collins (1986), and is based on the direct implementation of experimental observations reported by Selby and Vecchio (1993).

In the smeared cracking concept, crack openings are smeared over the element area and represented as crack strains. As a result, the element size influences the dissipated fracture energy, making the results mesh-sensitive. To address this issue, a length scale parameter known as crack bandwidth, h , is introduced into the softening portion of the stress-strain curves. In this study, the crack bandwidth was calculated as $h = \sqrt{A}$, where A is the total area of the element, according to the approach proposed by Rots (1988). This approach is noteworthy since energy-normalized curves have been reported to mitigate mesh-size sensitivity (Calderón; Arnau; Sandoval, 2019b; Lotfi; Shing, 1991).

The TSCM comprehensively describes tensile and compressive behaviors through one-dimensional stress-strain relationships; therefore, the behavior of masonry is characterized by nonlinear stress-strain relations for both cracking (tension) and crushing (compression). A parabolic softening curve was chosen for the compression regime, while the Hordijk et al. (1991) model was adopted for the tension regime. These uniaxial stress-strain curves are presented in Figure 62. When the tensile stress reaches the specified tensile strength, f_t , crack is initiated, and the transition from the uncracked to the cracked state occurs. In terms of compressive behavior, the model can be improved by incorporating factors such as an increase in compressive strength due to lateral confinement or a reduction in compressive strength due to lateral cracking (DIANA FEA, 2022). The lateral confinement effect was considered in this study according to the Selby and Vecchio model (1993), which is implemented in DIANA (DIANA FEA, 2022) for the TSCM.

The TSCM can follow the concept of fixed or coaxial stress-strain. In the fixed crack model, stress-strain relationships are established in a conventional coordinate system that remains fixed after cracking occurs. In the coaxial approach, often referred to as the rotating crack model, stress-strain relationships are defined in the principal directions of the strain vector; the crack orientation is continuously updated throughout the analysis (DIANA FEA, 2022).

Lourenço and Gaetani (2022) recommend the use of the rotating crack model particularly for unreinforced masonry and concrete structures, enabling correction of the initial mispredictions of the crack direction. In this study, the rotating crack model was adopted, allowing shear softening to occur implicitly as a result of the co-axiality of principal stress and strain, thereby eliminating the need for additional parameters (DIANA FEA, 2022; Lourenço; Gaetani, 2022; Rots; Blaauwendraad, 1989).

Eight-node quadrilateral isoparametric plane stress FEs with quadratic interpolation and Gauss integration (CQ16M element by DIANA software (DIANA FEA, 2022)) were employed to discretize both UGM and GM regions, as well as the RC beams. Regarding the choice of mesh size, previous researchers (Lotfi; Shing, 1991; Maleki, M., El-Damatty, A. A., Hamid, A. A., Drysdale, 2005; Shing; Cao, 1997) conducted mesh sensitivity analyses and concluded that using a mesh size that divides each block into two elements provides acceptable results with reasonable computational effort. Therefore, the mesh was generated with elements every half block, as outlined in Figure 62. Given the considerations above, the element aspect ratio is ideally set to 1.0. This methodology has been employed by several researchers (Elmeligy; Aly;

Galal, 2021; Haach; Vasconcelos; Lourenço, 2011; Lotfi; Shing, 1991; Maleki, M., El-Damatty, A. A., Hamid, A. A., Drysdale, 2005; Medeiros et al., 2022a).

The thickness of UGM regions was considered as the sum of the face shells of the hollow blocks, under the assumption that the web of the hollow blocks does not significantly contribute to the in-plane behavior of the walls, as the study exclusively focuses on face shell bedded masonry. For GM regions, the total thickness of the block was assumed. This approach aligns with those of Bolhassani et al (2016a, 2015; 2016b), Medeiros et al. (2022a), and Palhares et al. (2023).

4.2.2 Interface elements

The influence of horizontal and vertical mortar joints was taken into account by incorporating interface elements in UGM regions, as illustrated in Figure 61. Six-node interface FEs labeled as CL12I were employed. These elements are based on quadratic interpolation and a 3-point Newton-Cotes integration scheme (DIANA FEA, 2022).

The basic interface variables are the nodal displacements, from which the relative displacements and the normal and shear tractions are derived to characterize the interface behavior. According to Lourenço (1996), unit-mortar interfaces can fail under tension or shear; therefore, a Coulomb-based constitutive model (Figure 62) has been adopted to consider both failure modes, establishing a nonlinear relationship between the tractions and the relative displacements across the interface. The friction was expanded by considering a brittle gap criterion. In this case, it is assumed that a gap arises if the tensile traction normal to the interface, t_n , exceeds a specific threshold. After gap formation, t_n is reduced to zero immediately (brittle cracking) (DIANA FEA, 2022). This model has been widely used for modeling masonry (e.g., Calderón et al (2019b, 2021c, 2021d), Rahman and Ueda (2014), and Sandoval and Arnau (2017).

The material models governing interface elements establish either a linear or nonlinear relationship between tractions, i.e., normal traction t_n and shear traction t_t , and relative displacements across the interface, i.e., normal relative displacement Δu_n and shear relative displacement Δu_t . The normal, k_n , and tangential, k_t , stiffness components of the interfaces were determined as functions of the UGM elastic modulus, E , the Poisson's ratio, ν , and the average mortar joint height, h_{int} , following the Equations 7 and 8, respectively. These formulations have been adopted in other studies (Arnau, O.; Sandoval; Murià-Vila, 2015; Calderón; Arnau;

Sandoval, 2019b; Calderón; Milani; Sandoval, 2021d; Calderón; Sandoval; Arnau, 2017; Sandoval; Arnau, 2017).

$$k_n = \frac{E}{h_{int}} \quad \text{Eq. 7}$$

$$k_t = \frac{E}{2 \cdot (1 + \mu) \cdot h_{int}} \quad \text{Eq. 8}$$

The interface properties to assume, cohesion, c , and friction angle, ϕ , can be determined experimentally through direct shear tests (pre-compressed triplets). Furthermore, the tensile bond strength of interfaces, f_{ti} , can be estimated through direct tensile tests conducted on 2-unit stack-bond prisms. In cases where direct testing is not feasible, reference values from the literature for similar materials can be employed. In this study, as in other similar investigations (e.g.: Calderón et al. (2019b, 2021c, 2021d), Lourenço and Gaetani (2022)), a zero dilatancy angle was assumed. This assumption is equivalent to not considering the influence of rough surfaces in interface slipping and avoiding non-conservative predictions of the shear capacity when interfaces experience large relative shear displacements (van Zijl, 2004).

It is important to highlight that vertical joints may exhibit weaker strength parameters than horizontal joints, influenced by factors such as workmanship quality and execution technique (Lourenço; Gaetani, 2022). For simplicity, as adopted by other authors (e.g., Bolhassani et al. (2016a), Calderón et al. (2021c, 2021d); Haach et al. (2011), Medeiros et al. (2022a), the same parameters were utilized here for both horizontal and vertical joints.

4.2.3 Steel reinforcement

Vertical and horizontal steel reinforcement was modeled using 2-noded isoparametric truss elements, employing linear interpolation functions (DIANA FEA, 2022). The strains in the reinforcement are computed based on the displacement field of the continuum elements, assuming a perfect bond between the reinforcement and the surrounding GM by connecting them node by node, as illustrated in Figure 62. This consideration is consistent with previous experimental and numerical studies (Bolhassani et al., 2016; Bolhassani; Hamid; Moon, 2016; Calderón; Arnau; Sandoval, 2019b; Calderón; Milani; Sandoval, 2021d; Haach; Vasconcelos; Lourenço, 2011).

4.2.4 Numerical aspects of nonlinear analyses

When dealing with nonlinear problems, choosing suitable methods to solve the system of equations representing the model is as crucial as the assumptions and simplifications made during the modeling phase (Lourenço; Gaetani, 2022). This study follows the recommendations of Calderón et al. (2017, 2019b, 2021c, 2021d) and Sandoval and Arnau (2017) to define the numerical strategies for nonlinear analysis. The quasi-Newton (or secant) iterative method with the Broyden-inverse update method (Broyden, 1970), using a sparse Cholesky-based direct solver, was adopted. The convergence criterion was based on an energy-based norm, which has been shown to be adequate in previous research studies at the wall scale (Haach; Ramalho; Corrêa, 2013; Haach; Vasconcelos; Lourenço, 2011). A convergence tolerance of 10^{-4} was set as a compromise to obtain a relatively accurate solution. In addition, the line search algorithm technique was employed to enhance both the convergence rate and the likelihood of convergence. These numerical techniques are already implemented in the DIANA software; more details are provided in DIANA FEA (2022).

A quasi-static implicit nonlinear analysis was performed for all wall models in this study, considering physical and geometrical nonlinear (Total Lagrange) behavior. In the initial loading stage, the self-weight and a distributed vertical load at the wall top were applied, followed by a series of prescribed horizontal displacements (load steps), near the wall top, each with a determined size. For each load step, the top horizontal displacement and the base reaction force were tabulated to generate the force–displacement diagram, which is subsequently compared with the measured force–displacement curves from experiments.

A computational efficiency analysis (in terms of processing time) of the models evaluated in this study was not directly conducted, as no significant computational cost issues were encountered for the cases evaluated here. All analyses presented were processed on a regular computer with an Intel Core i7-7500U-2.70GHz CPU, dual-core processor, with 8GB RAM. Processing time depends on the application and may become relevant in more complex analyses, such as at the building scale.

4.2.5 Comparison with similar approaches

As previously demonstrated, significant contributions have been made with simplified micro-modeling to study the in-plane behavior of masonry (Abdulla; Cunningham; Gillie, 2017; Bolhassani et al., 2016; Bolhassani; Hamid; Moon, 2016; Calderón; Milani; Sandoval, 2021d; Haach; Vasconcelos; Lourenço, 2011; Howlader; Masia; Griffith, 2020). The unified

modeling strategy proposed in this study takes advantage of these previous studies by combining their strategies and simplifying some features. The main differences between the previous approaches in the literature and the one presented here are listed in Table 5.

Table 5: Comparison of the presented model with similar approaches.

Characteristics	Presented model	Haach et al. (2011)	Calderón et al. (2011d)	Howlader et al. (2020)	Bolhassani et al. (2016)	Abdulla et al. (2017)
Software	DIANA	DIANA	DIANA	DIANA	ABAQUS	ABAQUS
Dimension	2D	2D	2D	2D	3D	3D
Interface presence	Mortar joints	UGM	UGM and GM	UGM and GM	UGM	UGM
	Unit cracks	-	vertical crack at the middle	vertical crack at the middle and diagonal cracks	vertical crack each quarter	-
Masonry crack	Smeared	Smeared and discrete	Discrete	Discrete	Smeared	Smeared
Element type	Masonry¹	CQ16M	CQ16M	CT12M and CQ16M ¹	Q8MEM	C3D8R
	Interfaces²	CL12I ⁵	CL12I ⁵	CL12I ⁵	L8IF ⁵	Not reported
Mesh size	Elements by block	2	2	48	16	2
	UGM	face shells	block	equiv. net width	solid clay bricks	face shell
Walls thickness	GM	block	Block	equiv. net width	-	block
	Masonry model³	TSCM (RCM)	TSCM (FCM)	TSCM (RCM)	Linear elastic	CDP
Masonry stress-strain curves	Compression	Parabolic	Parabolic	Parabolic	Linear	Prism tests
	Tension	Hordijk	Exponential	Linear	Linear	Diagonal tension tests
Interface material model	Mortar joints	Coulomb friction	Combined crack-shear-crush	Coulomb friction	Combined crack-shear-crush	Cohesive surface
	Unit cracks	-	Discrete craking	Discrete craking	Discrete craking	-
Iterative method	Secant	Newton-Raphson	Secant	Not reported	Not reported	Newton Raphson
N° validated walls	UGM	9	2	-	8	-
	FGRM	2	-	-	-	-
	PGRM	20	4	2	-	6

1. CQ16M: eight-node quadrilateral for plane stress analysis; CT12M: six-node triangular element for plane stress analysis; Q8MEM: four-node rectangular element for plane stress analysis; and C3D8R: eight-node hexahedral element for 3D stress analysis.

2. Interface elements CL12I and L8IF are characterized by six-node and four-node configurations, respectively

3. TSCM includes a rotating crack model (RCM) and fixed crack model (FCM), while CDM represents concrete damage plasticity (Dassault Systèmes, 2014), and XFEM stands for Extended Finite Element Method (Abdulla; Cunningham; Gillie, 2017)

Source: Author.

From Table 5, it is noted that the well-known software packages DIANA (DIANA FEA, 2022) and ABAQUS (Dassault Systèmes, 2014) have been adopted to develop numerical models of masonry walls, all including features already implemented in these software packages to take advantage of their proven efficiency and avoid new routines without limited validation, and using numerical procedures already implemented in these packages.

Combining simplified micro-modeling for the UGM regions and macro-modeling for the GM regions presented here is based on the approach adopted by Bolhassani et al. (2016a, 2016b), which assumes the influence of mortar joints only in the UGM regions. The difference in the current study is that a 2D approach is adopted instead of the 3D modeling by Bolhassani

et al. (2016a, 2016b). Besides, the model here is developed using DIANA software instead of ABAQUS; consequently, different aspects are observed, such as the type of FEs, masonry model, and iterative method.

Regarding the model by Haach et al. (2011), although the same software package was employed, the approach presented in this current study is more straightforward since only smeared cracks are considered in the GM regions; the crack pattern within the units is not explicitly modeled. Thus, discrete cracks in the middle of the blocks, assumed by Haach et al. (2011), are not considered. Furthermore, the Coulomb friction nonlinear interface model adopted here requires significantly fewer input parameters than the combined crack-shear-crush interface adopted by Haach et al. (2011), which increases uncertainties when these properties are unknown. The TSCM model adopted here differs from that of Haach et al. (2011): they used the fixed crack model concept whereas the rotating crack model is adopted here. Another difference is that Haach et al. (2011) consider the properties based on the gross section area and then adopts the block thickness even in UGM regions with hollow concrete blocks. In contrast, this study considers the face shell thickness for UGM regions, and the material properties are based on the net cross-sectional area. Regarding the iterative method, Haach et al. (2011) adopted Newton-Raphson, while in this study the quasi-Newton (secant) is employed.

The 2D simplified approach adopted by Calderón et al. (2021d), also developed in DIANA, shares many characteristics with the present study. However, the model presented by Calderón et al. (2021d), although recognized by the authors as resulting from simplified micro-modeling, is undoubtedly more complex than the one presented in this study. Such complexity is due to considering different interfaces: mortar joints (UGM and GM regions) and discrete cracks (vertical cracks in the middle and diagonal cracks in the blocks). The mesh, composed of 6-node triangular elements (CT12M) and 8-node quadrilateral elements (CQ16M), requires a larger number of FEs compared to the model presented here, which, as in Haach et al. (2011) and Bolhassani et al. (2016a, 2016b) models, discretizes the mesh as two finite elements per block. The reinforcement model of all approaches assumed that the reinforcement was perfectly bonded to the surrounding grout.

Other models, although validated only for UGM walls with solid clay bricks (not covered here), also differ, in part, from the one presented here, as reported in Table 5. Although also adopting DIANA software, Howlader et al. (2020) use different FEs for the continuous elements and interfaces, and all nonlinearity is lumped into interface elements. Furthermore, a more significant number of elements per unit is required. Abdulla et al. (2017), using ABAQUS,

present a 3D approach requiring a significantly higher number of finite elements (42 elements per unit).

The number of validation cases in the present study is significantly larger than in other studies and certainly contributes to the advancement of new models and applications for masonry structures. In all studies referenced in Table 5, their models were also analyzed and validated through monotonic lateral loading.

4.3 FIRST-LEVEL MODEL VALIDATION

As a first step towards validating the modeling approach, URM and FGRM walls were evaluated. The outcomes were helpful for subsequent analyses of PGRM walls since the last ones combine the characteristics of URM and FGRM walls.

The validation of the numerical model is restricted to walls constructed in running bond with concrete masonry units and mortar on the face shells of the blocks. The walls were typically tested with cantilever-type support conditions under lateral in-plane loading. Overviews of the experimental tests are provided; further details are available in the respective references.

The names assigned to the wall specimens have been adjusted from their original designations in the experimental studies to represent the details of each wall more accurately. A nomenclature based on the main variables was employed as follows: Wall type (URM, PGRM, and FGRM) – Aspect ratio (height to length ratio) – Opening (NO: no opening, DO: door opening, and WO: window opening) – Axial stress level in MPa. For example, PGRM-1.0-NO- 3.8 denotes a PGRM wall, with aspect ratio 1.0, no openings, and under axial stress of 3.8 MPa (considering the net cross-sectional area).

The accuracy of the numerical models was evaluated by comparing the monotonic responses with the corresponding experimental results, including the monotonic curves or the backbone curves obtained from the hysteretic responses. It is worth highlighting that the TSCM implemented in DIANA relies on secant unloading and reloading. As such, it might not accurately simulate the cyclic response of masonry structures, as it leads to a significant underestimation of energy dissipation under cyclic loading conditions (Fusco et al., 2022; Lourenço; Gaetani, 2022; Schreppers et al., 2017; Sousamli; Messali; Rots, 2022). Nevertheless, previous studies (e.g., Calderón et al. (2019b, 2021c), Di Lallo et al. (2022, 2023), Lourenço et al. (2007), Magenes and Calvi (1997), Maleki et al. (2005), Palhares et al. (2023), Parris et al. (2022), Pettit (2023), Quinteros et al. (2012), Rahim et al. (2023), Shing and Cao

(1997), and Silva et al. (2018)) have shown that numerical monotonic responses can offer a reliable approximation of the cyclic response of masonry shear walls regarding the initial stiffness, the peak load and the displacement response. The representativeness of the monotonic loading in estimating the envelope of the hysteretic response of masonry walls was particularly evidenced in the experimental study by Magenes and Calvi (1997).

Key response measures were considered to evaluate the model accuracy: the maximum lateral force, $V_{\max}$, the displacement at the maximum lateral force, $d_{V_{\max}}$, the ultimate displacement, d_{Vu} , the secant stiffness, K_{sec} , and the dissipated energy, U . The d_{Vu} was defined in accordance with a lateral resistance drop of 20% in $V_{\max}$. The K_{sec} was calculated over a range of 5% to 33% of $V_{\max}$, following a common procedure in design standards (e.g., TMS 402/602 (2022) and CSA S304 (2014)) for calculating the Young's modulus for design.

These key measures are helpful to know the suitability of the models in estimating the parameters required for seismic design. For instance, in linear seismic analyses, the applied lateral forces are derived from the structure's natural period of vibration, which is directly influenced by its elastic stiffness. Therefore, computational models must demonstrate acceptable accuracy in predicting elastic stiffness and ensure that the seismic design is carried out effectively. Furthermore, the energy (area under the curve) is central to seismic analysis, specifically in deriving seismic parameters like the displacement ductility, μ_d , and the force modification factor, R . These seismic parameters are crucial when considering the nonlinear behavior of the structure implicitly in linear analyses. This is a major challenge in the linear seismic analysis (Lagomarsino; Marino; Cattari, 2020; Morandi et al., 2022).

4.3.1 URM walls

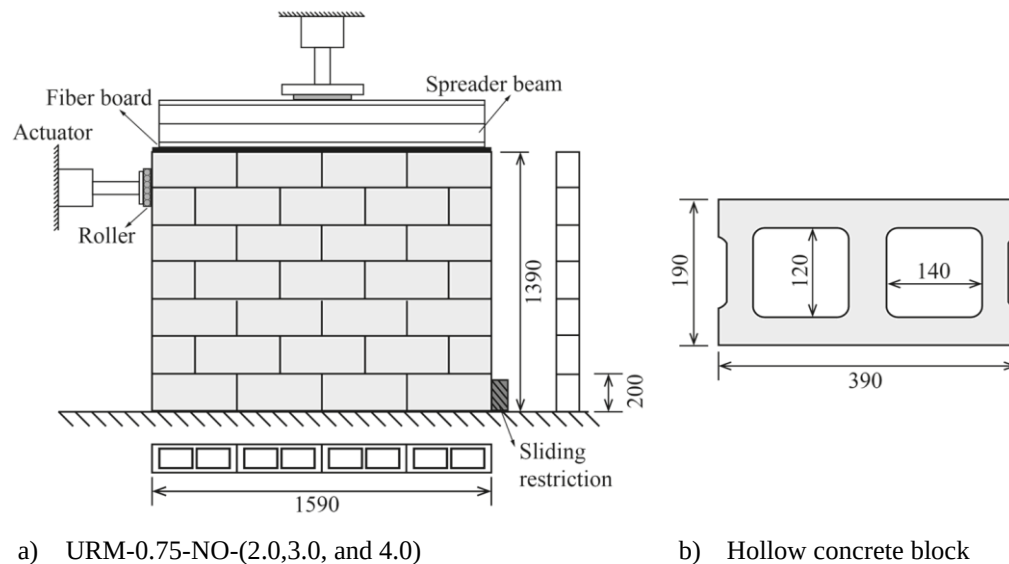
4.3.1.1 Description of the experimental walls

Nine full-scale URM walls, previously tested under in-plane bi-axial monotonic loading conditions by Oan (2013), serve as the experimental dataset for this study. The main variable in this selected dataset was the axial compressive stress. The walls were categorized based on three levels of axial stress (2 MPa, 3 MPa, and 4 MPa) determined via the net area of the wall. Three specimens were tested in each category for representativeness and to check for potential deviations.

The geometry of the walls is schematized in Figure 63, with each wall measuring 1.6 m in length (equivalent to four blocks) and 1.4 m in height (spanning seven courses). Horizontal loading was applied to the upper two courses of the walls (along half of each course height),

resulting in an aspect ratio of 0.75. Unlike conventional test setups, the walls were not built on concrete base. Then, to prevent the wall from sliding at the bottom due to the lateral load at the top, a load actuator was strategically positioned at the wall base, as represented in Figure 62. The construction of all walls employed mortar face-shell bedding, utilizing standard hollow concrete blocks ($390 \times 190 \times 190$ mm) and Type S mortar (nominal 1:0.5:4.5 (PC:lime:sand)).

Figure 63: Experimental arrangement of URM walls; dimensions in mm.



Source: Adapted from Oan (2013).

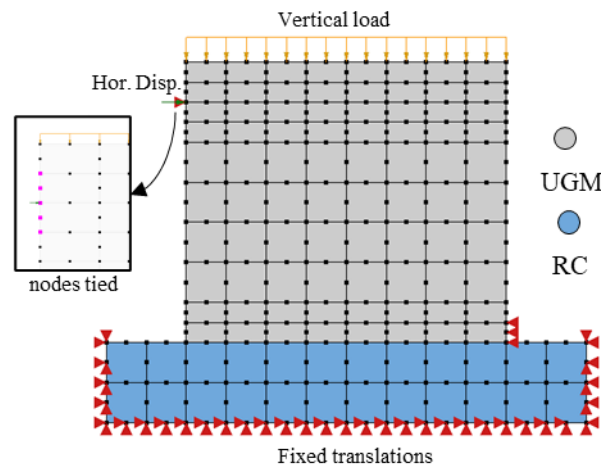
4. 3.1.2 Description of the numerical model

The configurations of the numerical models of the experimental walls above are depicted in Figure 64. The UGM regions have a thickness of 70 mm, equal to the sum of the thicknesses of the two face shells of the hollow concrete blocks. Each FE has a size of 200×200 mm except for the regions close to the top and the base, where it was necessary to refine the mesh to accommodate the model's boundary conditions.

Regarding the boundary conditions, the nodes at the foundation of the model were constrained horizontally and vertically. To simulate the experimental setup, the nodes at the bottom side of the wall opposite to the direction of the load application were constrained horizontally. This constraint was set to prevent possible sliding of the wall on the base.

In the initial phase, the walls were subjected to a vertical load corresponding to their axial stress, followed by a horizontal displacement at the top in constant increments of 0.025mm. The self-weight was considered based on the mass density of 2400 kg/m^3 attributed to the masonry, and the external axial load was applied as a constant distributed load on the wall top.

Figure 64: Numerical model of URM-0.75-NO-(2.0,3.0, and 4.0) walls.



Source: Author.

The material properties of the UGM and the interfaces used to simulate these walls are provided in Table 6. The compressive strength, f_m , of UGM was defined based on the average results of prism tests conducted by Oan (2013). The Young's modulus, E , tensile strength, f_t , and fracture energy in compression, G_c , and tension, G_f , were calculated as functions of the compressive strength. The Young's modulus, E , was estimated as $900 \cdot f_m$, according to the recommendation of TMS 402-602 (2022). A Poisson's ratio, ν , of 0.23 was adopted based on Oan's experimental tests on hollow concrete blocks. The tensile strength, f_t , was estimated according to Equation 9, proposed by Wong et al. (2013), as previously used in the numerical models by Medeiros et al. (2022a) and Rahim et al. (2023). The tensile fracture energy, G_f , was calculated using Equation 10, as proposed by the FIB Model Code (2013) for concrete-like materials. The compressive fracture energy, G_c , was computed according to Equation 11, as suggested by Magenes et al. (2008) and adopted by Ademovic and Hrasnica (2015), Filippou et al. (2019, 2020), and Schreppers et al. (2017). The RC base was modeled as a linear elastic material with a Young's modulus of 30 GPa and a Poisson's ratio of 0.15.

$$f_t = 0.33 \cdot \sqrt{f_m} \quad \text{Eq. 9}$$

$$G_f = 73 \cdot f_m^{0.18} \quad \text{Eq. 10}$$

$$G_c = 15 + 0.43 \cdot f_m - 0.0036 \cdot f_m^2 \quad \text{Eq. 11}$$

where f_m is the compressive strength of masonry in MPa.

Table 6: Material properties adopted for URM walls.

Element	Young's modulus (E)	Compressive strength (f_m)	Compressive fracture energy (G_c)	Tensile strength (f_t)	Tensile fracture energy (G_f)	Normal stiffness (k_n)	Shear stiffness (k_t)	Cohesion (c)	Friction angle (ϕ)	Tensile strength (f_{ti})
	N/mm ²	N/mm ²	N/mm	N/mm ²	N/mm	N/mm ²	N/mm ²	N/mm ²	deg.	N/mm ²
URM-0.75-NO-(2.0, 3.0, and 4.0) (Oan, 2013)										
UGM	14580	16.2*	21.02	1.33	0.12			-		
Interfaces			-			1458	593	0.14*	44.7*	0.10

*Based on experimental data

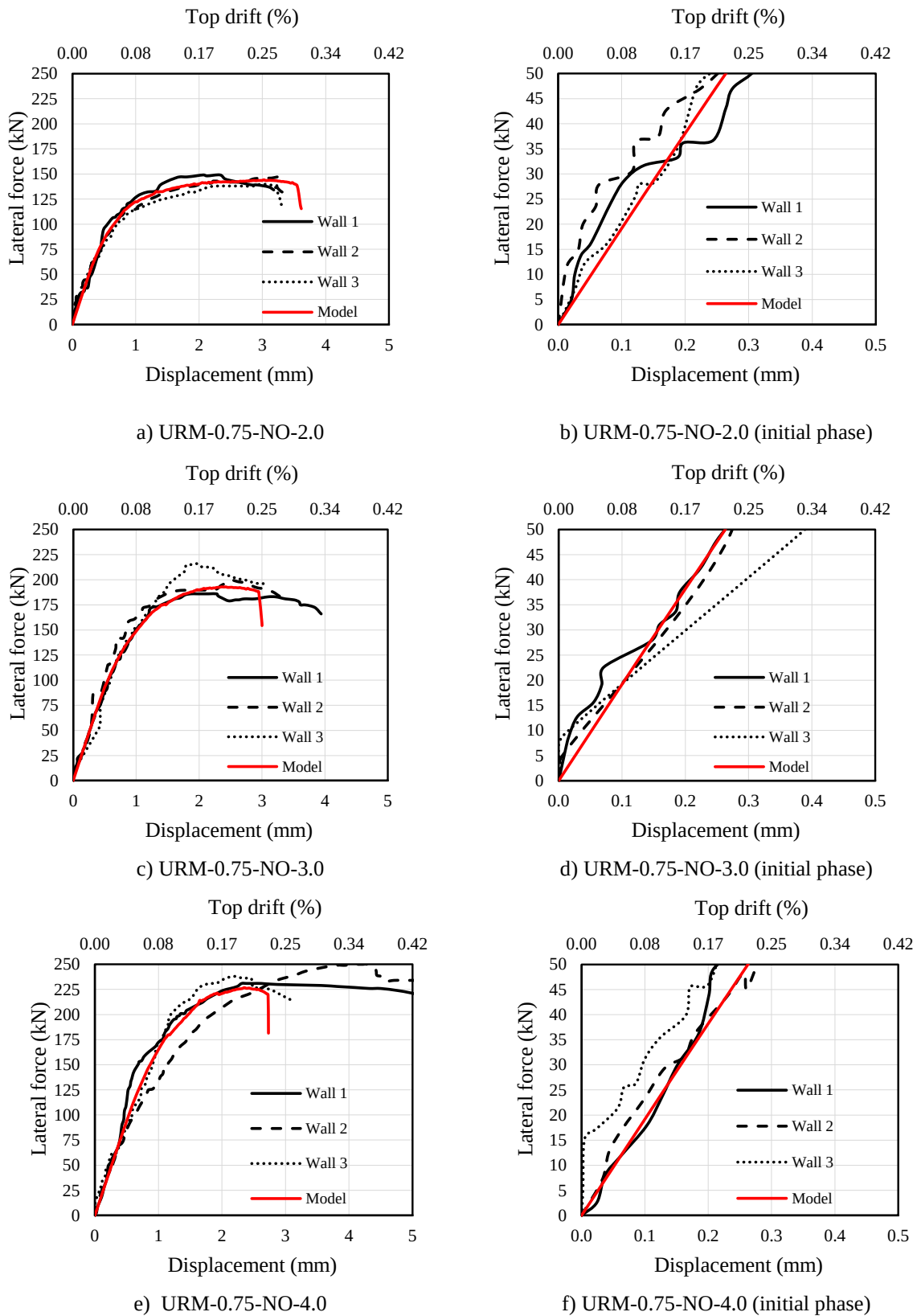
Source: Author.

For the interfaces, as mortar joint shear tests were not carried out by Oan (2013), the cohesion, c , and the friction angle, ϕ , were defined based on the shear triplet test results by Ba Rahim (2023) and Pettit (2023), in which blocks and mortar similar to those used by Oan (2013) were employed. A tensile strength of 0.10 MPa was assumed for the interfaces, f_{ti} , according to the recommendations of Lourenço and Gaetani (2022). The normal and shear stiffness components were calculated according to Equations 7 and 8.

4. 3.1.3 Numerical vs. experimental comparison

The numerical and experimental results were initially compared through force-displacement curves, as presented in Figure 65. Note that the complete responses are presented on the left, while a zoomed-in view of the initial response phase is provided on the right. The top drift values, i.e., displacement/height ratio, where the height is the distance from the base of the wall and the load actuator axis, are also indicated on the upper horizontal axis.

Figure 65: Force–displacement curves of URM walls.



Source: Author.

Visual analysis of Figure 65 highlights qualitatively the capacity of the model to accurately predict the walls' response in terms of initial stiffness, maximum lateral force, and energy dissipation. The models correctly captured the initial degradation in lateral stiffness observed in the specimens at the beginning of the tests. Subsequently, the models replicated the gradual decline in lateral stiffness until reaching the maximum lateral force, reflecting the behavior observed in the experimental tests.

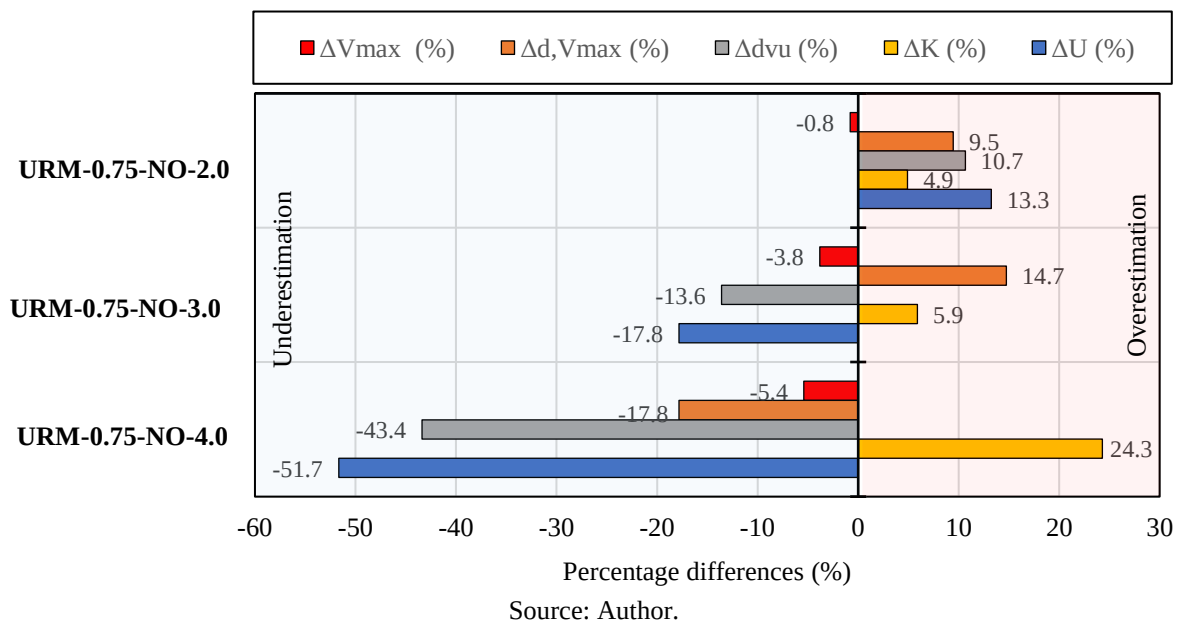
The values of key response measures from the force–displacement curves are reported in Table 7. The percentage differences between the results of the models and the average test responses in terms of the identified key response measures are shown in Figure 66. Note that the percentage difference graph includes shaded backgrounds to indicate regions of underestimation (light blue) and overestimation (light red), with corresponding labels placed on the left and right sides, respectively.

Table 7: Summary of key response measures in the force–displacement curves for URM walls.

Response	V_{max} (kN)	d_{Vmax} (mm)	d_{Vu} (mm)	K_{sec} (kN/mm)	U (kJ)
URM-0.75-NO-2.0					
Wall 1	149.0	2.1	3.2	154.0	387.2
Wall 2	147.0	3.2	3.3	179.9	396.1
Wall 3	140.0	3.0	3.3	208.1	377.0
Test average	145.3	2.8	3.3	180.6	386.8
Model	144.2	3.0	3.6	189.5	438.0
URM-0.75-NO-3.0					
Wall 1	186.0	1.9	3.9	168.2	610.1
Wall 2	200.0	2.5	3.4	219.6	543.8
Wall 3	216.0	2.0	3.1	147.6	494.6
Test average	200.7	2.1	3.5	178.5	549.5
Model	193.0	2.4	3.0	189.0	451.6
URM-0.75-NO-4.0					
Wall 1	231.0	2.3	5.4	194.7	1081.3
Wall 2	250.0	4.0	5.9	159.4	1171.9
Wall 3	238.0	2.2	3.1	152.5	563.0
Test average	239.7	2.9	4.8	168.9	938.7
Model	226.7	2.4	2.7	209.9	453.8

Source: Author.

Figure 66: Percentage differences between the model and average test response measures for URM walls.



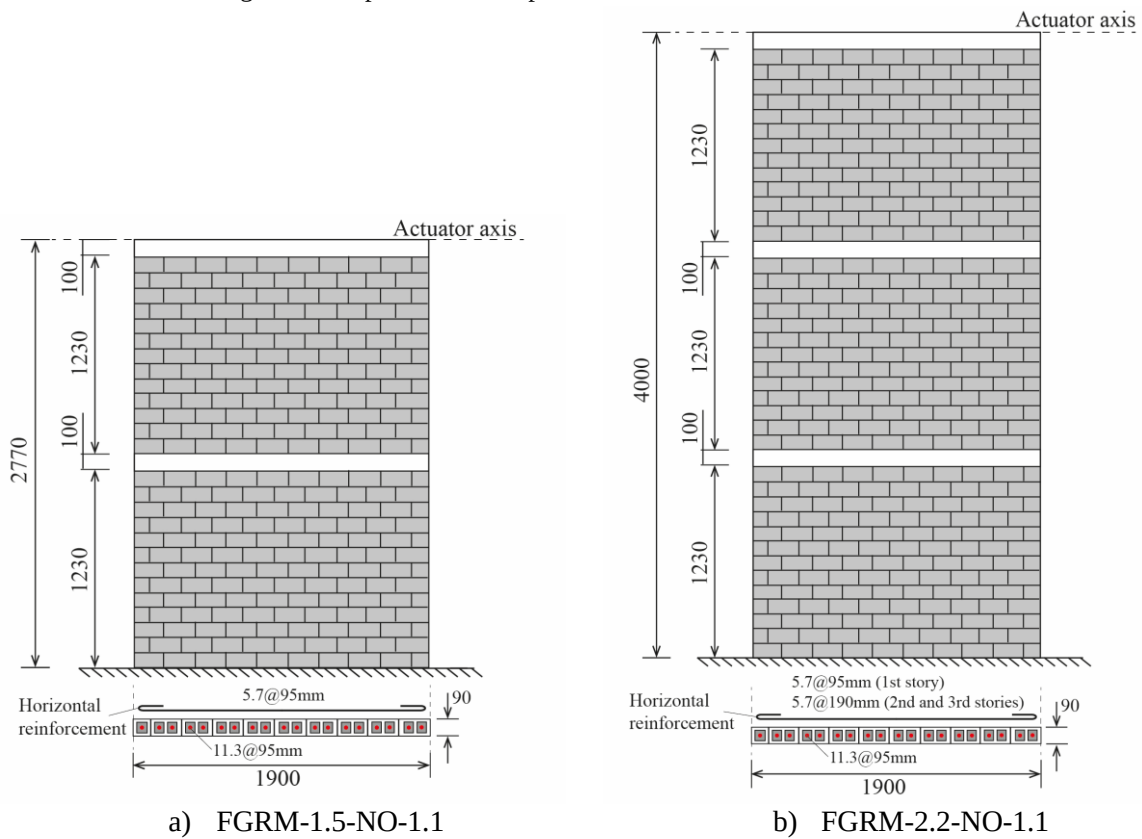
The model effectively captures the positive impact of the increase in axial stress on the lateral force resistance of the wall. The accuracy in predicting the capacity of the wall is remarkable, especially considering the complexity of the mechanisms involved. Comparing the numerical results against the average of the tests, the percentage difference varies from 0.8% to 5.4%. However, the increase in the axial load resulted in a reduction of d_{vu} and, consequently, a decrease in the ductility of the numerical response. While this behavior is in line with what is reported in the existing literature (Bolhassani et al., 2016; Ghanem et al., 1992; Haach; Vasconcelos; Lourenço, 2011; Ramírez; Sandoval; Almazán, 2016; Voon; Ingham, 2006), it is noteworthy that Oan's experimental walls, in general, deviate from this trend. As shown in Figure 66, this deviation led to a conservatively high percentage difference in the ultimate displacement and dissipated energy concerning the average of the three walls URM-0.75-NO-4.0. Closer agreement is evident when comparing only Wall 3 of URM-0.75-NO-4.0 with the numerical model results. Specifically for this wall, the percentage differences for ultimate displacement and energy are -13% and -19.4%, respectively. This highlights the complexity and well-known high variability of the behavior of masonry walls, even if nominally with the same characteristics.

4.3.2 FGRM walls

4.3.2.1 Description of the experimental walls

Two half-scale FGRM walls tested by Shedid et al. (2010b) were selected as benchmarks for this study. The aspect ratio is the notable difference between the two selected walls; they represent two and three-storey structures, with aspect ratios of 1.5 and 2.2, respectively, as illustrated in Figure 67. The walls were subjected to lateral cyclic loading with displacement control and an axial stress of approximately 1.1 MPa. These walls were designed to behave mainly in flexure. The vertical reinforcement consisted of 19 bars 11.3 mm in diameter, while the horizontal reinforcement consisted of bars with a diameter of 5.7 mm.

Figure 67: Experimental setup of FGRM walls; dimensions in mm.



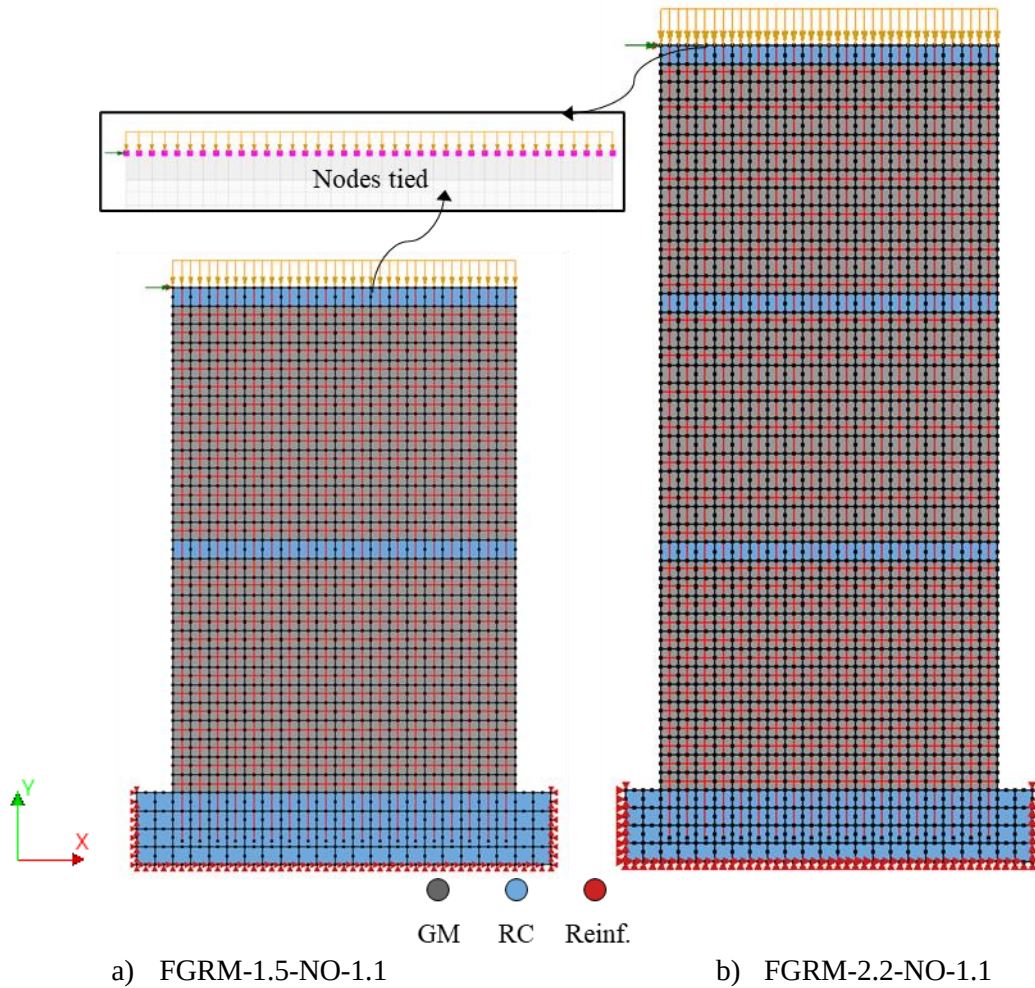
Source: Adapted from Shedid et al. (2010b)

4.3.2.2 Description of the numerical model

The configurations of the numerical models of the FGRM walls described above are depicted in Figure 68. The GM regions have a 90 mm thickness corresponding to the total half-scale block's thickness. Each FE has a size of 95×95 mm. The nodes at the foundation in the model were constrained horizontally and vertically. The self-weight based on the mass density

of 2400 kg/m³ attributed to the masonry was added to the structure, along with a constant distributed vertical load on the top equal to 90 N/mm, representing the external axial load, corresponding to an applied axial stress of approximately 1.1 MPa. The horizontal displacement at the top was applied in constant increments of 0.02 mm.

Figure 68: Numerical model of FGRM walls.



Source: Author.

The material properties of the FGRM wall models are provided in Table 8. The compressive strength, f_m , was defined based on the average results of tests conducted on GM prisms by Shedid et al. (2010b). The Young's modulus, E , was estimated as $900 \cdot f_m$, according to TMS 402-602 (2022). A Poisson's ratio, ν , of 0.23 was adopted based on (2013). The tensile strength (f_t) was estimated according to Equation 9 and previously used in the numerical models by Medeiros et al. (2022a) and Rahim et al. (2023). The tensile fracture energy, G_f , was calculated through Equation 10. The compressive fracture energy, G_c , for GM was assumed to be $1.69 \cdot f_m$. This last assumption, related to the definition of an average ductility index in

compression (ratio between fracture energy and strength), is based on experimental data reported by Nakamura and Higai (1999) for cylindrical concrete specimens and was previously employed by Calderón et al. (2021d).

As mentioned previously, the anisotropy and plane of weakness introduced by mortar joints are neglected for GM regions. For the steel reinforcements, the yield strength, f_y , and Young's modulus, E , values were adopted from the experimental program (Shedid; El-Dakhakhni; Drysdale, 2010b). The RC base was modeled as a linear elastic material with a Young's modulus of 30 GPa and a Poisson's ratio of 0.15.

Table 8: Material properties adopted for FGRM walls.

Element	Young's modulus (E) N/mm ²	Compressive strength (f_m) N/mm ²	Compressive fracture energy (G_c) N/mm	Tensile strength (f_t) N/mm ²	Tensile fracture energy (G_f) N/mm	Yield strength (f_y) N/mm ²
FGRM-1.5-NO-1.1 and FGRM-2.2-NO-1.1 (Shedid; El-Dakhakhni; Drysdale, 2010b)						
GM	14760	16.4*	27.72	1.34	0.121	-
Ver. Reinf.	200600			-		534*
Hor. Reinf.	200600			-		495*

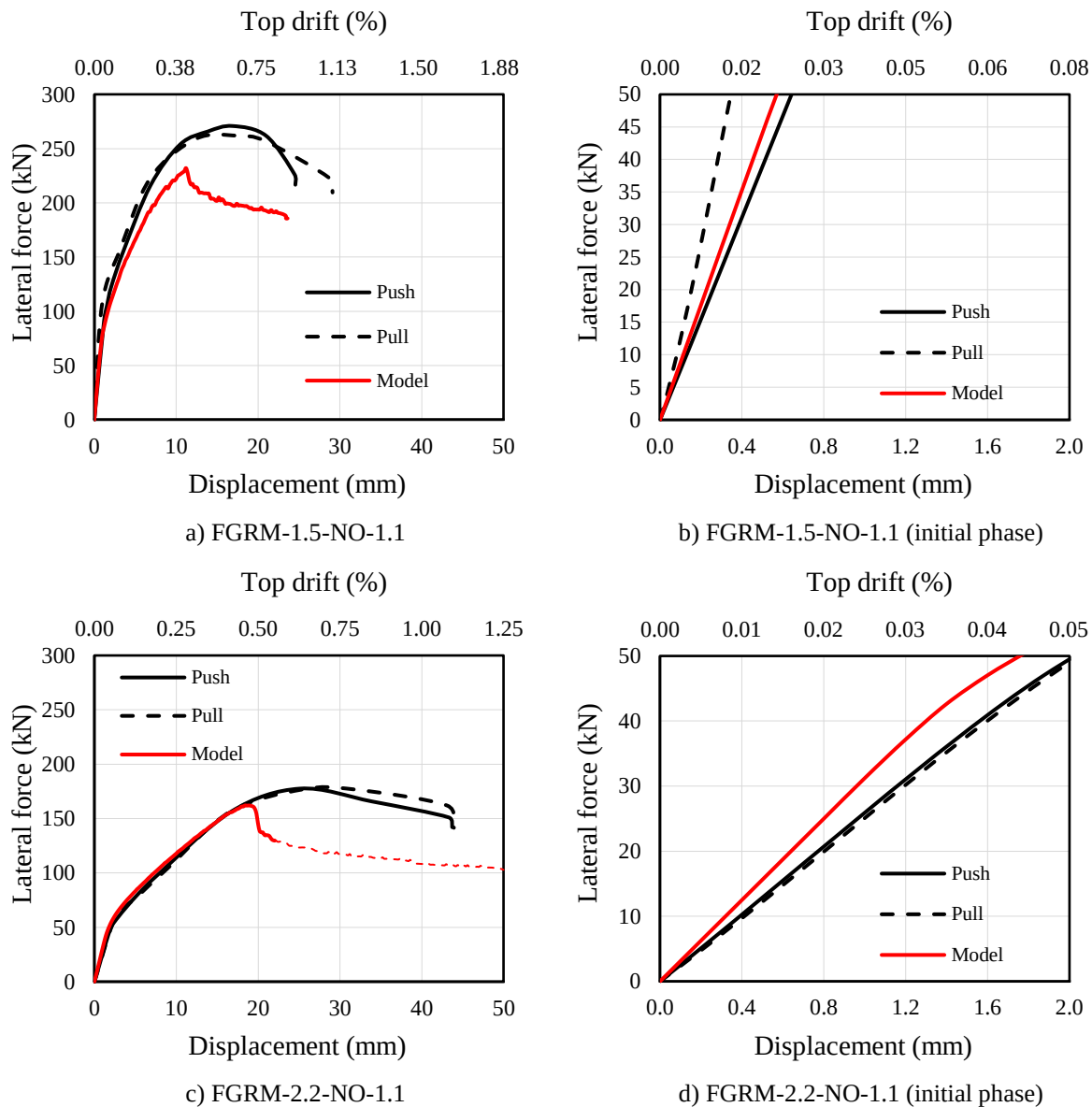
*Based on experimental data

Source: Author.

4. 3.2.3 Numerical vs. experimental comparison

The comparison of the force–displacement curves is presented in Figure 69. Again, the complete responses are presented on the left and a zoomed-in view of the initial response on the right.

Figure 69: Force–displacement curves of FGRM walls.



Source: Author.

It can be observed that the numerical models exhibit a development very similar to that obtained experimentally, from the initial phase up to the numerical peak load, which was, in any case, conservative compared to the experimental results in both walls. Nevertheless, the numerical model satisfactorily captured the wall aspect ratio influence on V_{max} , d_{Vmax} , and K_{sec} . When the aspect ratio is increased (in this case by adding another storey), V_{max} and K_{sec} reduce while d_{Vmax} increases, both in the numerical and experimental cases. The force–displacement curves are quantitatively summarized in Table 9. Additionally, the percentage differences between the model results and the experimental pull and push average responses are revealed in Figure 70.

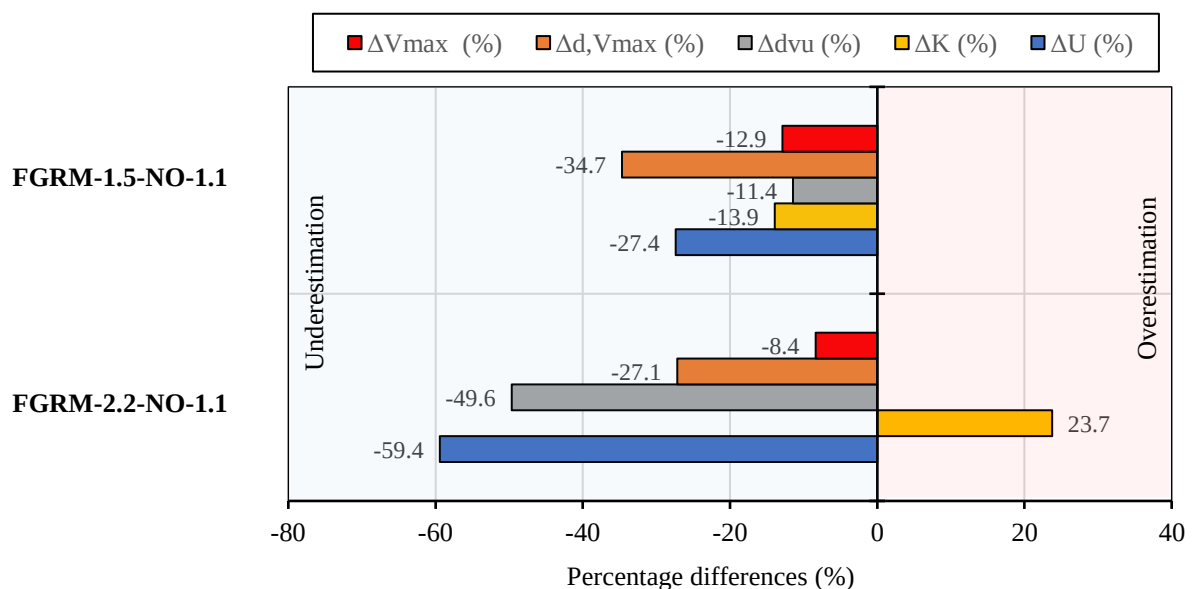
Table 9: Summary of key response measures in the force-displacement curves for FGRM walls.

Response	V_{max} (kN)	d_{Vmax} (mm)	d_{Vu} (mm)	K_{sec} (kN/mm)	U (kJ)
FGRM-1.5-NO-1.1					
Push	271.0	16.9	24.6	76.5	5477.9
Pull	262.0	17.4	29.1	112.4	6462.4
Test average	266.5	17.2	26.8	94.4	5970.2
Model	232.1	11.2	23.6	78.3	4306.3
FGRM-2.2-NO-1.1					
Push	177.4	25.1	43.7	20.0	6080.7
Pull	176.7	25.4	43.9	23.2	6229.1
Test average	177.1	25.3	43.8	21.6	6154.9
Model	162.2	18.4	22.1	26.6	2496.8

Source: Author.

The model clearly estimated the lateral force resistance conservatively. However, the percentage differences of -12.9% and -8.4% for the FGRM-1.5-NO-1.1 and FGRM-2.2-NO-1.1, respectively, can be considered satisfactory for engineering purposes. The secant stiffness is adequately predicted, with the two-storey model having lower stiffness than the experimental value (-13.9%), while the three-storey wall exhibits higher stiffness (+23.7%). These results indicate that the simplified estimation of Young's modulus at $900 \cdot f_m$ proved to be suitable for capturing the initial stiffness of these walls.

Figure 70: Percentage differences between the model and average test measures for FGRM walls.



Source: Author.

The most divergent results concerning the experiment, as evidenced in Figure 70, are d_{Vu} (-49.6%) and Energy (-59.4%). This conservative result occurs because, after reaching V_{max} , the numerical model quickly undergoes a 20% degradation of V_{max} (see Figure 69). Consequently, the numerical d_{Vu} was less than the experimental d_{Vu} . The area of the graph beyond this point is not taken into account for energy measurement. Nevertheless, it can be observed in Figure 69c that, after this point (d_{Vu}), the softening regime of the numerical model continues with a low degradation rate.

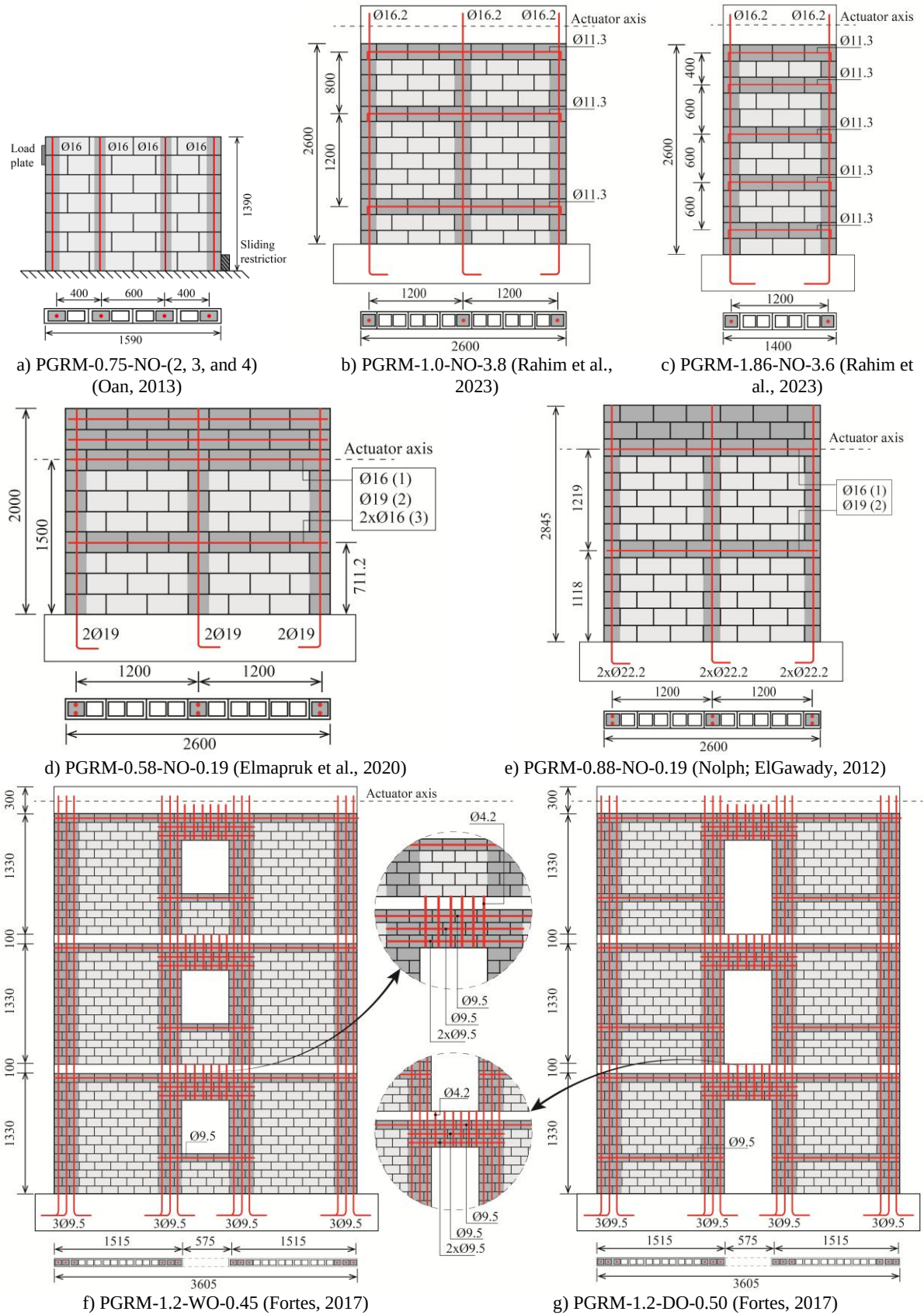
4.4 VALIDATION OF PGRM WALLS

Recognizing the greater complexity of PGRM compared to URM and FGRM, this section delves into the evaluation for a second-level validation of the model, including a range of scenarios involving PGRM walls.

4.4.1 Description of the experimental walls

The considered experimental tests were conducted by different authors, namely Oan and Shrive (2010, 2013), Rahim et al. (2023), Elmapruk et al. (2020), Nolph and ElGawady (2012), and Fortes (2017). The structural details of the walls are presented in Figure 71. The walls modeled present varied characteristics, highlighting axial stress, aspect ratio, presence of openings, and loading protocol. By integrating data from multiple sources, a more nuanced understanding of the variability and performance of PGRM walls is covered, contributing to a more robust dataset and reliable model validation.

Figure 71: Experimental arrangement of PGRM walls; dimensions in mm.



Source: Author.

Nine full-scale PGRM walls were selected from Oan's experimental program (Oan, 2013; Oan; Shrive, 2010, 2013). These walls share similar characteristics with the URM walls previously presented, categorized based on three axial stress levels (2 MPa, 3 MPa, and 4 MPa) determined by the net area of the wall. Three replicas were tested in each category. The vertical reinforcement in walls consisted of 16 mm diameter steel bars. This reinforcement was not fixed to the concrete base beam to avoid any yielding of the vertical bars due to the uplifting of the wall heel. These bars adhered only to the grout surrounding them. This specific configuration was chosen to study PGRM as an independent material, disregarding the structural effects of connecting the reinforcement to other elements within a building, including the dowels for vertical reinforcement. The arrangement of the vertical reinforcement and further details are illustrated in Figure 71a.

Two full-scale PGRM wall specimens subjected to cyclic in-plane lateral loads, tested by Rahim et al. (2023), were also modeled in this study. One wall was squat, and the other was slender, featuring aspect ratios of 1.00 and 1.86, respectively. Hollow concrete blocks with dimensions of $390 \times 190 \times 190 \text{ mm}^3$ were used to build the walls in running bond. The vertical reinforcement consisted of 16.2 mm diameter steel bars, while the horizontal reinforcement comprised 11.3 mm diameter steel bars. A high axial stress level of approximately 3.8 MPa (based on net area) was applied on these walls. A comprehensive visualization of the design details for these walls is presented in Figure 71b and Figure 71c. The design of these walls aimed to replicate practical details and construction practices typically observed in partially grouted walls built in regions characterized by low seismic hazards, meeting the minimum requirement ratios in TMS 402/602 (2022) and CSA S304 (2014).

Three full-scale walls with shear-dominated behavior, originating from the experimental investigation conducted by Elmapruk et al. (2020), were also modeled. These walls, with an aspect ratio of 0.58, were subjected to cyclic in-plane lateral loads. An axial stress of approximately 0.19 MPa (based on net area) was applied to simulate typical loads on a single-storey masonry building. The reinforcement in the walls was composed of 16 mm and 19 mm steel bars, as detailed in Figure 71d. Notably, the vertical reinforcement consisted of two bars per each grouted cell, and the horizontal reinforcement varied for each wall. The walls were constructed in face shell bedding using hollow concrete blocks with dimensions of $390 \times 190 \times 190 \text{ mm}^3$.

Two full-scale PGRM wall specimens from the study by Nolph and Elgawady (2012) were also investigated. These walls had an aspect ratio of 0.88 and were built and

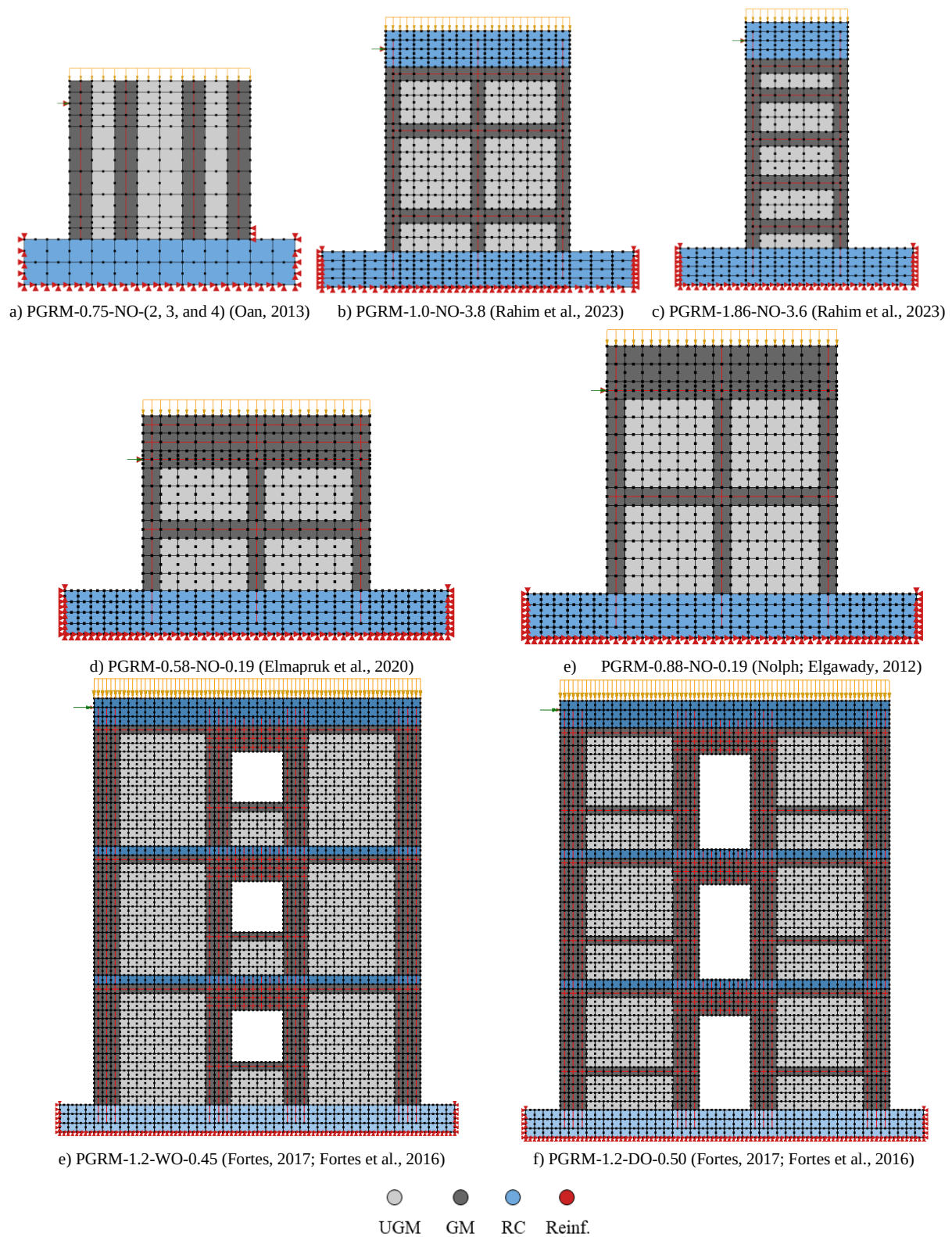
experimentally set up similarly to the walls of Elmapruk et al. (2020): they also showed shear-dominated behavior. The reinforcement comprised steel bars of around 22 mm for vertical reinforcement and 19 mm and 16 mm for horizontal reinforcement. The vertical reinforcement consisted of two bars per grouted cell, and the horizontal reinforcement varied for each wall, as shown in Figure 71e.

Lastly, four half-scale PGRM wall specimens subjected to cyclic in-plane lateral load, from the study conducted by Fortes et al. (2016), were also modeled. Among these specimens, two nominally identical walls featured door openings, while the other two, also nominally identical, had window openings. The walls had a three-storey configuration, resulting in an aspect ratio of 1.2. The axial stress applied to these walls was approximately 0.5 MPa. The walls were constructed using half-scale hollow concrete blocks ($185 \times 90 \times 90 \text{ mm}^3$) with face shell mortar bedding using half-scale 5 mm mortar joints. The vertical and horizontal reinforcement consisted of steel bars with 9.5 mm diameter. Stirrups with 4.2 mm diameter were strategically placed to connect courses above the openings with the RC slab wing, forming the masonry beam. The structural details are depicted in Figure 71f and Figure 71g.

4.4.2 Description of the numerical model

The meshed models of those experimental walls are displayed in Figure 72. As the mesh size is based on the half block size, each finite element has dimensions of $200 \times 200 \text{ mm}^2$, except for the Fortes' walls (Fortes, 2017; Fortes et al., 2016) (PGRM-1.2-WO-0.50 and PGRM-1.2-DO-0.45) constructed at half scale. For these ones, the dimension of each finite element was $95 \times 95 \text{ mm}^2$. The elements in the model have the net effective cross-section of the corresponding regions of the wall. Therefore, the elements for the UGM and GM regions were defined as 70 mm and 190 mm, respectively, except for PGRM-1.2-WO-0.50 and PGRM-1.2-DO-0.45 walls, where thicknesses of 30 mm and 90 mm were assigned to the UGM and GM regions, respectively.

Figure 72: Numerical models of PGRM walls.



Source: Author.

To simulate the cantilever conditions of the experimental walls, the nodes at the base of the models were constrained horizontally and vertically. In the first step, the walls were subjected to a vertical load corresponding to their axial stress. Self-weight was considered based on a mass density of 2400 kg/m³ attributed to UGM and GM, and the external axial load was applied as a constant distributed load on the wall top. In subsequent steps, horizontal displacement at the wall top was applied in constant increments. The displacement increments were set at 0.025 mm for the PGRM-0.75-NO-(2.0, 3.0, and 4.0) walls, 0.05 mm for the PGRM-0.58-NO-0.19 walls, and 0.10 mm for the other walls. These increments were chosen based on the expected ultimate displacement level of each wall to ensure an adequate number of steps for accurate analysis.

The material properties of the UGM, GM, and interfaces used to simulate the PGRM walls are provided in Table 10. These properties were selected based on a combination of material testing and model calibration. The compressive strength, f_m , of both UGM and GM was determined through compression tests conducted on masonry prisms by the authors referenced for each wall.

Young's modulus, E , was initially estimated at $900 \cdot f_m$, following TMS 402-602 (TMS, 2022). However, it is essential to mention the relatively high variability of E for masonry walls. Adjustments were necessary to calibrate the initial stiffness of the walls tested by Ba Rahim (2023) and Nolph and ElGawady (2012). For the UGM regions of the PGRM-1.0-NO-3.8 and PGRM-1.9-NO-3.6 walls, $E = 300 \cdot f_m$ was adopted, while $E = 600 \cdot f_m$ was used for the GM regions. The value of $300 \cdot f_m$ was derived from Ba Rahim's experimental data (2023) for ungrouted prism tests, and the $600 \cdot f_m$ value was calibrated by evaluating the initial stiffness of both the experimental walls and the numerical model. The value of $600 \cdot f_m$ has also been employed in numerical models of PGRM walls by Koutras and Shing (2020) based on the prism test data of Mavros (2015). For the PGRM-0.88-NO-0.19 walls, it is worth highlighting the low values needed for calibrating the initial stiffness, on the order of $100 \cdot f_m$ for both UGM and GM. Although the model stiffness adjusted well to the two experimental walls, the E/f_m ratio is much lower than that reported in the literature for PGRM, emphasizing the high variability in this type of masonry. Tomažević (1999) suggests Young's modulus in a range of $200\text{--}1,000 \cdot f_m$ for the design of masonry buildings. Values in the order of $100 \cdot f_m$ of magnitude have been reported for URM walls with lime mortar (Costigan; Pavía; Kinnane, 2015). The remarkably low initial stiffness may also be attributed to potential pre-existing cracks before the tests, resulting in the assumed low E/f_m ratio corresponding to a cracked stiffness.

For all wall models, a Poisson's ratio, ν , of 0.23 was adopted based on Oan's experimental test on hollow concrete blocks. The tensile strength, f_t , was estimated using Equation 9 presented previously, except for walls PGRM-1.2-WO-0.45 and PGRM-1.2-DO-0.45 (Fortes et al., 2016), where smaller values, defined according to CSA S304 (2014), allow better adjustment to experimental responses. The tensile fracture energy, G_f , and compressive fracture energy, G_c , for UGM were calculated as before. For GM regions, G_c was assumed to be $1.69 \cdot f_m$, also as before. An exception to the previous formulations for G_f and G_c was needed for PGRM-1.2-WO-0.45 and PGRM-1.2-DO-0.45 (Fortes et al., 2016). The values adopted by Haach et al. (2013) in numerical models of PGRM walls, also built on half scale units, provided a better fit for these experimental walls.

In addition to the lateral confinement model applied to all walls, for the walls tested by Elmapruk et al. (2020), Nolph and ElGawady (2012), and Fortes (2017), a compressive strength reduction model due to lateral cracking associated with the TSCM was considered. For this, the model proposed by Vecchio and Collins (1993), already implemented in the DIANA software, was adopted with a lower bound reduction curve of 0.6. According to Lourenço (1996), this correction helps adjust the internal force distribution, preventing an overestimation of resistance and displacement capacity. This approach effectively allowed the model to capture the behavior of the walls.

For the steel reinforcement, the yield strength, f_y , and ultimate strength, f_u , values were adopted from the respective experimental programs. In cases where experimental values of Young's modulus, E , for the steel reinforcement were not available, a standard value of 200 GPa was adopted. The RC base was modeled as a linear elastic material with a Young's modulus of 30 GPa and a Poisson's ratio of 0.15.

Table 10: Material properties adopted for PGRM walls.

Element	Young's modulus (E)	Compressive strength (f_m)	Compressive fracture energy (G_c)	Tensile strength (f_t)	Tensile fracture energy (G_f)	Cohesion (c)	Friction angle (ϕ)	Tensile strength (f_{ti})	Yield strength (f_y)	Ultimate strength (f_u)
	N/mm ²	N/mm ²	N/mm	N/mm ²	N/mm	N/mm ²	deg.	N/mm ²	N/mm ²	N/mm ²
PGRM-0.75-NO-(2.0, 3.0, and 4.0) (Oan, 2013; Oan; Shrive, 2010, 2013)										
UGM	14580	16.2*	21.0	1.33	0.120					
GM	8838	9.82*	16.6	1.03	0.110					
Interfaces			-			0.15*	44.7*	0.10		
Ver.Reinf.	200000								480*	-
PGRM-1.0-NO-3.8 and PGRM-1.9-NO-3.6 (Rahim et al., 2023)										
UGM	6600*	22*	22.7	1.55	0.127					
GM	9840	16.4*	27.7	1.34	0.121					
Interfaces			-			0.15*	44.7*	0.10		
Ver.Reinf.	188960								455*	679*
Hor. Reinf.	208989								521*	643*
PGRM-0.58-NO-0.19 (Elmapruk; ElGawady; Hassanli, 2020)										
UGM	15696	17.4*	21.4	1.38	0.122					
GM	21960	24.4*	41.2	1.63	0.129					
Interfaces			-			0.25	50.20	0.10		
Ver.Reinf.	200000								427*	-
Hor. Reinf.	200000								425*	-
PGRM-0.88-NO-0.19 (Nolph; ElGawady, 2012)										
UGM	1130	11.3*	19.4	1.11	0.113					
GM	1970	19.7*	33.3	1.46	0.125					
Interfaces			-			0.15*	44.7*	0.10		
Ver.Reinf.	200000								439*	-
Hor. Reinf.	200000								439*	-
PGRM-1.2-WO-0.45 and PGRM-1.2-DO-0.45 (Fortes, 2017; Fortes et al., 2016)										
UGM	10620	11.8*	10	0.4	0.113					
GM	10980	12.2*	10	0.65	0.114					
Interfaces			-			0.15*	44.7*	0.10		
Ver.Reinf.	203512								540*	742*
Hor. Reinf.	222799								743*	812*

*Based on experimental data

Source: Author.

Cohesion, c , and friction angle, ϕ , are typically assessed using the European triplet test (EN 1052-3, 2002). As this specific test to characterize the block-mortar shear interaction was not conducted for all walls evaluated in this study, the values of c and ϕ assumed for the interfaces were inferred from shear triplet test results reported by Ba Rahim (2023) and Pettit (2023). These studies involved hollow concrete blocks and mortar compositions that resemble those utilized in the walls evaluated in this study. However, the shear-dominated behavior of the PGRM-0.58-NO-0.19 walls (Elmapruk; ElGawady; Hassanli, 2020) caused higher demands on the interface elements, requiring higher values of c and ϕ to match the experimental results better. Simultaneously, increasing c and ϕ enhanced the lateral load capacity and delayed the stiffness degradation after the initial linear regime, which is attributed to interface damage. Additionally, a tensile strength, f_{ti} , of 0.10 MPa was assumed for interfaces in all models. The normal and shear stiffness components were calculated using Equations 7 and 8, as before, based on the elastic properties of UGM.

4.4.3 Numerical vs. experimental comparison

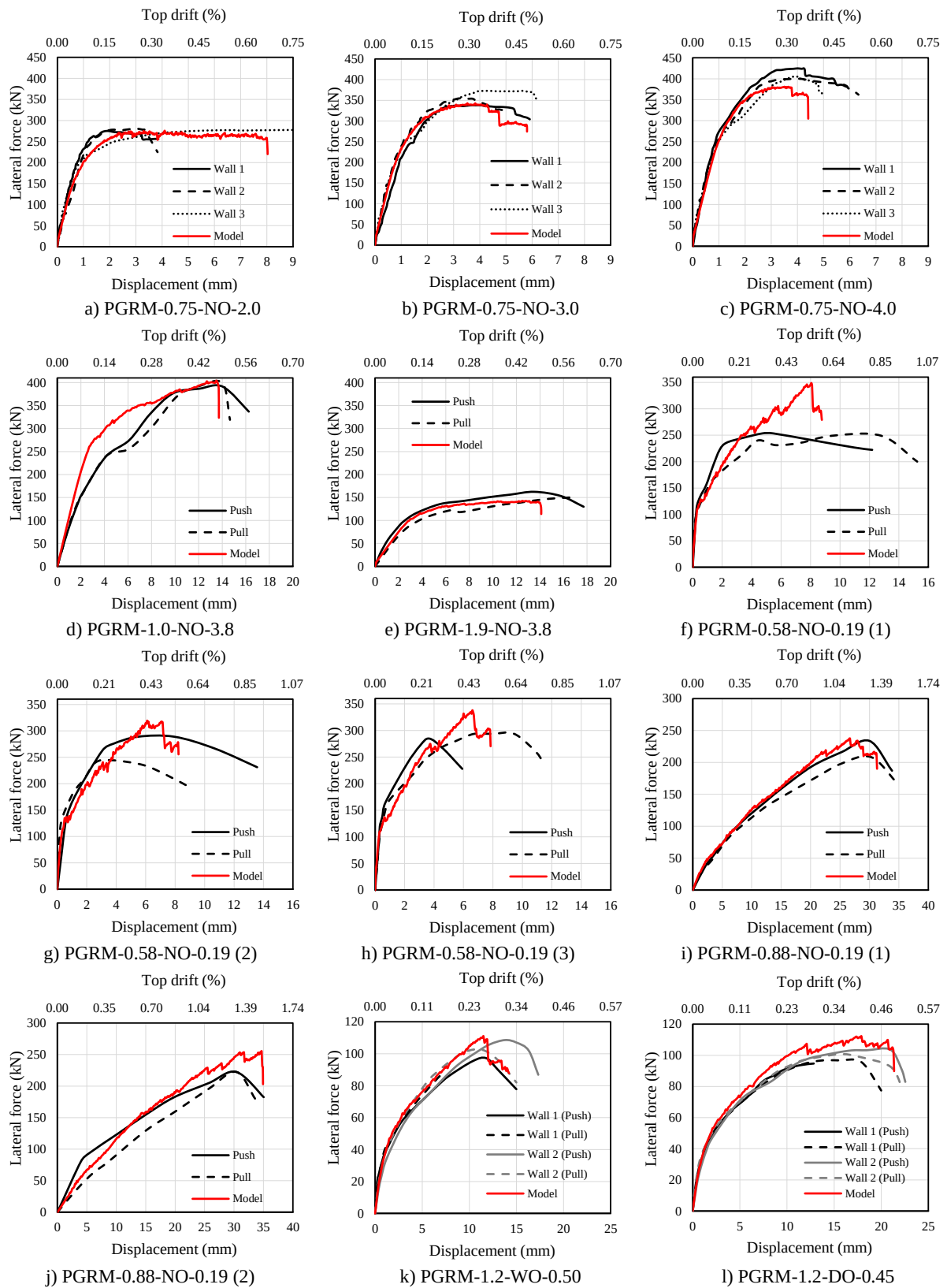
PGRM walls exhibit a more complex behavior than URM and FGRM due to the interaction between the grouted/reinforced masonry and the ungrouted regions. These two regions exhibit differences in stiffness, and partial grouting introduces non-uniformity in the cross-section. Additionally, the literature predominantly focuses on UGM and FGRM, with limited data available on PGRM. Therefore, the numerical and experimental results were compared more comprehensively than for PGRM than for URM and FGRM walls. Damage patterns and reinforcement strains were also evaluated, as well as the force–displacement curves.

The force–displacement curves derived from the tests and numerical analysis are superposed in Figure 73. Note that the scales of the graph axes vary for each experimental program. This arrangement ensures optimal visualization of all graphs in Figure 73 and facilitates direct comparisons between walls within each experimental program.

The key response measures are reported in Table 11, providing a comprehensive evaluation of the model performance. These results include initial stiffness (K_{sec}), maximum load (V_{max}) and corresponding displacement ($d_{V_{max}}$), ultimate displacement corresponding to a 20% reduction in V_{max} , and the dissipated energy (area under the curve).

In general, the simulated force–displacement curves closely resemble the experimental ones. For the walls tested by Oan (2013), there is an excellent agreement in predicting the maximum lateral force with percentage differences compared to the experimental average ranging from -1.3% to -7.2% (Figure 74). The numerical model accurately captures the influence of the axial stress level on the lateral force capacity of the PGRM walls. Compared to URM walls with the same characteristics that are not grouted/reinforced, it is evident that the model also convincingly captured the influence of grouting and reinforcement on the wall behavior. Satisfactorily conservative values were estimated for the initial stiffness, with the most significant percentage variations occurring for $d_{V_{max}}$ and d_{vu} of the PGRM-0.75-NO-2.0 walls. However, the high variability found between these experimental walls themselves should be noted. The dissipated energy corresponding to the numerical response of the walls is conservative compared to the experimental measurements. The rapid decrease in the bearing capacity after reaching an extensive plastic deformation and the peak load point indicates a mixed shear-flexure behavior for the numerical walls.

Figure 73: Force–displacement curves of PGRM walls.



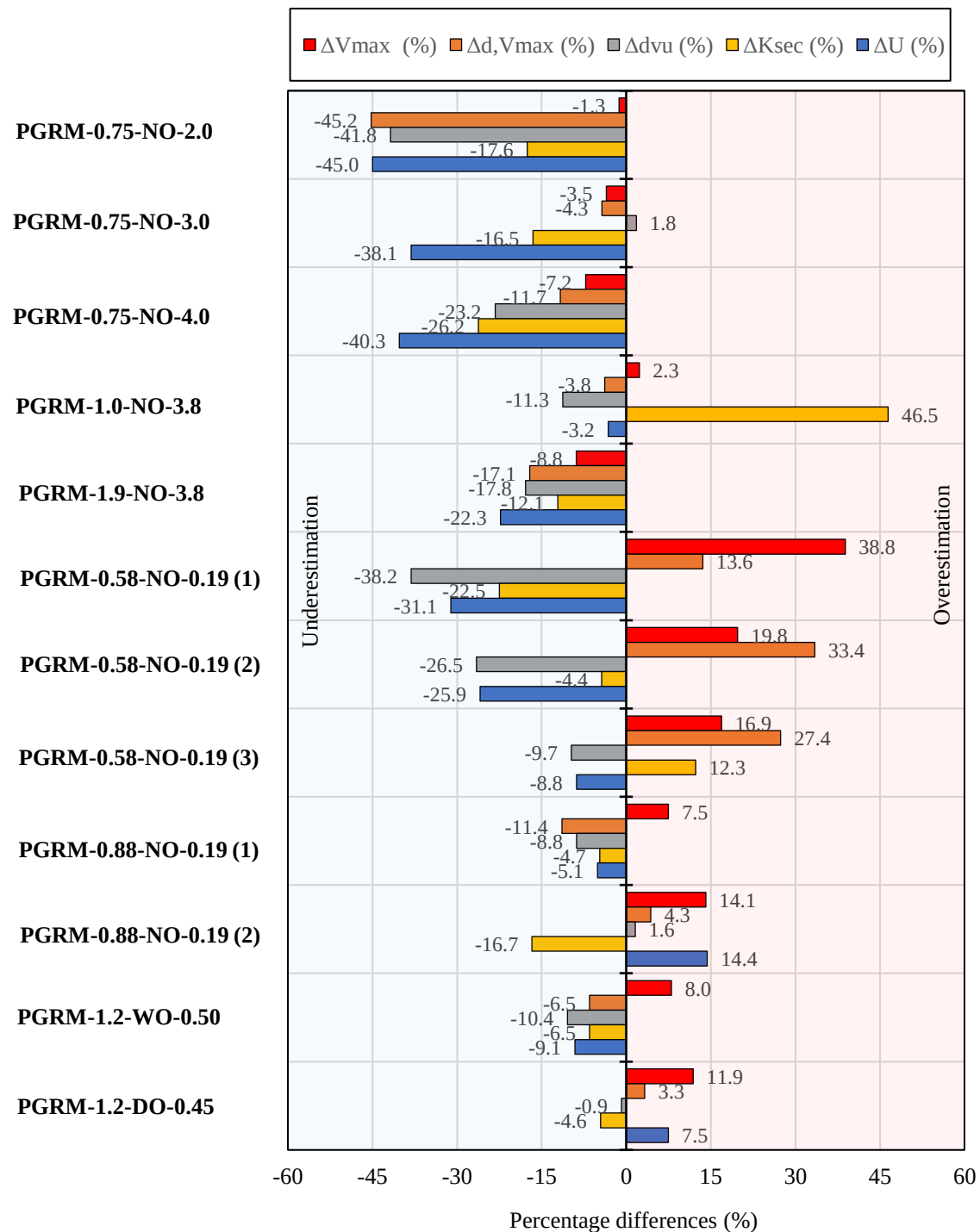
Source: Author.

Table 11: Summary of key response measures in force–displacement curves for PGRM walls.

	V_{max} (kN)	d_{Vmax} (mm)	d_{Vu} (mm)	K_{sec} (kN/mm)	U (kJ)
PGRM-0.75-NO-2.0 (Oan, 2013)					
Wall 1	275.0	1.9	5.1	367.7	1184.2
Wall 2	281.0	3.0	3.8	224.7	868.1
Wall3	279.0	13.2	32.5	387.5	8599.8
Test average	278.3	6.0	13.8	326.7	3550.7
Model	274.8	3.3	8.0	269.3	1953.5
PGRM-0.75-NO-3.0 (Oan, 2013)					
Wall 1	338.0	3.6	5.9	337.0	1655.6
Wall 2	355.0	3.3	5.0	330.8	1465.1
Wall3	373.0	4.2	6.2	289.5	1894.5
Test average	355.3	3.7	5.7	319.1	1671.7
Model	342.8	3.5	5.8	266.3	1034.0
PGRM-0.75-NO-4.0 (Oan, 2013)					
Wall 1	425.0	4.0	5.9	494.0	2051.2
Wall 2	400.0	3.8	6.3	310.9	2132.2
Wall3	406.0	4.1	5.0	307.1	1582.7
Test average	410.3	4.0	5.7	370.7	1922.0
Model	380.8	3.5	4.4	275.6	1148.1
PGRM-1.0-NO-3.8 (Rahim et al., 2023)					
Push	391.3	14.0	16.3	67.5	4748.5
Pull	398.8	14.0	14.7	80.1	4061.5
Test average	395.1	14.0	15.5	73.8	4405.0
Model	404.2	13.5	13.7	108.1	4265.4
PGRM-1.9-NO-3.8 (Rahim et al., 2023)					
Push	162.4	13.5	17.7	45.8	2335.9
Pull	150.1	16.6	16.6	34.3	1902.8
Test average	156.3	15.1	17.2	40.1	2119.4
Model	142.4	12.5	14.1	35.2	1646.3
PGRM-0.58-NO-0.19 (1) (Elmapruk; ElGawady; Hassanli, 2020)					
Push	253.5	4.7	13.1	429.2	2961.8
Pull	249.1	9.4	15.2	528.9	3367.9
Test Average	251.3	7.1	14.2	479.1	3164.8
Model	349.0	8.1	8.8	371.2	2180.2
PGRM-0.58-NO-0.19 (2) (Elmapruk; ElGawady; Hassanli, 2020)					
Push	289.1	5.5	13.6	232.8	3408.2
Pull	244.7	3.7	8.8	583.8	1896.8
Test average	266.9	4.6	11.2	408.3	2652.5
Model	319.6	6.1	8.2	390.5	1965.2
PGRM-0.58-NO-0.19 (3) (Elmapruk; ElGawady; Hassanli, 2020)					
Push	284.7	3.7	5.9	356.7	1344.2
Pull	293.6	6.7	11.4	326.0	2856.6
Test average	289.1	5.2	8.7	341.4	2100.4
Model	338.0	6.6	7.8	383.3	1915.0
PGRM-0.88-NO-0.19 (1) (Nolph; ElGawady, 2012)					
Push	233.8	30.0	33.8	15.0	5213.2
Pull	208.1	30.3	34.8	13.0	4933.2
Test Average	221.0	30.1	34.3	14.0	5073.2
Model	237.5	26.7	31.3	13.4	4814.1
PGRM-0.88-NO-0.19 (2) (Nolph; ElGawady, 2012)					
Push	222.4	30.4	35.1	20.2	5367.7
Pull	222.4	30.4	33.7	9.3	4426.0
Test average	222.4	30.4	34.4	14.7	4896.9
Model	253.8	31.7	34.9	12.3	5601.3
PGRM-1.2-WO-0.50 (Fortes, 2017; Fortes et al., 2016)					
Wall 1 (Push)	97.2	11.8	15.0	40.3	1122.4
Wall 1 (Pull)	-	-	-	45.6	-
Wall 2 (Push)	108.5	13.8	17.3	25.1	1415.8
Wall 2 (Pull)	102.5	11.3	15.0	34.6	1199.3
Test average	102.7	12.3	15.7	36.4	1245.8
Model	111.0	11.5	14.1	34.0	1132.7
PGRM-1.2-DO-0.45 (Fortes, 2017; Fortes et al., 2016)					
Wall 1 (Push)	-	-	-	34.8	-
Wall 1 (Pull)	96.8	15.5	20.0	37.3	1583.1
Wall 2 (Push)	103.6	21.0	22.6	27.5	1887.8
Wall 2 (Pull)	100.7	15.5	22.1	37.5	1832.0
Test average	100.4	17.3	21.6	34.3	1767.7
Model	112.3	17.9	21.4	32.7	1899.7

Source: Author.

Figure 74: Percentage differences between the model and average test measures for the PGRM walls.



Source: Author.

For the walls tested by Ba Rahim (2023), there is again an excellent agreement in predicting V_{max} , with percentage differences about the average pull and push cycles of 2.3% and -8.8% for the squat and slender walls, respectively. Regarding the initial stiffness, while the model satisfactorily predicted K_{sec} of the slender wall (-12.1%), the numerical model

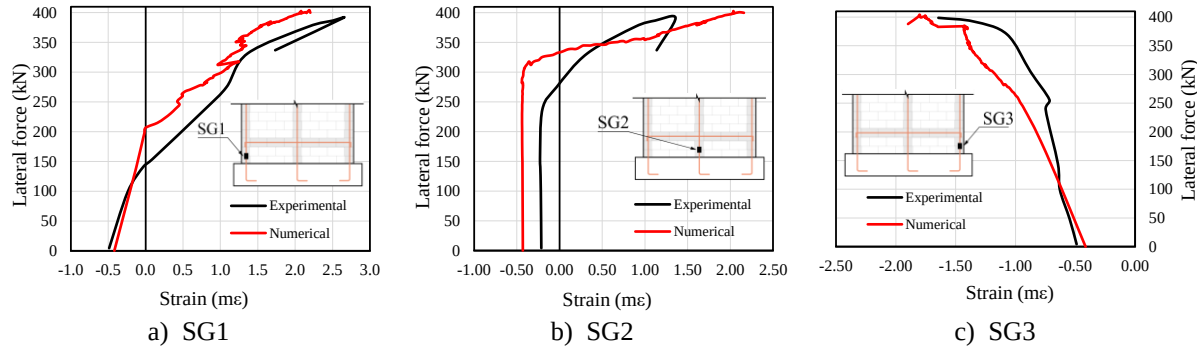
overestimated the stiffness of the squat wall (+46.5%). The overvaluation of the stiffness predicted by the numerical model for the squat wall can be attributed, in part, to a higher level of potential pre-existing cracks (prior to testing) in this wall compared to the slender walls. Lower E values could be adopted to improve the match. However, the response of the slender wall tested under the same laboratory conditions was satisfactorily captured using the same E values. The other parameters were predicted reasonably well, and conservative values of the dissipated energy were obtained.

For the walls tested by Elmapruk et al. (2020), the numerical models predicted a V_{max} slightly higher than the corresponding experimental value, in part due to irreversible damage induced by cyclic loading on the walls. Nevertheless, conservative values of dissipated energy were estimated. The initial stiffness is predicted with a percentage difference between -22.5% and 12.3%. Quantitatively, the response of the walls by Nolph and Elgawady (2012) and Fortes et al. (2016) were reproduced by the numerical model with a good level of agreement, as shown in Figure 74.

Considering the variability of the masonry itself, the association with grout and reinforcement in PGRM walls, the potential differences that may occur in the behavior of walls even between nominally identical tests and the influence of the loading protocol, the numerical models showed good accuracy in predicting the lateral response of the walls. The numerical differences reported here are within the ranges of experimental and numerical variability of masonry structures reported by various authors (Araya-Letelier et al., 2019; Arnau, Oriol; Sandoval; Murià-Vila, 2015a; Calderón; Milani; Sandoval, 2021d; Calderón; Sandoval; Arnau, 2017; Haach; Vasconcelos; Lourenço, 2011; Schreppers et al., 2017).

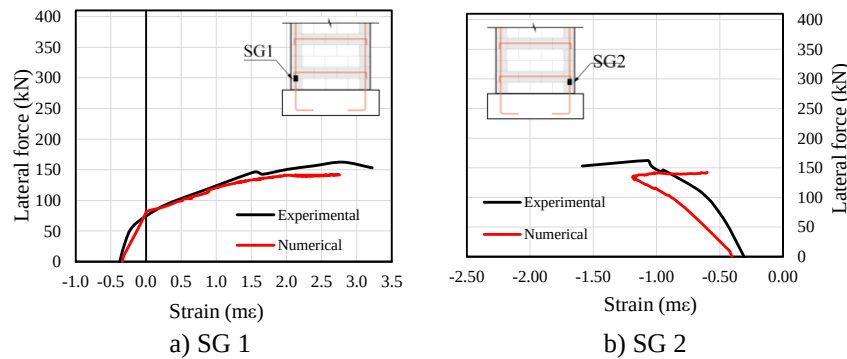
In addition to the force-displacement curves, the strains of the vertical reinforcement in the walls tested by Ba Rahim (2023) and Fortes et al. (2016) and the corresponding numerical models are compared in Figure 75 to Figure 78. The trend of the numerical force-strain curves is remarkably similar to the experimental responses. Therefore, considering the inherent variability in the experiments, it can be inferred that the model reproduced the behavior of critical reinforcements with satisfactory accuracy. Potential improvements could be achieved by introducing sliding interfaces between the reinforcement elements and the masonry. However, implementing such enhancements would increase model complexity and computational demand, deviating from the main objective of this study, which is to balance these issues by adopting a relatively low level of refinement for the model.

Figure 75: Force-strain curves of vertical reinforcement in PGRM-1.0-NO-3.8 wall (Rahim et al., 2023).



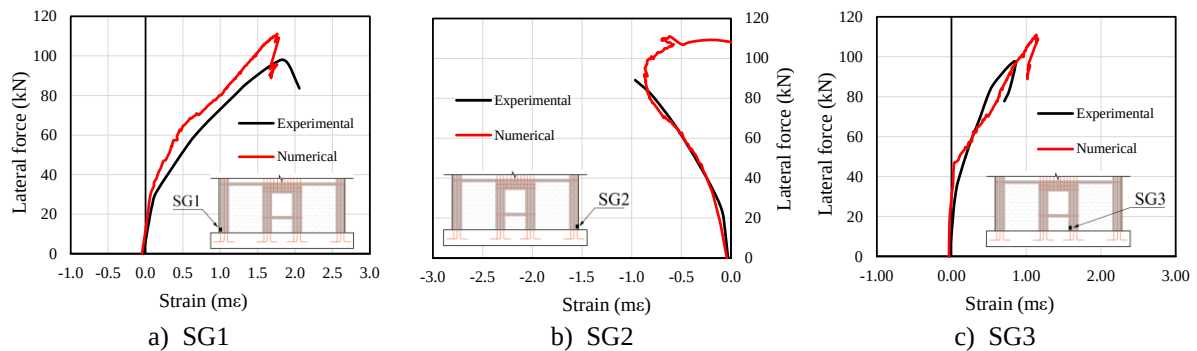
Source: Author.

Figure 76: Force-strain curves of vertical reinforcement in PGRM-1.9-NO-3.8 wall (Rahim et al., 2023).



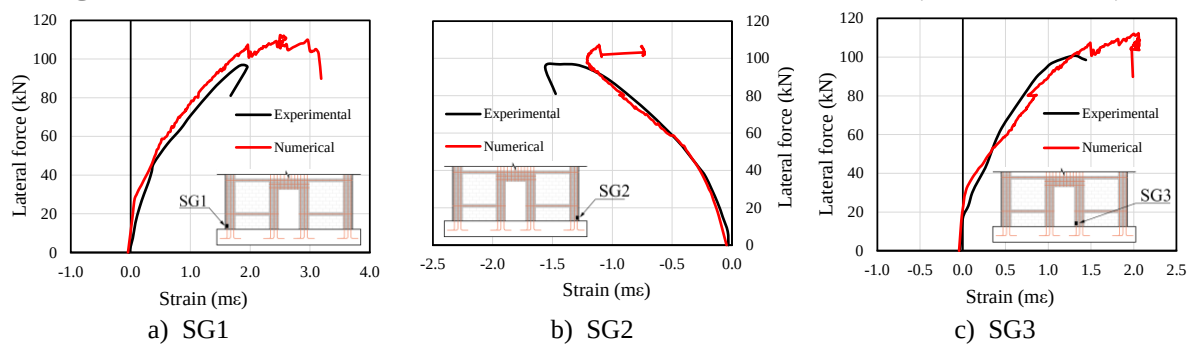
Source: Author.

Figure 77: Force-strain curves of vertical reinforcement in PGRM-1.2-WO-0.50 (Fortes et al., 2016).



Source: Author.

Figure 78: Force-strain curves of vertical reinforcement in PGRM-1.2-DO-0.45 (Fortes et al., 2016).



Source: Author.

To demonstrate the damage pattern produced by the numerical models, the two walls tested by Ba Rahim (2023) were selected: PGRM-1.0-NO-3.8 and PGRM-1.9-NO-3.8 because these two walls were designed for different failure modes. PGRM-1.0-NO-3.8 wall is squat-type, where the predominant failure mode is shear, while PGRM-1.9-NO-3.8 wall is slender-type, with predominant flexural failure.

The experimental and numerical damage patterns of the PGRM-1.0-NO-3.8 and PGRM-1.9-NO-3.8 walls are displayed in Figure 79 and Figure 80, respectively. The cracking patterns of the experimental walls are indicated based on a schematization presented by Ba Rahim (2023) and a photo of the final stage after the test. Regarding the PGRM-1.9-NO-3.8 wall, it is worth noting that the observed spalling of the face-shell is a consequence of wall transportation after the test and is not part of the experimental damage pattern. For the numerical model, the damage pattern corresponds to the maximum lateral force is illustrated in Figure 78b and Figure 79b with maps of lateral displacement, interfaces detachment (equivalent to cracks in the mortar joints), crack direction on the walls, and the formation of compressive struts (measured in terms of minimal principal stresses).

Figure 79: Experimental and numerical damage pattern for the PGRM-1.0-NO-3.8 wall.

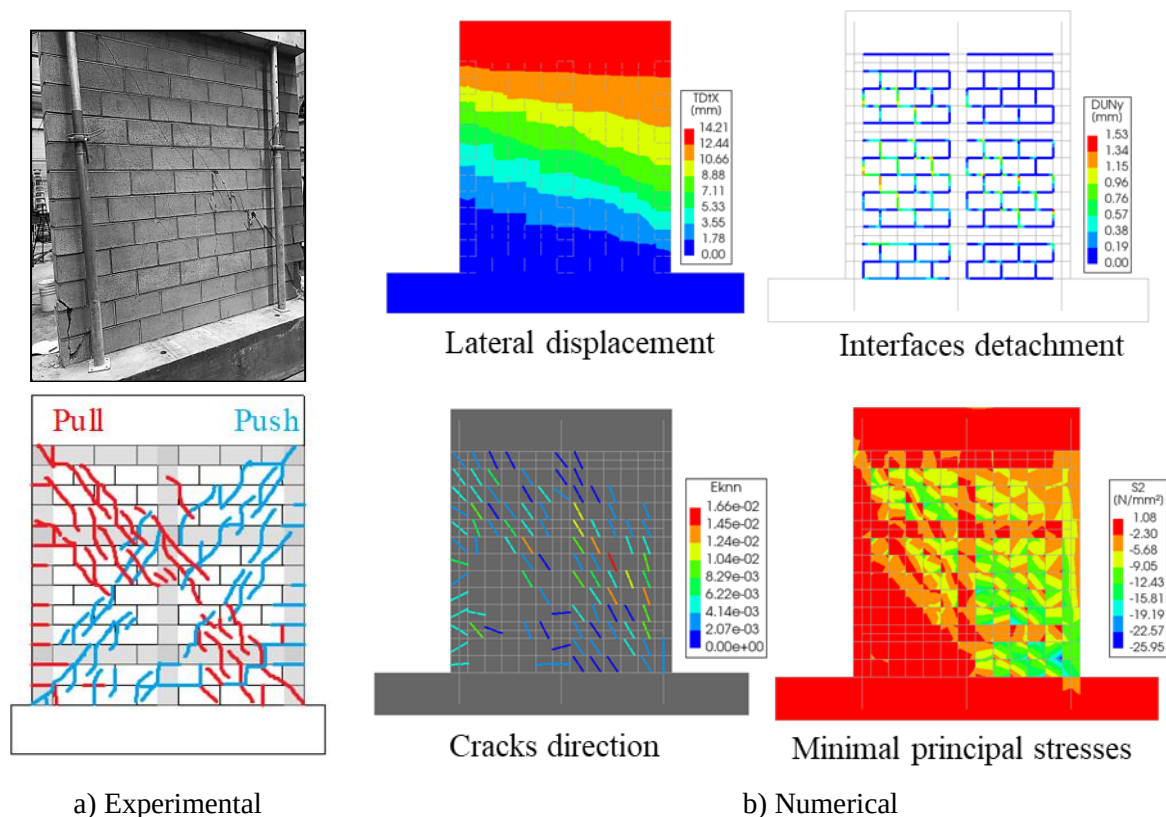
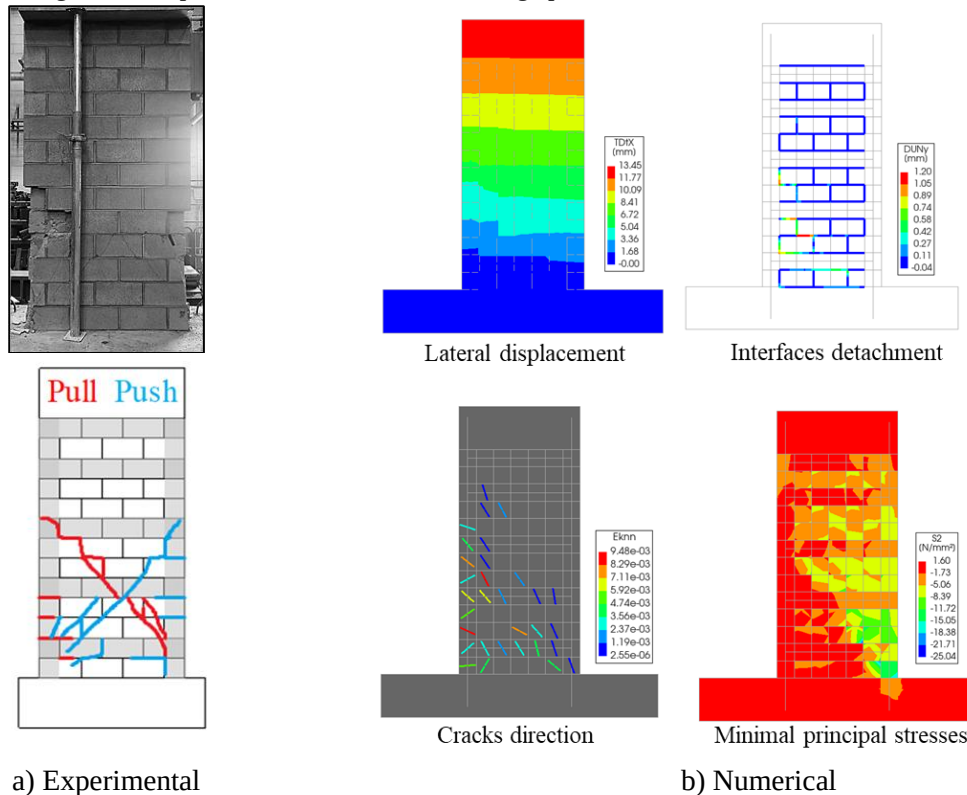


Figure 80: Experimental and numerical damage pattern for the PGRM-1.9-NO-3.8 wall.



Source: Author.

The comparison in Figure 79 and Figure 80 reveals an apparent similarity between the numerical and experimental crack patterns. The model successfully replicated the primarily shear-dominated model of the PGRM-1.0-NO-3.8. This included the formation of stair-stepped cracks along the mortar joints, simulated by interfaces, and diagonal cracking on the wall, reproduced by the continuous elements. Additionally, horizontal cracks were visible in the tensioned heel (left edge). Vertical cracks and high compressive stresses on the right edge were observed on the model, indicating toe crushing.

In contrast, the PGRM-1.9-NO-3.8 slender wall failed primarily by flexure. This is also confirmed by the vertical reinforcement strains presented in Figure 76, which revealed that the outermost vertical bars were yielding. The damage pattern is characterized by horizontal cracks in the bottom of the walls. Diagonal cracks were visible in the lower wall half, where the shear load is transferred, followed by toe crushing. Therefore, considering the complexity of the cracking phenomenon in masonry specimens, the results indicate that the model accurately reproduced the damage pattern in the tested walls.

4.5 SENSITIVITY ANALYSIS

A deterministic sensitivity analysis is important to measure the impact of each input parameter in the results of numerical models due to the variability in the estimation procedure for each parameter. It empowers the user to prioritize accurate estimates of the most influential inputs. This is indeed of great relevance to practice and can give an idea of the potential differences and uncertainties faced in estimates for design purposes.

The sensitivity analysis conducted is not intended to be a generalized study of how determined parameters influence the PGRM wall behavior. Instead, its purpose is to demonstrate the implications during the calibration process. A complex interaction between input parameters influences in the nonlinear behavior of PGRM walls. Therefore, it is expected that different governing parameters can be identified depending on the characteristics of each model evaluated, such as wall aspect ratio, axial load level, boundary conditions, and reinforcement detailing.

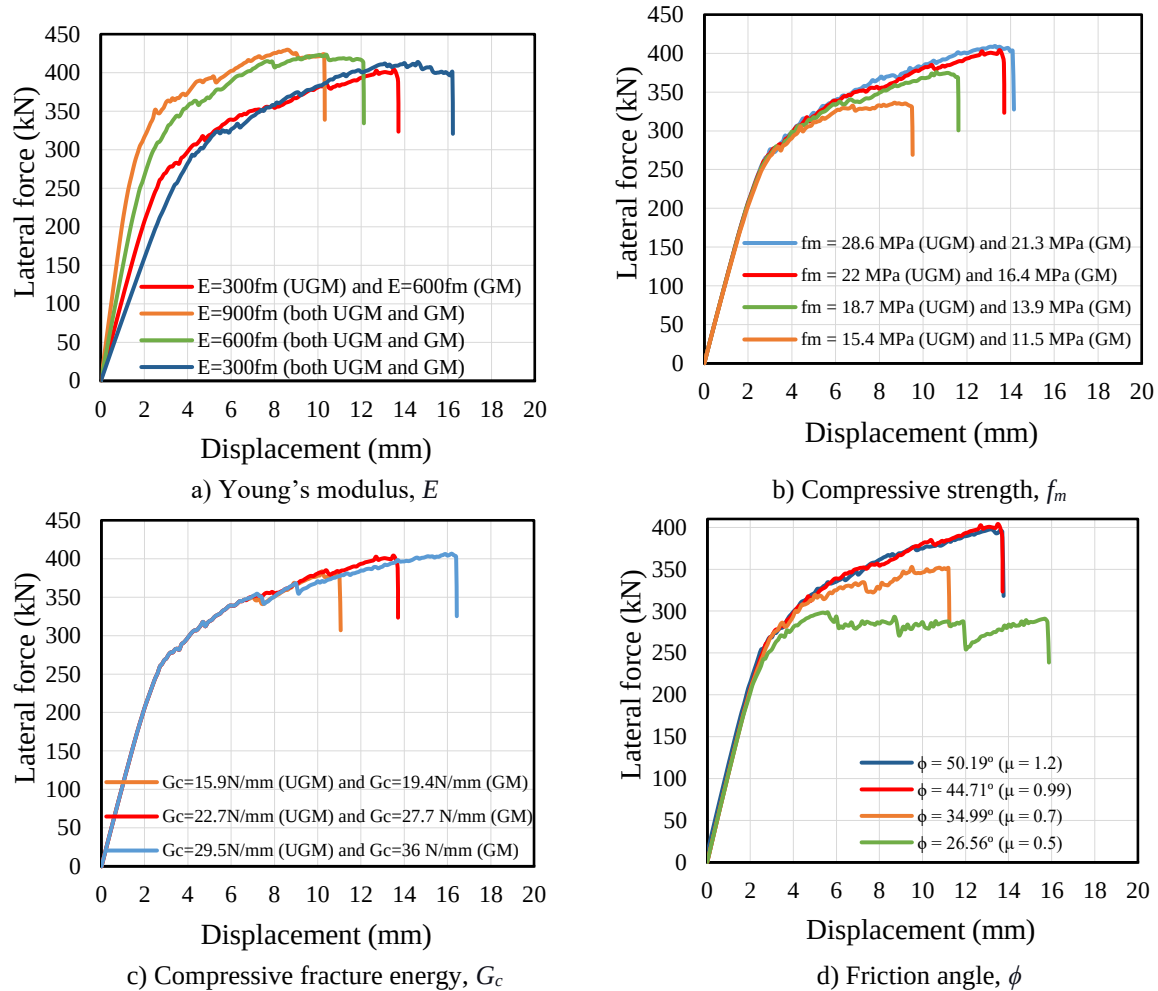
The PGRM-1.0-NO-3.8 wall was selected for the sensitivity analysis for brevity and objectivity. This wall was extensively evaluated regarding force–displacement curves, damage patterns, and reinforcement strains, which complements the validation results presented before.

Accordingly, the influence of some input parameters on the force–displacement response of the model is measured. The input parameters evaluated herein include properties of the masonry (UGM and GM regions) and the interface. As mentioned, URM is simulated as an assemblage of blocks with interface joints while GM is modeled as a continuous material.

The influence of the parameters that most impact the model is reflected in the force–displacement responses in Figure 81. These parameters are Young’s modulus, E , compressive strength, f_m , compressive fracture energy, G_c , and friction angle, ϕ (also presented in terms of friction coefficient, μ , in Figure 81d). For the sensitivity analysis of f_m and G_c , each parameter was individually changed within an interval of $\pm 30\%$ of the calibrated model value. The values of these parameters were changed simultaneously, but in the same proportion, for the UGM and GM regions to ensure that there were no significant differences between the two regions concerning the validated parameters. The range for the value of E was chosen to be within the interval found by Tomažević (1999) ($200\text{--}1000 \cdot f_m$). The variation range of the friction angle was based on the results of van der Pluijm (1993), in which the friction coefficient ranged from 0.7 to 1.2 for different unit-mortar combinations. The value of each parameter is indicated in

Figure 81 through the subtitles for each curve. Note that when a certain parameter was changed, the others were kept at the same values listed in Table 10.

Figure 81: Sensitivity analysis applied to the wall PGRM-1.0-NO-3.8 (Rahim et al., 2023).



Source: Author.

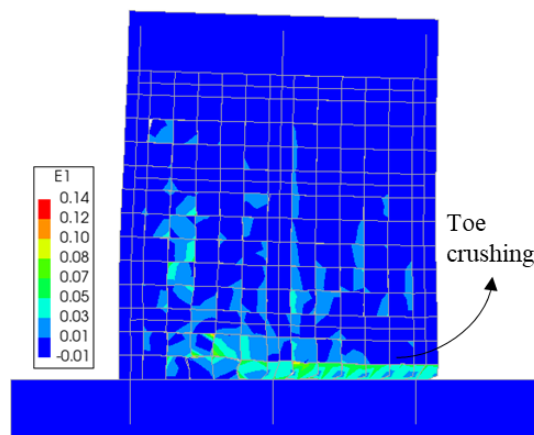
As expected, from Figure 81, it can be concluded that the initial stiffness is controlled exclusively by E , whose value proportionally influenced the stiffness in the response of the model. For example, adopting $E = 300 \cdot f_m$ (i.e., 33.3% of $E = 900 \cdot f_m$) resulted in a stiffness of 38.1% compared to the stiffness obtained with $E = 900 \cdot f_m$. Similarly, considering $E = 600 \cdot f_m$, i.e., 66.7% of $E = 900 \cdot f_m$, the stiffness was 70.5% of that when $E = 900 \cdot f_m$ was adopted. The E value also influenced (with a moderate reduction) the $d_{V_{\max}}$ and d_{V_u} proportionally, while a slight influence in relation to $V_{\max}$ of around 6% was observed.

The compressive parameters f_m and G_c influenced the $V_{\max}$, $d_{V_{\max}}$, and d_{V_u} responses. As seen in Figure 81b, a 30% and 15% reduction of f_m in the UGM and GM regions led to proportional reductions in $V_{\max}$, $d_{V_{\max}}$, and d_{V_u} values. However, a 30% increase in f_m has no significant influence on the overall model response. This asymmetry in the response of the wall

when changing parameters highlights the complex relationship that these parameters develop throughout the nonlinear process.

Regarding G_c , its increase and decrease resulted in a proportional change in ductility, i.e., the higher the G_c , the higher the ductility of the model, and vice versa. The variation of the compressive parameters revealed that the ultimate displacement is determined by the toe crushing of the wall, as seen in the amplified deformed mesh in Figure 82 (for reference model). The toe crushing leads to a significant drop in lateral force, shortly after V_{max} .

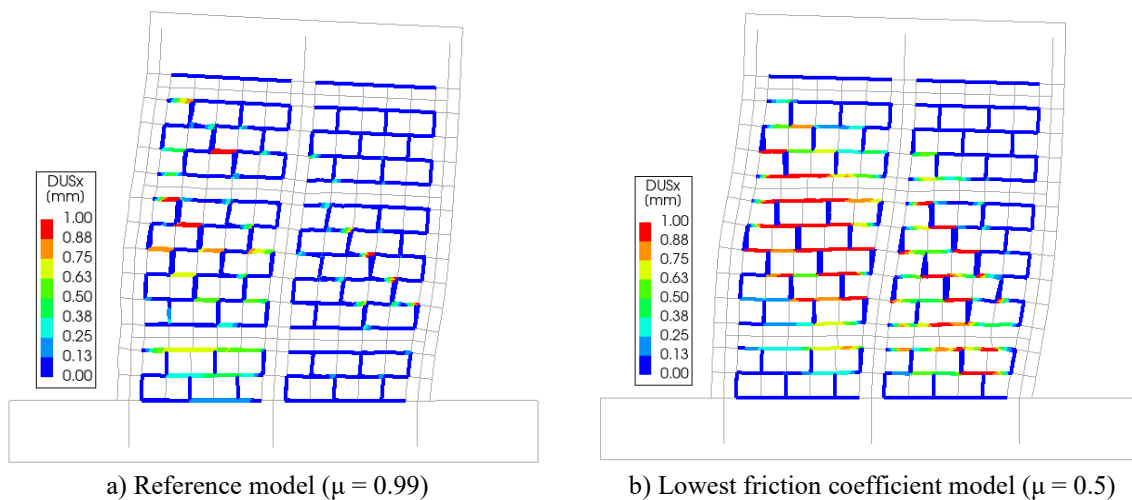
Figure 82: Toe crushing of the PGRM-1.0-NO-3.8 model after V_{max} (for reference model).



Source: Author.

For interfaces, the parameter with the most significant influence on the overall behavior of the model was the friction angle (friction coefficient). The friction angle significantly influenced the post-yield branch of the response, with V_{max} increasing from 300 kN to 400 kN when its value was doubled. The change in d_{vU} was not proportional to the friction coefficient reduction, as a friction coefficient equal to 0.7 led to a lower d_{vU} than the reference model, while a friction coefficient equal to 0.5 led to the development of a plateau after the peak force and, consequently, a higher d_{vU} . The impact of the friction coefficient on interface damage is demonstrated in Figure 83. Sliding displacements are compared for identical wall displacement levels after an extensive plastic stage (10 mm) for both the reference model ($\mu = 0.99$) and the model with the lowest friction coefficient ($\mu = 0.5$). As can be seen, a reduced friction coefficient increases interface damage along the entire wall, thus partially explaining the decreased load-bearing capacity.

Figure 83: Interface damage for different friction coefficient values after an extensive plastic stage (10 mm).



Source: Author.

Within the range established for this study ($\pm 30\%$ around the reference values), negligible changes were observed (not shown here for brevity) when varying the f_t and G_f and the interface parameters c and f_{ti} . The failure mode was insensitive to changes in these values, indicating their limited influence on the overall behavior, most likely due to the reinforcement and the confinement of the unreinforced part by the grouted/reinforced part. These findings highlight the complexity of the aforementioned parameters related to the nonlinear behavior of PGRM walls.

4.6 RECOMMENDATIONS FOR MODELING AND SYNTHESIS FOR INPUT PARAMETERS

The present section provides an overview of recommendations for modeling assumptions and input parameters (including corresponding suggested values, formulas and/or references) required to perform advanced nonlinear analyses of concrete masonry shear walls using the current unified modeling approach.

GM and UGM regions can be suitably represented by employing the TSCM following the concept of the rotating crack model and applying eight-node finite elements (CQ16M). The mesh size is conditioned to the size of each half block so that each block can be divided into two elements. A Coulomb-based constitutive model expanded by considering a brittle gap criterion should be adopted to simulate the unit-mortar interfaces. These considerations are satisfactory to catch the complete force-displacement behavior of the walls, considering the observed experimental failure modes.

Note that mechanical properties are based on the net cross-section area. Therefore, the UGM regions should be represented by a thickness corresponding to the sum of the face shells of the hollow blocks. The net area is equal to the gross area for GM regions; therefore, it is assumed that the thickness of the GM regions could be equal to the wall thickness.

Compressive strength, f_m , for both UGM and GM, is one of the most critical parameters, as it can be closely linked with other properties without more experimental information. Therefore, evaluating this parameter through prism uniaxial compressive tests is essential. A good estimate of the initial stiffness of the model depends mainly on the accurate value of Young's modulus, E . The TMS 402-602 recommendation is suitable for walls without pre-existing cracks.

It is recommended that cohesion and friction angle values are based on experiments, such as the European triplet test (EN 1052-3, 2002), due to the recognized variability of these parameters (Lourenço; Gaetani, 2022). While triplet test results in the literature predominantly focus on clay bricks, recent studies have presented data on these parameters for hollow concrete block masonry (Bui, 2022; Pettit, 2023; Rahim et al., 2023), which can serve as a reference for further applications.

The essential mechanical properties for UGM, GM, and interfaces, as well as their corresponding recommended values, formulas and/or references, are synthesized in Table 12.

Table 12: Essential mechanical properties and suggested values for unified modeling application.

Parameter	UGM	Interfaces	GM
Compressive strength (f_m)	Prism test	-	Prism test
Young's modulus (E)	$E = 600-900 \cdot f_m^{0.1}$	-	$E = 600-900 \cdot f_m^{0.1}$
Poisson's ratio	$0.20-0.25^2$	-	$0.20-0.25^2$
Compressive fracture energy (G_c)	$G_c = 15 + 0.43 \cdot f_m - 0.0036 \cdot f_m^2$	-	$G_c = 1.69 \cdot f_m$
Tensile strength (f_t)	$f_t = 0.33 \cdot \sqrt{f_m}$	-	$f_t = 0.33 \cdot \sqrt{f_m}$
Tensile fracture energy (G_f)	$G_f = 73 \cdot f_m^{0.18}$	-	$G_f = 73 \cdot f_m^{0.18}$
Normal stiffness (k_n)	-	$k_n = \frac{E}{h_{int}}$	-
Shear stiffness (k_t)	-	$k_t = \frac{E}{2 \cdot (1 + \mu) \cdot h_{int}}$	-
Cohesion (c)	-	Triplet test	-
Friction angle (φ)	-	Triplet test	-
Tensile strength (f_{ti})	-	0.10	-

1. The multiplied of f_m is to be selected based on results from tests or from databases relevant to the type of masonry.

2. The value of ν is to be selected based on results from tests. Otherwise, it could be estimated starting from a certain E/G ratio, where G is the shear modulus, as provided in some design standards (Lourenço; Gaetani, 2022) and then, assuming the masonry as isotropic, calculated using the formula $\nu = (E/2G) - 1$.

Source: Author.

Due to the considerable variability in masonry materials and workmanship, these recommendations provide a foundation for understanding the behavior of regular concrete masonry walls. Practitioners must conduct a critical assessment of the input parameters as well as results on a case-by-case basis.

4.7 CONCLUSIONS

A unified modeling approach has been introduced to simulate the in-plane behavior of concrete masonry shear walls. The results with the implemented model were compared with experimental data from different test scenarios, including URM, FGRM, and PGRM walls, for several cases in terms of aspect ratios, openings, axial stress levels and loading protocols.

The developed model allowed capturing the lateral force-displacement response of the experimental walls under different configurations, presenting good accuracy. This achievement is noteworthy, particularly considering the complexity of the involved phenomena and the well-known high variability of masonry specimens. Furthermore, the trends of damage and vertical reinforcement strains for walls exhibiting both predominant failure modes, i.e., shear and flexure, have been accurately simulated.

The observed differences in terms of key response measures ($V_{\max}$, $d_{V\max}$, d_{Vu} , K_{sec} , and dissipated energy) are in line with the experimental and numerical variability reported in the literature. Considering the percentage difference between the predicted response measures by the numerical model and the experimental averages of these measures on all evaluated walls (URM, FGRM, and PGRM), the following values are obtained: 4.1% for $V_{\max}$, -2.2% for $d_{V\max}$, -16.3% for d_{Vu} , -1.0% for K_{sec} , and -19.2% for dissipated energy (area under the curve). Therefore, the numerical modeling approach implemented in this study is suitable for carrying out comprehensive studies on the response of walls similar to those studied here.

A sensitivity analysis was conducted to study the influence of input parameters on a reference PGRM wall model. The findings highlight that the Young's modulus, the properties related to the compressive behavior (f_c and G_c) and the friction angle applied to interfaces are the most influential parameters in the force-displacement response of the evaluated model. Notably, changes when varying f_t , G_f and interface parameters c and f_{ti} were negligible. Given the complex interplay between the input parameters that influence the nonlinear behavior of PGRM walls, different governing parameters other than those presented in this study can be identified based on model-specific characteristics, such as axial load level, aspect ratio, boundary conditions, and reinforcement detailing. A comparison of the presented model with

similar approaches highlighted their main differences. The described approach has a variety of novel and beneficial features in terms of the required mesh size, interface elements, input parameters, and, especially, the number and diversity of validated wall cases. Although comparative analyses on which approach works better in terms of accuracy and computational efficiency are outside the scope of this study, the modeling approach developed against the experimental cases examined here offers an expanded discussion of numerical models for concrete masonry walls and brings reliability to use them in analyses of similar walls. Relevant insights into the modeling approach were achieved, allowing complex phenomena in the tested walls to be simulated.

Recommendations on assumptions and criteria for input parameters were also presented, which is advantageous for future applications. Based on the insights gained, future work is planned to apply this modeling approach in the seismic assessment of concrete masonry structures through pushover analysis of entire building systems.

The current unified modeling approach was validated exclusively through pushover analysis (monotonic loading) of two-dimensional wall structures subjected to only in-plane loads. Further research is necessary to allow extending the application of this modeling approach to cyclic and dynamic loads, and even combined in-plane and out-of-plane loads in three-dimensional structures. Additionally, this study is limited to concrete masonry walls; future research should be developed to evaluate the modeling approach for clay blocks/bricks and stone walls.

5. SEISMIC ASSESSMENT OF PARTIALLY GROUTED AND REINFORCED MULTI-STOREY MASONRY WALLS IN 3D SYSTEMS

5.1 INTRODUCTION

Partially grouted reinforced masonry (PGRM) is an economical alternative to fully grouted reinforced masonry (FGRM), particularly for regions experiencing low to moderate seismic activity. Understanding the behavior of PGRM is crucial for optimizing its application.

The seismic behavior of PGRM shear walls has been studied widely (Bolhassani et al., 2016; Calderón et al., 2020; 2021b; Elmapruk; ElGawady; Hassanli, 2020; Elmeligy; Aly; Galal, 2021; Ghanem et al., 1992; Minaie et al., 2010; Nolph; ElGawady, 2012; Ramírez; Sandoval; Almazán, 2016; Voon; Ingham, 2006; Zhang et al., 2021), highlighting the significant influence of various parameters, including reinforcement, material properties, wall geometry, and the pre-compression level, on the wall response. However, most research has mainly examined single walls with rectangular cross-sections, and there is limited knowledge about the seismic behavior of three-dimensional (3D) PGRM structural systems.

Koutras and Shing (2020) provided valuable insights into the seismic behavior of 3D PGRM structures through shake-table testing and numerical analyses of a single-storey PGRM building. Their study revealed key aspects of the dynamic response and failure mechanisms. However, although it has contributed to understanding the seismic behavior of PGRM buildings, some aspects crucial for real-world applications, such as multi-storey configurations, flange contributions, or different reinforcement detailing schemes, were not comprehensively considered.

Flanged shear walls can occur when a main shear wall intersects with an orthogonal wall, and these walls are connected through interlocking units. A portion of the orthogonal wall serves as a flange, while the main shear wall acts as the web. Design standards, such as CSA S304 (2014), specify the maximum effective flange width that can be considered in the wall design based on factors such as the type of section (T-section or L-section), the actual flange width, flange thickness, and wall height.

The analysis of flanges is crucial, as they often interconnect orthogonal walls in masonry buildings, potentially enhancing the in-plane performance of the entire wall system.

Haach et al. (2013) developed multiple numerical models to investigate the influence of flanges on the behavior of unreinforced masonry (URM) walls under horizontal loading. The results demonstrated that flanges increase the stiffness of the structural system and significantly improve the lateral capacity of masonry buildings. Additionally, flanges play a crucial role in walls that fail under flexure, while their impact on walls failing under shear is less pronounced.

Relevant experimental studies of FGRM walls conducted by Shedid et al. (2010a; 2010b, 2011) show that flanged walls exhibit greater energy dissipation and displacement ductility compared to rectangular walls. The ductility of flanged walls was at least 39% higher than that of rectangular single walls. This increased ductility is crucial for reducing the danger of sudden failure during earthquakes. Furthermore, flanges can limit damage localization, particularly at the toe of the wall, preventing accelerated capacity degradation and improving the overall performance of the structure during and after seismic events.

Another interesting point is the influence of vertical detailing on the seismic performance of PGRM walls. Paulay and Priestley (1992) suggest, from a flexural point of view, that concentrated reinforcements placed only at the toe and heel are more effective than distributed reinforcement for enhancing horizontal load capacity. However, according to Paulay and Priestley (1992), uniform reinforcement distribution can provide greater ductility capacity than concentrated reinforcement. Additionally, it can add the advantage of improving shear performance. In North America, design standards are available for the design of masonry structures, including the TMS 402/602 (2022) and CSA S304 (2024). These standards generally provide guidelines on the material properties, detailing requirements, minimum reinforcements, and other considerations. Concerning detailing, these design standards suggest that grouted/reinforced cells should be uniformly distributed along a shear wall rather than concentrated.

The seismic performance of masonry structures, whether single walls or structural systems like buildings, can be quantified by the seismic performance factors (SPFs): ductility, μ_d ; overstrength, Ω ; ductility-related force modification factor, R_d ; response force modification factor, R ; and displacement amplification factor, C_d . Using appropriate values of the response modification factor, R , it is possible to employ linear elastic analyses for seismic design, taking into account, in a simplified way, the inelastic dissipative capacity of structural systems, as prescribed in design standards (ASCE, 2022; EN 1998-1, 2004; NBCC, 2020; NTC, 2018).

In design standards such as TMS 402/602 (2022) and CSA S304 (2014), the ductility capacity of masonry structures is directly associated with the reinforcement details of the shear walls. TMS 402/602 (2022) classifies shear walls into ordinary, intermediate and special categories, while CSA S304 (2014) categorizes them as conventional, moderate and ductile. Each wall category is assigned an R factor that can be used in linear force-based seismic design. These design standards specify limits for each category's maximum uniform reinforcement spacing and the minimum rebar diameter. However, it appears that, for the sake of simplicity, such parameters are derived from considering isolated walls, disregarding the overall behavior of the building, which would provide a more realistic representation.

In this context, the behavior of PGRM walls within a multi-storey, three-dimensional structural system has therefore been investigated, using a simple archetype as a reference. The effects of flanges, axial load level, number of storeys, and vertical reinforcement distribution have been investigated. A unified modeling approach was employed for pushover analysis, integrating macro-modeling concepts for grouted masonry (GM) regions and simplified micro-modeling for ungrouted masonry (UGM) regions.

Comparative analyses were conducted between configurations with and without the inclusion of orthogonal walls. Two scenarios of axial load and four aspect ratios (corresponding to different number of storeys) were analyzed. The behavior of walls with identical dimensions, reinforced with the same vertical reinforcement ratio, and with the steel concentrated either at both ends of the walls or distributed along the length of the walls was examined.

Nonlinear static (pushover) analyses were conducted until failure, i.e., an ultimate state corresponding to 20% degradation of lateral capacity or the complete loss of load-carrying capacity. PGRM wall structural systems were studied through qualitative and quantitative analyses considering pushover-based seismic response parameters, with a focus on the response force modification factor, R . Eigenanalyses were also performed, the results of which are compared with estimates using analytical equations provided by ASCE SEI 7 (2022), NBCCC (2020) and Eurocode 8 Part 1 (2004) to estimate the fundamental period, T , of the masonry buildings.

5.2 FRAMEWORK FOR SEISMIC PERFORMANCE EVALUATION

This section first presents new validation cases of the finite element-based micro-modeling approach adopted in this study by comparing the numerical lateral load response with

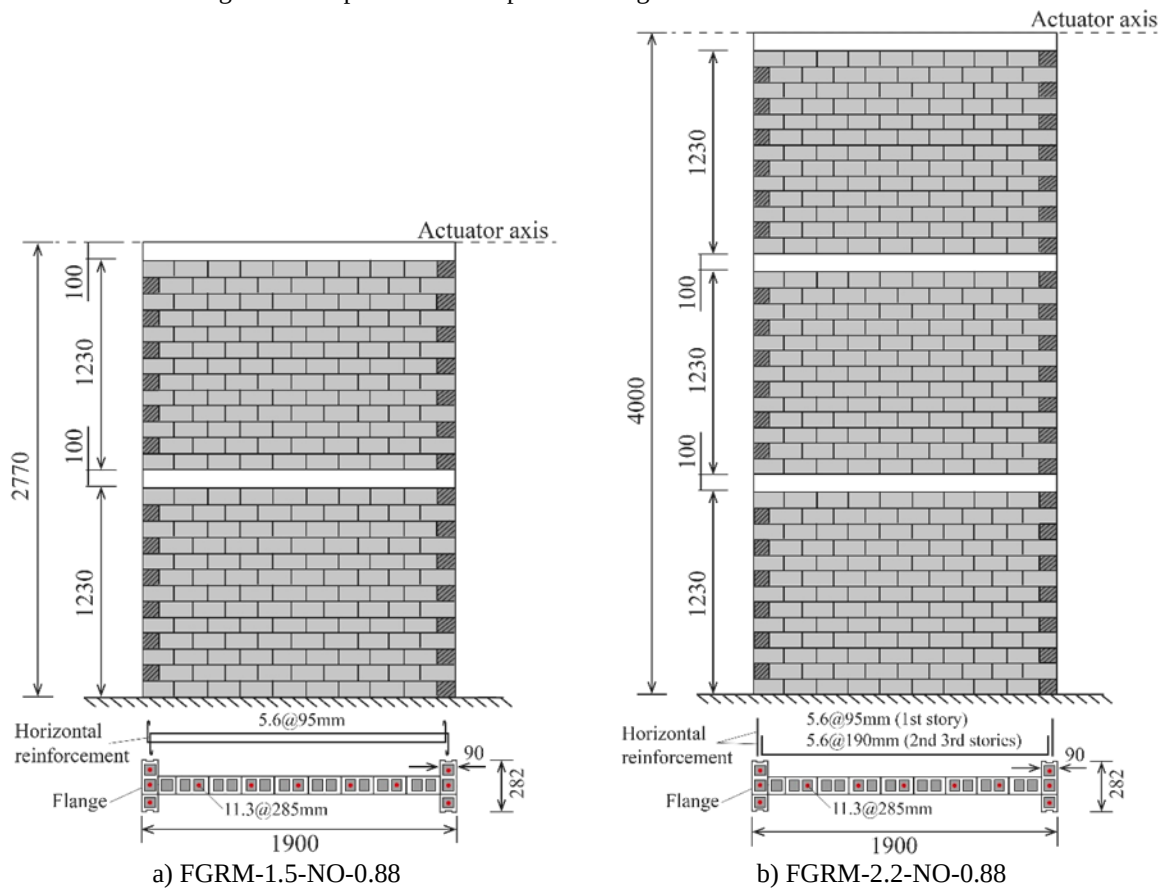
experimental data for two multi-storey flanged masonry walls. Subsequently, the characteristics of the PGRM structural system are detailed, including its geometry, reinforcement detailing, and material properties, along with the corresponding numerical models, highlighting key assumptions and simplifications.

5.2.1 Numerical model validation for flanged walls

A finite element-based micro-modeling approach previously conceived and validated by Palhares et al. (2024) is extended here for flanged walls using a three-dimensional model. Eight-node quadrilateral isoparametric curved shell elements with five degrees of freedom (DOF) per node and a total of 40 DOF per element (CQ40S – DIANA (DIANA FEA, 2022)) were adopted. The default DIANA configuration, employing a regular 2×2 in-plane Gauss integration scheme, was applied to these finite elements. Shear deformation is considered according to the Mindlin-Reissner theory, while the normal stress component perpendicular to the surface element is assumed to be zero.

The validation is carried out by evaluating the lateral response of two half-scale FGRM flanged walls tested by Shedid et al. (2010b). These walls were designed to behave mainly in flexure and represent two and three-storey structures, with aspect ratios of 1.5 and 2.2, respectively, as depicted in Figure 84. They were subjected to cyclic lateral loading with displacement control and an axial stress of approximately 0.88 MPa. The vertical and horizontal reinforcement detailing is shown in Figure 84. These flanged walls are identified here according to the nomenclature defined by Palhares et al. (2024) for other validated experimental walls, which indicates the wall type (FGRM) - the aspect ratio (height to length ratio) - the indication of openings (NO: no opening) - and the axial load level in MPa.

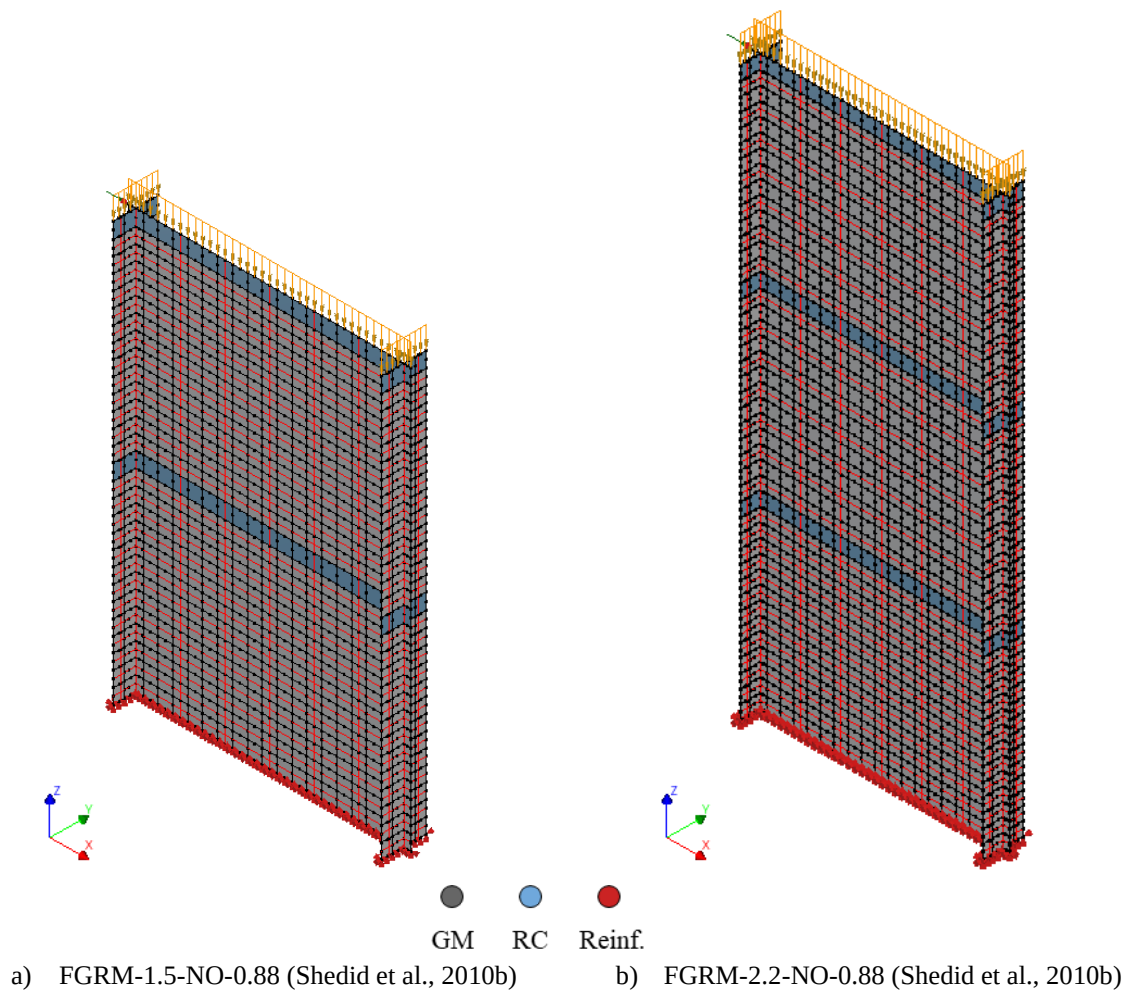
Figure 84: Experimental setup of the flanged walls. Dimensions in mm.



Source: Adapted from (Shedid et al., 2010b).

The configurations of the numerical models are illustrated in Figure 85. Each grouted masonry (GM) region has a thickness of 90 mm, equivalent to the total half-scale block thickness, with finite elements sized at $95 \times 95 \text{ mm}^2$. The nodes at the base of the model were constrained horizontally and vertically. Self-weight, based on a masonry mass density of 2400 kg/m^3 , was incorporated into the structure, along with a constant distributed vertical load on the top, representing an applied axial stress of approximately 0.88 MPa. Horizontal displacement at the top was incrementally applied in constant increments of 0.25 mm.

Figure 85: Numerical model of the flanged walls.



Source: Author.

The material properties of the flanged wall models are provided in Table 13. The compressive strength, f_m , was defined based on the average results of tests conducted on GM prisms by Shedid et al. (2010b). Young's modulus, E , was calibrated at $600 \cdot f_m$, to match the initial stiffness in the responses. The tensile strength, f_t , was estimated according to Equation 12, proposed by Wong et al. (2013), as previously used in the numerical models by Medeiros et al. (2022a) and Rahim (2023). The tensile fracture energy, G_f , was calculated using Equation 13, as outlined in the FIB Model Code (2013) for concrete-like materials. The compressive fracture energy, G_c , for GM was estimated to be $1.69 \cdot f_m$. This last formula, related to the definition of an average ductility index in compression (ratio between fracture energy and strength), is based on experimental data reported by Nakamura and Higai (1999) for cylindrical concrete specimens and was previously employed by Calderón et al. (2021d).

$$f_t = 0.33 \cdot \sqrt{f_m} \quad \text{Eq. 12}$$

$$G_f = 73 \cdot f_m^{0.18} \quad \text{Eq. 13}$$

Following the same approach used by Palhares et al. (2024), the anisotropy and plane of weakness introduced by mortar joints are neglected for GM regions. For the steel reinforcements, the yield strength, f_y , and Young's modulus, E , values were adopted from the experimental program (Shedid et al., 2010b). The concrete material of slabs was modeled as a linear elastic material with a Young's modulus of 33.6 GPa and a Poisson's ratio of 0.15.

Table 13: Material properties adopted for the flanged wall models.

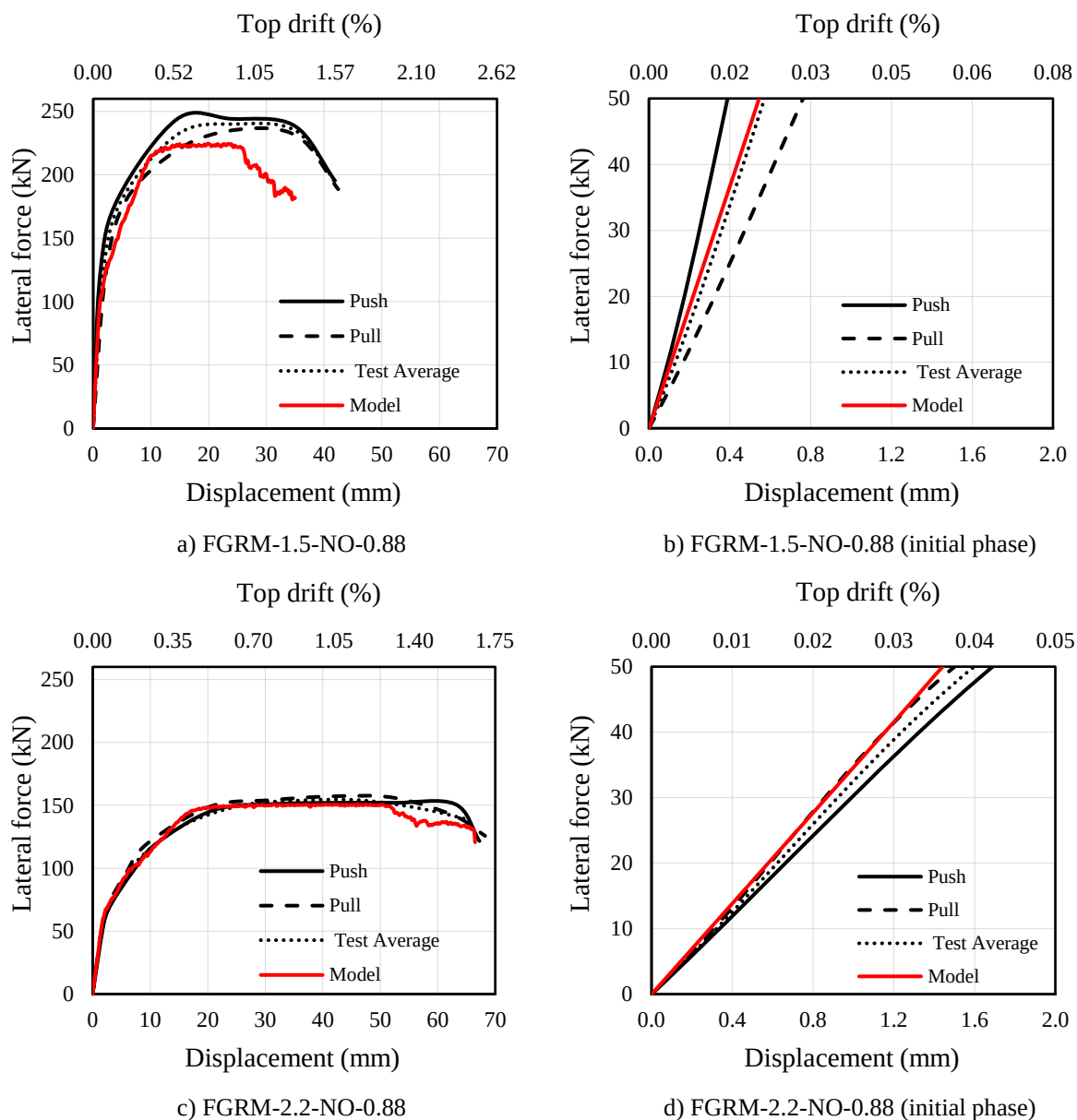
Element	Young's modulus (E) N/mm ²	Compressive strength (f_m) N/mm ²	Compressive fracture energy (G_c) N/mm	Tensile strength (f_t) N/mm ²	Tensile fracture energy (G_f) N/mm	Yield strength (f_y) N/mm ²
FGRM-1.5-NO-1.1 and FGRM-2.2-NO-1.1 (Shedid et al., 2010b)						
GM	9840	16.4	27.72	1.34	0.121	-
Ver. Reinf.	200600			-		534
Hor. Reinf.	200600			-		495

Source: Author.

The accuracy of the numerical models was evaluated by comparing the monotonic numerical responses with the experimental envelopes of the hysteretic responses in the push and pull directions. The idea is that the model remains simple and computationally efficient while maintaining sufficient accuracy to analyze entire building structures. It is worth noting that for the TSCM and stress-strain relationship adopted in DIANA, secant unloading, and reloading behavior is considered, which may not accurately capture cyclic energy dissipation in masonry structures, leading to a significant underestimation of hysteretic damping effects (Fusco et al., 2022; Lourenço; Gaetani, 2022; Schreppers et al., 2017; Sousamli; Messali; Rots, 2022). Nevertheless, previous studies (Calderón et al., 2021; Calderón; Arnau; Sandoval, 2019b; Di Lallo et al., 2022, 2023; Lourenço et al., 2007; Magenes; Calvi, 1997; Maleki, M., El-Damatty, A. A., Hamid, A. A., Drysdale, 2005; Palhares et al., 2023; Parisse et al., 2022; Pettit, 2023; Quinteros; Oller; Nallim, 2012; Rahim et al., 2023; Shing; Cao, 1997; Silva; Lourenço; Milani, 2018) have shown that monotonic numerical responses can offer a reliable approximation of the cyclic response of masonry shear walls regarding the initial stiffness, the peak load and the displacement response. In this study, a simplified modeling approach is adopted with the aim of simulating the overall behavior of PGRM buildings. The main objective is to analyze the nonlinear response of building systems, for which pushover analysis is a more robust and computationally efficient approach.

The comparison of the force-displacement curves is presented in Figure 86. The complete responses are presented on the left, and a zoomed-in view of the initial response on the right. The solid and dashed lines represent the envelope of the cyclic response in the push and pull directions of the experimental walls, respectively. The dashed line corresponds to the test average curve. For visualization purposes, the push and pull curves are presented on the same directional axis, allowing for a clearer comparison of their backbone envelopes.

Figure 86: Force-displacement curves of the flanged walls.



Source: Author.

Key response measures were considered to evaluate the model accuracy: the maximum lateral force, $V_{\max}$, the displacement at the maximum lateral force, $d_{V\max}$, the ultimate

displacement, d_{Vu} , the secant stiffness, K_{sec} , and the dissipated energy, U . The d_{Vu} was defined as that corresponding to a 20% reduction in V_{max} . The K_{sec} was calculated over a range of 5% to 33% of V_{max} , following a common procedure in design standards (e.g., TMS 402/602 (2022) and CSA S304 (2014)).

The numerical response is similar to that obtained experimentally, from the initial phase up to the post-peak load. The numerical model satisfactorily captured the wall aspect ratio influence on V_{max} , d_{Vmax} , and K_{sec} . In both the numerical simulations and experimental tests, an increase in the aspect ratio (achieved by adding another storey) resulted in reduction of V_{max} and K_{sec} while d_{Vmax} increased, demonstrating consistent trends between experimental and numerical results.

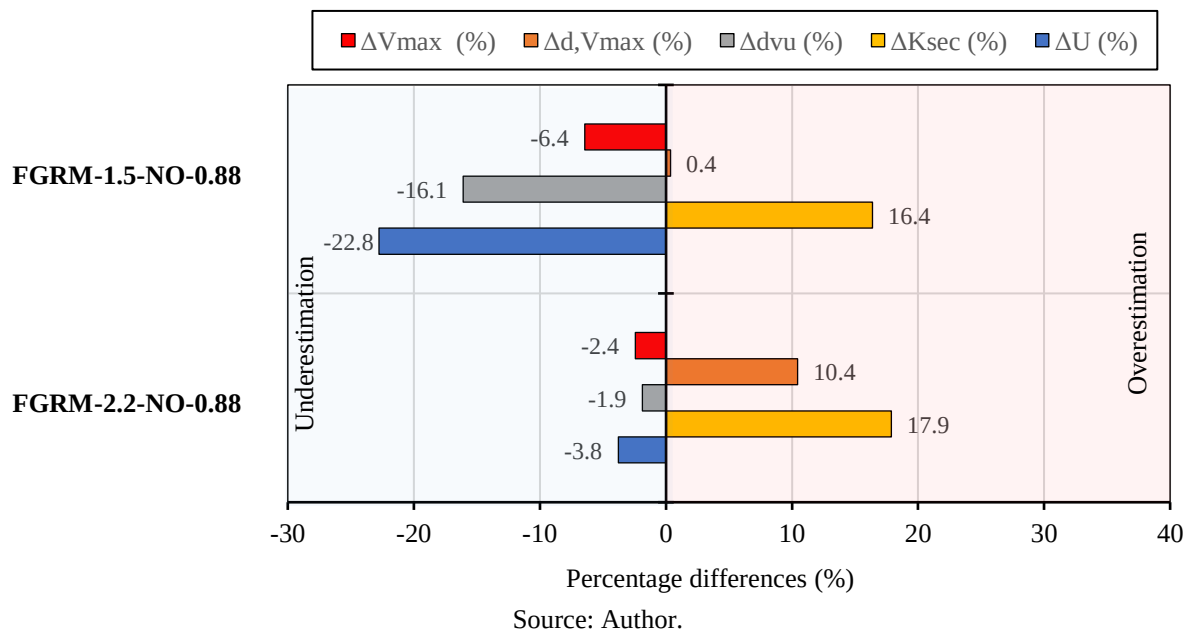
The force–displacement curves are quantitatively summarized in Table 14. Additionally, the percentage differences between the model results and the experimental pull and push average responses are revealed in Figure 87.

Table 14: Summary of key response measures in the force-displacement curves for the flanged walls.

Response	V_{max} (kN)	d_{Vmax} (mm)	d_{Vu} (mm)	K_{sec} (kN/mm)	U (kJ)
FGRM-1.5-NO-0.88					
Push	244.2	14.6	41.9	98.8	9177.3
Pull	235.6	25.2	42.5	56.5	8713.1
Test average	239.9	19.9	42.2	77.7	8945.2
Model	224.5	20.0	35.4	90.4	7009.0
FGRM-2.2-NO-0.88					
Push	152.0	41.7	67.2	28.8	9132.9
Pull	157.0	41.6	68.2	29.9	9437.2
Test average	154.5	41.7	67.7	29.4	9285.0
Model	150.7	46.0	66.5	34.6	8933.7

Source: Author.

Figure 87: Percentage differences between the model and average test measures.



The numerical model accurately estimated the lateral force resistance, with percentage differences of -2.4% and -6.4% for the push-pull average response of FGRM-1.5-NO-0.88 and FGRM-2.2-NO-0.88, respectively. The displacement $d_{V\max}$ was well predicted, with the maximum percentage difference of 10.4%. For the less ductile wall (FGRM-1.5-NO-0.88), the model provided conservative energy dissipation, U , and ultimate displacement, d_{vu} , estimates, predicting lower values than observed experimentally (-16.1% for d_{vu} and -22.8% for U). Up to the peak load, the difference in energy dissipation remains relatively low (approximately 7.0%). However, in the post-peak regime, the discrepancy increases as the experimental curves exhibit a more ductile response, while the numerical model underestimates deformation capacity. Indeed, post-peak simulation is particularly challenging due to the strong nonlinear effects related to damage mechanisms that can occur. However, minimal differences were observed for the three-storey wall case (FGRM-2.2-NO-0.88), with percentage differences of -1.9% and -3.8% for d_{vu} and U , respectively.

The model overvalued the secant stiffness, $K_{\sec}$, with percentage differences of 16.4% and 17.9% for the average response of FGRM-1.5-NO-0.88 and FGRM-2.2-NO-0.88, respectively. Note that the difference for $K_{\sec}$ was even higher between the experimental push and pull directions of the first wall. The above differences (within 20%) can be considered acceptable for engineering purposes, but, qualitatively, from Figure 86b and Figure 86d, it is evident that the numerical response closely follows the experimental force-displacement curves

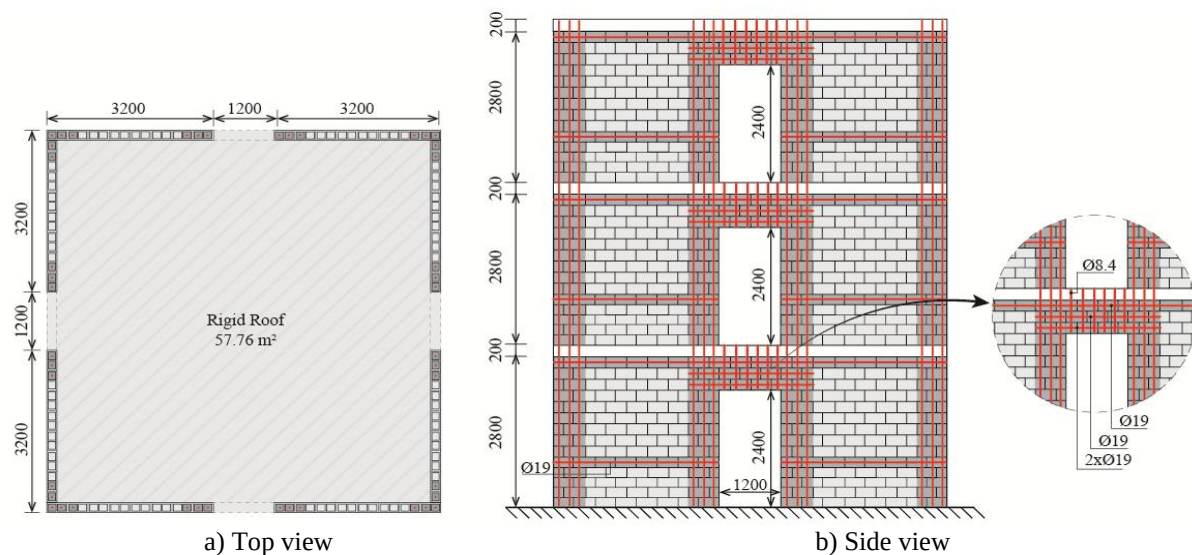
during the initial phase, confirming good agreement with the initial stiffness, which further strengthens the validation of the numerical model.

5.2.2 Considered PGRM wall structural systems

5.2.2.1 Characteristics of modeled structures

To enhance comprehension of PGRM behavior, this study extends previous experimental research conducted by Fortes et al. (2016) and numerical evaluations by Medeiros et al. (2022a, 2023) and Palhares et al. (2023, 2024) into a three-dimensional structure simulation framework. The reinforcement detailing follows typical Brazilian construction practices, where grout and reinforcement are concentrated at the ends of the walls to enhance flexural stiffness. Unlike previous studies for individual walls, a three-dimensional assembly of four orthogonally interconnected walls was examined under a rigid diaphragm assumption. By investigating this approach level, this study provides new insights that can contribute to broader discussions about masonry design and construction worldwide. Models are full-scale to better reflect real-world structures and avoid scale effects in scaled-down specimens. The geometry of the structural system is depicted in Figure 88.

Figure 88: Configuration of the PGRM structural systems with reinforcement concentrated at the ends of the walls.



Source: Author.

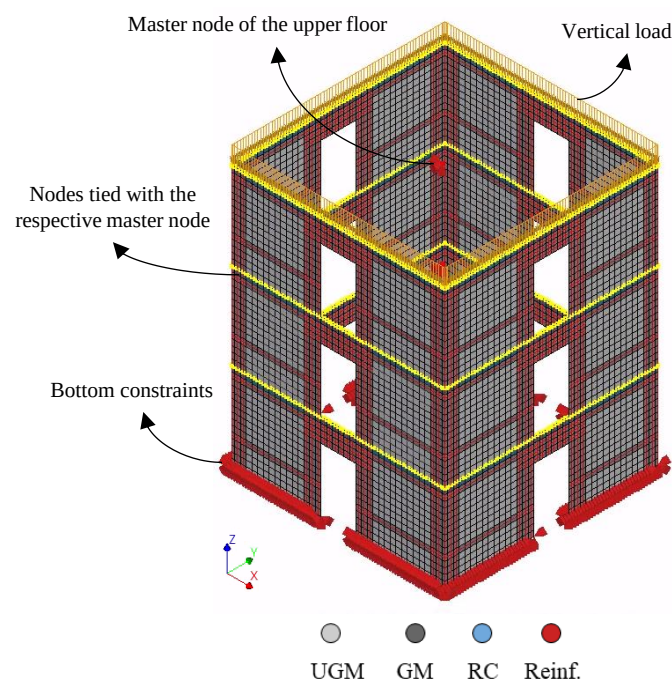
The walls are each 7.6 m long (equivalent to 18.5 blocks), 3.0 m high (spanning 14 masonry courses plus a 200mm slab) and feature a central door opening. The total base area is 57.8 m². Both the vertical and horizontal reinforcement are composed of 19 mm diameter steel

bars. Stirrups, with a diameter of 8.4 mm, connect the courses above the openings with the RC slab wing, forming the masonry beam.

5.2.2.2 Description of the numerical modeling

The unified modeling approach previously conceived by Palhares et al. (2024) and validated here for flanged walls was employed. The numerical model was developed using the finite element analysis software DIANA[®] (DIANA FEA, 2022) and the meshed model of a three-storey building is depicted in Figure 89.

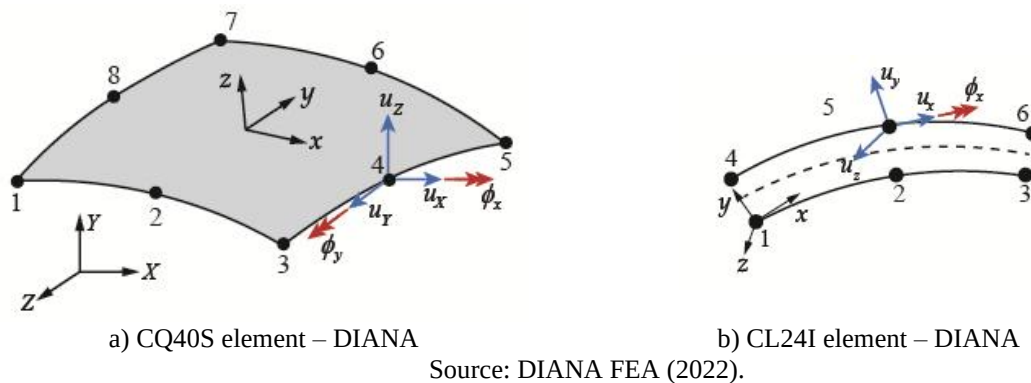
Figure 89: Meshed numerical model of the three-storey building.



Source: Author.

For the ungrouted masonry (UGM) and grouted masonry (GM) regions, the CQ40S elements were used. The mortar joints in UGM regions were modeled considering the interface elements (CL24I – DIANA (DIANA FEA, 2022)). This interface element employs quadratic interpolation. The used finite elements are illustrated in Figure 90.

Figure 90: Elements adopted in the numerical modeling (DIANA FEA, 2022).



The mesh was set to match the half-block size, with each cell of the blocks represented by shell elements having $200 \times 200 \text{ mm}^2$. The elements in the model represent the net effective cross-section of the corresponding regions of the wall. For the GM regions, a thickness of 200 mm was assigned, while for the UGM regions, a thickness of 70 mm was used, equivalent to the thickness of the face shells added together. After meshing, the final 3D numerical model consists of 2841 elements and 9585 nodes per storey. To reduce the computational cost, interface elements are considered solely on the walls in the loading direction, along which displacements are imposed.

A master node was set at the centroid of each floor at the slab level. These master nodes were then tied to the top nodes of the walls at the corresponding floor level, ensuring that they had the same displacement in the XY plane to simulate the rigid diaphragm effect of the slabs. For the eigenvalue analysis, the mass of each storey was concentrated at its respective master node. The nodes at the base were constrained horizontally and vertically, disregarding any influence from soil-structure interaction.

The material properties and references from where they are adopted, including UGM, GM, interfaces and reinforcement, used for the numerical analysis of PGRM wall structural systems, are outlined in Table 15. The compressive strength, f_m , of both UGM and GM was specified as 8.0 MPa derived from a simple predictive rule, which estimates 1.6 MPa per storey in the net cross-sectional area, with a minimum of 8.0 MPa. The Young's modulus, E , tensile strength, f_t , and fracture energies in compression, G_c , and tension, G_f , were calculated as functions of the compressive strength, according to the same strategy by Palhares et al. (2024). The Total Strain Crack Model (TSCM) in DIANA (DIANA FEA, 2022) is adopted and a compressive strength reduction model due to lateral cracking is applied according to the Vecchio and Collins 1993 model (Vecchio; Collins, 1993), with a lower bound reduction curve

set at 0.6. Cohesion (c) and friction angle (ϕ) for the interfaces were defined based on shear triplet test results (Pettit, 2023; Rahim, 2023), and a tensile strength of interfaces (f_{ti}) equal to 0.10 MPa is assumed in all models. The normal and shear stiffness components were calculated based on the elastic properties of UGM and a mortar joint height, h_{int} , was assumed equal to 10 mm. A Poisson's ratio (ν) of 0.23 was adopted based on experimental tests on hollow concrete blocks (Oan, 2013). For steel reinforcement, Young's modulus (E), yield strength (f_y) and ultimate strength (f_u) values were adopted from the experimental program by Fortes et al. (2016).

Table 15: Material properties adopted for PGRM structural systems.

Parameter	Material					Reference for the adopted value
	Masonry			Reinforcement		
	UGM	GM	Interfaces	Long. rebars	Stirrups	
Compressive strength (f_m) N/mm ²	8.0	8.0	—	—	—	Specified
Young's modulus (E) N/mm ²	7200	7200	—	20351 2	222799	$E = 900 \cdot f_m$ (Masonry) (TMS, 2022) Test (Reinforcement) (Fortes et al., 2016)
Compressive fracture energy (G_c) N/mm	18.2	13.5	—	—	—	$G_c = 15 + 0.43 \cdot f_m - 0.0036 \cdot f_m^2$ (UGM) (Magenes et al., 2008) $G_c = 1.69 \cdot f_m$ (GM) (Calderón et al., 2021d)
Tensile strength (f_t) N/mm ²	0.93	0.93	—	—	—	$f_t = 0.33 \cdot \sqrt{f_m}$ (Wong et al., 2013)
Tensile fracture energy (G_f) N/mm	0.10 6	0.106	—	—	—	$G_f = 73 \cdot f_m^{0.18}$ (fib Model Code 2010, 2013)
Normal stiffness (k_{nn}) N/mm ²	—	—	720.0	—	—	$k_n = \frac{E}{h_{int}}$ (Arnau et al., 2015; Calderón et al., 2021d)
Shear stiffness (k_{tt}) N/mm ²	—	—	257.1	—	—	$k_t = \frac{E}{2 \cdot (1 + \mu) \cdot h_{int}}$ (Arnau et al., 2015; Calderón et al., 2021d)
Cohesion (c) N/mm ²	—	—	0.15	—	—	Test (Pettit, 2023; Rahim, 2023)
Friction angle (φ) deg.	—	—	44.70	—	—	Test (Pettit, 2023; Rahim, 2023)
Tensile strength (f_{ti}) N/mm ²	—	—	0.10	—	—	Recommendation (Lourenço; Gaetani, 2022)
Yield strength (f_y) N/mm ²	—	—	—	540	743	Test (Fortes et al., 2016)
Ultimate strength (f_u) N/mm ²	—	—	—	742	812	Test (Fortes et al., 2016)

Source: Author.

5.3 NUMERICAL ANALYSIS

This section presents the numerical analyses of the multi-storey PGRM structural systems. First, eigenanalyses are conducted to determine the fundamental period and assess its agreement with analytical formulations from design standards. Next, pushover analyses are performed to investigate the nonlinear lateral response of the structural systems. Finally, the influence of key parameters on structural performance — including the effects of flanges, axial load, aspect ratio, and vertical reinforcement distribution — is evaluated.

5.3.1 Eigenanalyses

Before conducting the pushover analyses, eigenanalysis (modal analysis) was performed to compute the global translational vibration modes and the respective fundamental period, T . T is an essential parameter for seismic analysis, providing insights into the structure's dynamic characteristics, and also having great influence on performance-based seismic assessment, particularly using nonlinear static procedures. The numerical model was specifically developed to consider only the global translational mode by concentrating weight at the center of mass of each floor. This approach is consistent with the lumped-mass shear building idealization described by Chopra (2012), in which the mass of multistorey buildings is assumed to be concentrated at floor levels due to the constraining effects of rigid diaphragms. This assumption is fundamental in seismic design, as the global translational mode primarily governs the lateral force distribution in regular buildings and serves as the basis for equivalent static force procedures and response spectrum analysis.

The analyses were performed for the masonry structural system, varying the number of storeys, ranging from 1 to 10. The results were compared with the estimates by analytical equations from the design standards ASCE SEI 7 (2022), NBCCC (2020), and Eurocode 8 (2004), presented in Equations 14, 16, and 17 respectively, developed to predict the fundamental period of masonry-walled buildings. Jacobs (2008) discusses the limitations of standard-based empirical formulas, emphasizing that they are often derived from regression analyses of recorded building data rather than fundamental mechanics. Besides, these equations are intentionally formulated to provide a conservative (short) estimation of the fundamental building period. While seismic standards provide empirical equations to estimate the fundamental period for various structural types, few studies have critically assessed their accuracy, particularly for concrete masonry buildings. As a result, their reliability in predicting the period of masonry structures remains uncertain and warrants further investigation.

$$T = \frac{C_q}{\sqrt{C_w}} \cdot h \quad \text{Eq. 14}$$

where h is the height (m) of the structure above the base, $C_q = 0.006233$ m, and C_w is calculated from Equation 15.

$$C_w = \frac{100}{A_B} \sum_{i=1}^x \frac{A_i}{\left[1 + 0.83 \left(\frac{h}{l_{wi}}\right)^2\right]} \quad \text{Eq. 15}$$

where A_B is the area of the base of the structure (m²); A_i is the web area of the shear wall i (m²); l_{wi} is the length of the shear wall i (m); x is the number of shear walls in the building effective in resisting lateral forces in the direction under consideration.

$$T = 0.05 \cdot h \quad \text{Eq. 16}$$

$$T = C_t \cdot h^{3/4} \quad \text{Eq. 17}$$

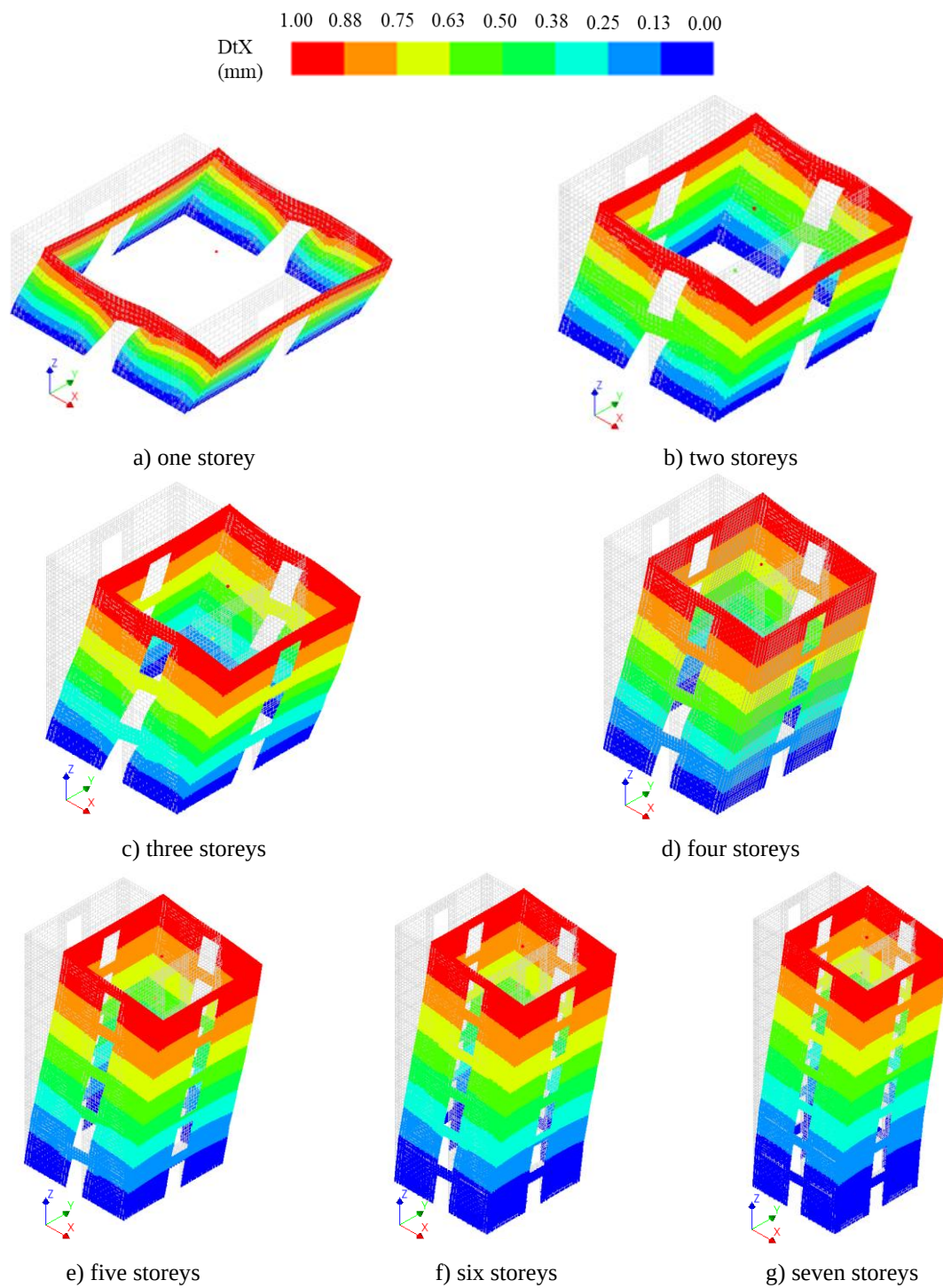
where $C_t = 0.075/\sqrt{A_c}$ and A_c is calculated from Equation 18.

$$A_c = \sum \left[A_i \cdot \left(0.2 + \frac{l_{wi}}{h}\right)^2 \right] \quad \text{Eq. 18}$$

where A_c is the total effective area of the shear walls in the first storey of the building (m²). For Eq. 18, l_{wi}/h should not exceed 0.9.

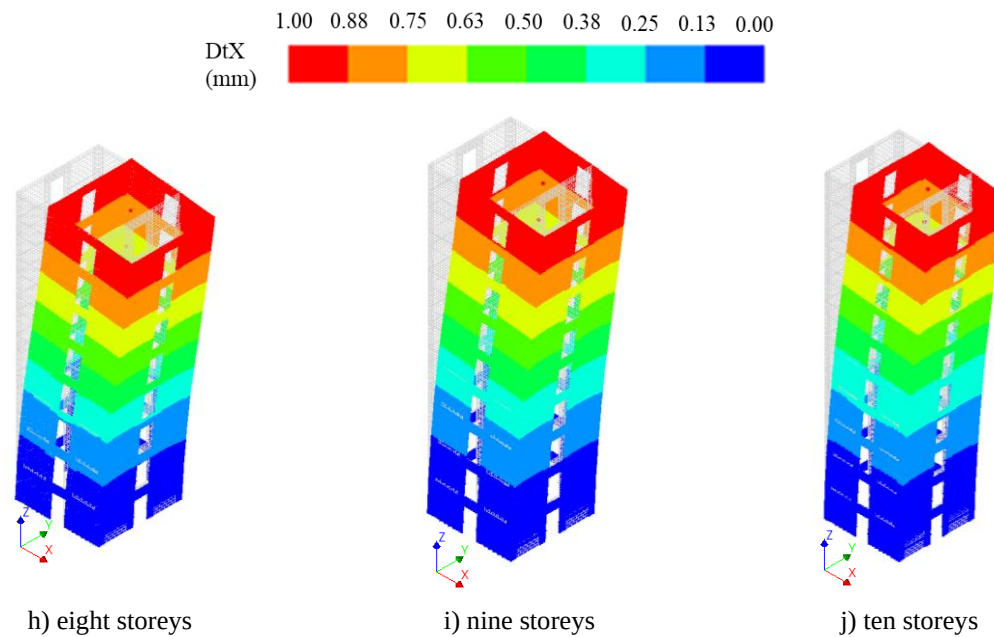
To determine the numerical vibration modes, a modeling strategy that concentrates each storey's mass on its respective master node was defined. The mass calculation was derived from the self-weight of the walls and slab panels, resulting in a mass of 54200 kg (54.2 tons) per floor. The eigenmodes and corresponding eigenfrequencies obtained from the analysis are illustrated in Figure 91. A comparison between the fundamental period, T , calculated from eigenanalysis and that predicted by the formulas in design standards is presented in Figure 92 and summarized in Table 16.

Figure 91: Global translation modes in X direction.

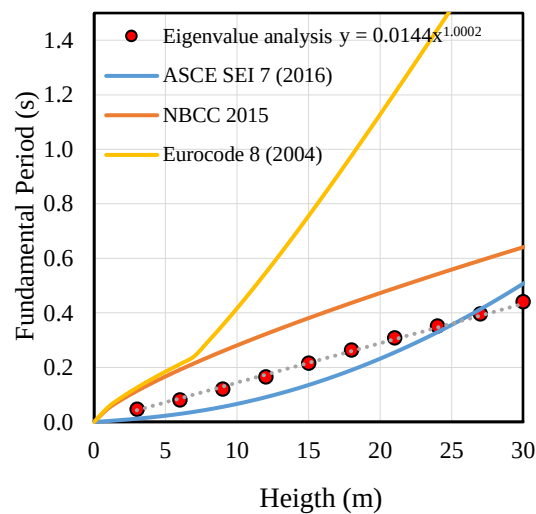


Source: Author.

Figure 90: Global translation modes in X direction (continued).



Source: Author.

Figure 92: Comparison of the fundamental period (T) from eigenanalysis and formulas in design standards.

Source: Author.

Table 16: Summary of comparison of estimated fundamental period (T) values using different methods.

Method	Height (m)	No. of storeys	T (s)	ΔT (%)
Eigenvalue analysis	3	1	0.047	—
NBCC (2020)			0.114	140.9
Eurocode 8			0.126	166.4
ASCE SEI 7 (2022)			0.012	-74.1
Eigenvalue analysis	6	2	0.082	—
NBCC (2020)			0.192	134.6
Eurocode 8			0.212	159.4
ASCE SEI 7 (2022)			0.030	-63.4
Eigenvalue analysis	9	3	0.121	—
NBCC (2020)			0.260	115.0
Eurocode 8			0.356	194.2
ASCE SEI 7 (2022)			0.056	-53.8
Eigenvalue analysis	12	4	0.165	—
NBCC (2020)			0.322	95.0
Eurocode 8			0.547	231.0
ASCE SEI 7 (2022)			0.091	-45.0
Eigenvalue analysis	15	5	0.216	—
NBCC (2020)			0.381	76.5
Eurocode 8			0.756	250.1
ASCE SEI 7 (2022)			0.136	-37.1
Eigenvalue analysis	18	6	0.265	—
NBCC (2020)			0.437	65.0
Eurocode 8			0.976	268.6
ASCE SEI 7 (2022)			0.190	-28.1
Eigenvalue analysis	21	7	0.309	—
NBCC (2020)			0.490	58.6
Eurocode 8			1.205	289.6
ASCE SEI 7 (2022)			0.255	-17.6
Eigenvalue analysis	24	8	0.353	—
NBCC (2020)			0.542	53.8
Eurocode 8			1.439	308.2
ASCE SEI 7 (2022)			0.329	-6.6
Eigenvalue analysis	27	9	0.396	—
NBCC (2020)			0.592	49.4
Eurocode 8			1.677	323.0
ASCE SEI 7 (2022)			0.413	4.3
Eigenvalue analysis	30	10	0.442	—
NBCC (2020)			0.641	45.0
Eurocode 8			1.918	333.9
ASCE SEI 7 (2022)			0.507	14.8

Source: Author.

In agreement, all approaches for calculating T indicate that the taller the building, the greater the natural period. This trend confirms what was expected since buildings reduce their initial stiffness with increasing height. The eigenanalysis results exhibit a linear relationship with building height, as shown by the fitted trend line equation $y = 0.0144x$, which provides an empirical fit close to the computed eigenvalue periods. This suggests that the fundamental period can be approximated as a direct function of height for the structural configurations analyzed in this study.

The period values estimated by the equation from ASCE SEI 7 (2022) showed the closest agreement with those predicted by eigenanalysis. For example, there was a percentage difference of -74.1% for the case of one storey (three meters in height), which reduced to 14.8% for the structure with ten storeys.

The equation from NBCC (2020) estimates T according to a similar trend to that of eigenanalysis, although with higher values. The percentage difference decreases as the number of storeys increases, ranging from 140.9% for one storey to 45.0% for ten floors. The Eurocode 8 Part 1 (2004) prediction equation initially shows results that were very close to those from the NBCC formula, up to two storeys (six meters high). However, it then tended to estimate considerably higher values beyond this height than other equations and eigenanalysis. This discrepancy is due to the low l_{wi}/h ratio, initially limited to 0.9 for heights up to six meters. Beyond six meters, the l_{wi}/h ratio values decrease, causing an increase in the estimated value of T , resulting in a significant deviation from the eigenvalue analyses and the predictions of other design standards.

Given the discrepancies observed for T , an alternative empirical formula for masonry structures could be considered, for example, the linear relationship found in this study between eigenanalysis results and height ($T = 0.0144H$). It is important to mention that the equations provided in design standards are formulated for actual buildings, while the structural system evaluated here represents a simplified idealization. Therefore, generalized conclusions based solely on these results should not be drawn.

5.3.2 Pushover analyses of the three-storey structure

5.3.2.1 Convergence criteria

The numerical analyses presented here were performed employing the solution methods outlined by Palhares et al. (2024). The Secant (Quasi-Newton-Raphson) iterative method with the Broyden-inverse updated method (Broyden, 1970) was adopted with a sparse Cholesky-based direct solver. It should be noted that finite element models on this scale (three-dimensional structural systems) entail significant computational demands and are challenging in achieving numerical convergence. Therefore, it is crucial to evaluate numerical strategies for efficient, full-regime nonlinear analysis.

To ensure computational efficiency while maintaining numerical accuracy, two convergence criteria were considered: one based on an energy norm tolerance of 0.01% and another with a less restrictive energy norm tolerance of 0.1%. A maximum of 100 iterations was also set as a secondary convergence criterion, ensuring that excessive computation time was avoided.

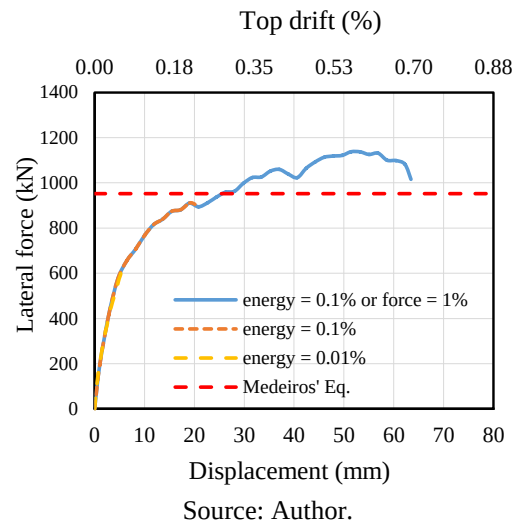
Subsequently, two strategies to define the load step were tested. The first strategy involved an automatic reduction of step size already implemented in DIANA (DIANA FEA, 2022), starting with a displacement of 0.3 mm. Then, if convergence was not reached, the step size was halved, and the solution was tried again. If convergence was still not achieved, the step size was halved again. The numerical process was stopped if convergence was not reached within 100 iterations after five consecutive attempts.

For the second strategy, the convergence criterion was defined based on a report by Hendriks et al. (2017), also adopted by Mahmoudimotlagh (2020), where load increments are converged if at least one of the force or energy norms is satisfied within the maximum number of 100 iterations. Convergence tolerances for energy and force norms are 0.1% and 1%, respectively. Load increments that did not fully meet this criterion may still be deemed acceptable if the next step achieves convergence. The process allows computation to progress even if a particular step fails to converge within 100 iterations despite local failures.

The analyses were performed by applying displacements of 0.30 mm on the top controller node and constant gravity loads. The axial load was distributed at the upper nodes of the wall, and a masonry mass density of 2250 kg/m³ was adopted to consider for the self-weight, totaling an axial load level of 4% of the masonry compressive strength, f_m . The force-

displacement responses obtained are shown in Figure 93. The top drift (displacement/height ratio) values relative to the base of the wall are also indicated on the upper horizontal axis.

Figure 93: Force-displacement responses of the 3-storey building for different numerical strategies and convergence criteria



Regarding the first strategy, the analyses find convergence problems from early load steps for both convergence tolerances. When the energy norm was relaxed to 0.1%, the model response demonstrates some progression but encounters numerical convergence issues before the post-peak phase. Since the goal is to proceed with a nonlinear analysis until failure, this strategy does not work well. In contrast, the second strategy proves to be satisfactory. This approach effectively allows capturing the complete nonlinear behavior, including the post-peak phase.

To increase confidence of the numerical response, the magnitude of the peak load has also been evaluated through Equation 19, proposed by Medeiros et al. (2022a). This equation considers the contributions of masonry resistance, aspect ratio, vertical and horizontal grouting and reinforcement, and axial loading on the wall shear load capacity. Here, the application of the equation was adapted for cases with openings, assuming an effective cross-sectional area of the wall web subtracting the opening's cross-section. The aspect ratio was calculated assuming an effective height equal to the height of the opening, and the total wall length. Finally, the shear load capacity calculated for one wall was multiplied by two to consider for the contribution of the two walls of the building system, which resist lateral loads together.

The results indicate that Equation 19 reasonably estimates peak lateral resistance. However, it does not take into account variations in axial load between individual piers as lateral loading is applied. Since axial force variation influences the shear capacity of each pier, the predicted values may deviate from the actual response. Despite this, the equation serves as a valuable reference for assessing the global shear capacity of the system.

The ultimate drift achieved, of approximately 0.72%, is higher than that observed in the half-scale single wall experimental tests by Fortes et al. (2016), that is around 0.52%. This difference may be attributed to flanges' contribution to enhancing the structural system's ductility. Nonetheless, the ultimate drift level achieved falls within the range of drifts reported in other experimental tests of single PGRM walls in the literature, from 0.43% to 1.05% (Elmapruk; ElGawady; Hassanli, 2020). Thus, the latter numerical strategy following the described convergence criterion will be adopted for subsequent stages of the work due to its efficacy in nonlinear solving stages.

$$V_n = k_{gv}k_{gh}\beta_r A_{eh}\sqrt{f_m^*} + 0.4P \tan \theta + 0.02A_v f_{yv}\sqrt{f_m^*} + 0.02\rho_h A_{ev} f_{yh}\sqrt{f_m^*} \quad \text{Eq. 19}$$

where: k_{gv} is the factor concerning the vertical grouting; k_{gh} is the factor concerning the horizontal grouting; β_r is the factor concerning the shear span ratio; A_{eh} is the effective horizontal cross-sectional area of wall; f_m^* is the prism compressive strength corrected to a height-to-thickness ratio of prism equal to 5; P is the axial compressive load on the section under consideration; θ is the angle formed between the wall axis and the strut; A_v is the total cross-sectional area of vertical reinforcement; f_{yv} is the yield strength of vertical reinforcement; ρ_h is the total horizontal reinforcement ratio; A_{ev} is the effective vertical cross-sectional area of wall; f_{yh} is the yield strength of horizontal reinforcement. Further details are presented in Medeiros et al. (2022b).

5.3.2.2 Loading approaches

The previously presented pushover analysis scheme, which applied displacement at the top of the structure, was carried out to verify the consistency of the numerical response based on the walls tested by Fortes et al. (2016). From now on, the behavior of the structural system will be investigated through a load pattern-based pushover analysis. Pushover analysis is widely used to evaluate the nonlinear behavior of structures subjected to lateral loads, such as those induced by seismic events (Aşıkoğlu; Vasconcelos; Lourenço, 2021; Cattari et al., 2022; Easa;

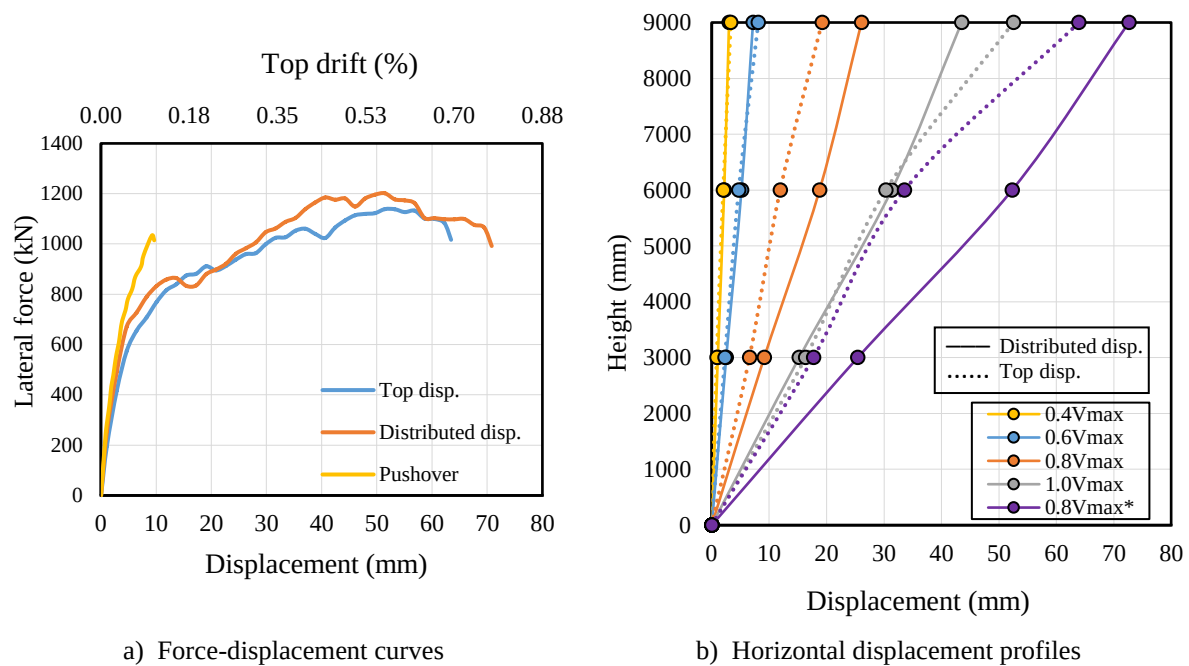
Yan, 2019; Endo; Pelà; Roca, 2017; Kuria; Kegyes-Brassai, 2023; Rosin et al., 2018; Wang; Martinez-Vazquez; Zhao, 2020). It involves applying incremental lateral forces to the structure according to a prescribed load pattern. This type of analysis is covered by several building standards, such as Eurocode 8 Part 1 (2004), FEMA 440 (2013), and NTC (2018).

Firstly, in this work, a pushover analysis procedure based on force control was attempted, according to the approach implemented in DIANA (DIANA FEA, 2022) to consider a distribution of lateral forces associated with the dominant vibration modes (“modal pushover analysis”). However, numerical convergence issues were encountered for early load steps, making the analysis impractical. The potential reasons for this may be related to the complexities of the model, primarily introduced by the interface elements, which make convergence challenging to achieve using force control. As an alternative, a displacement-controlled pushover analysis strategy was adopted, where displacements are imposed at each floor level following the first modal shape obtained from eigenanalysis.

In fact, if the seismic mass is equal on each floor, as is the case, the displacement distribution results in a load pattern similar to that of the first modal distribution (displacements of the first mode multiplied by the masses), since in the calculation of the equivalent seismic forces the mass of each floor can be factored in the formula of the load distribution. Beyond the first modal distribution, the load pattern could also be considered proportional to the mass. Although it is usually assumed that the actual response of a structure corresponds to a distribution within the envelope of these two (Aşıkoğlu; Vasconcelos; Lourenço, 2021), here, the first modal distribution is considered more representative of the behavior of the structural system under study. Due to the symmetrical configurations of the structures, the analysis was conducted only in the X direction.

The force-displacement curves corresponding to the two pushover analysis strategies (distributed displacements and force-based pushover) and the previous analysis applying displacement at the top of the walls are plotted in Figure 94a. The force-displacement curves represent the sum of base shear reactions against the top translation. They were considered up to a 20% degradation of the structure's lateral capacity. Furthermore, in Figure 94b, the displacement profile corresponding to the analyses following the mode shape and the analyses applying displacement only at the top are presented. The evolution of displacement along the height of the structural systems was monitored at each floor level, considering the force levels of $0.2V_{\max}$, $0.4V_{\max}$, $0.6V_{\max}$, $0.8V_{\max}$, $V_{\max}$, and $0.8V_{\max}^*$. The latter corresponds to a post-peak degradation of 20% of the maximum lateral force capacity.

Figure 94: Comparison of the different pushover analysis strategies.



Source: Author.

As may be seen in Figure 94a, the nonlinear response of the evaluated structural systems could not be fully captured with force-based pushover analysis, and limited stiffness degradation is observed. When comparing the results of the pushover analyses using the displacement distribution pattern according to the first modal shape with applying displacement only at the top, similar results were observed with only slight differences (around 6.4% on V_{max}). This comparison reinforces the reliability of the displacement-controlled pushover analysis proposed in this research. Hence, this method was deemed numerically more suitable and, therefore, used for subsequent analyses.

Regarding horizontal displacement profiles, the deformed shape is almost the same among the analyses up to the $0.6V_{max}$ level. However, from $0.8V_{max}$ onwards, there is a tendency for larger drifts in the top floor when displacement is applied at the top. This trend becomes more pronounced at peak load, as the displacements between the first and second floors are similar, while the third floor shows larger displacements. During the post-peak stage, when displacement is only applied at the top, the increments in displacement are even more noticeable on the third floor. This distribution of displacements between the floors consequently alters the stress distributions and load path in the walls of each floor, justifying the slight differences observed in the overall load-displacement response.

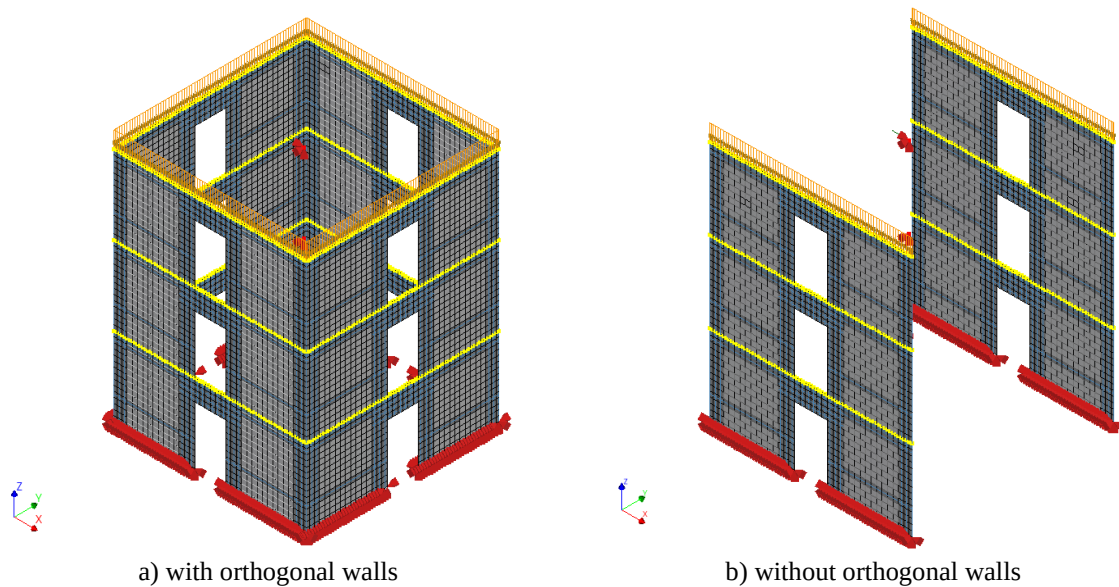
5.3.3 Systematic analyses

5.3.3.1 Influence of flanges

In simplified design procedures, walls are typically treated as rectangular cross-sectional elements, disregarding their connection with orthogonal walls and its potential impact on the overall building behavior under lateral forces. The influence of considering orthogonal walls by comparing configurations with and without their inclusion is evaluated here, as shown in Figure 95. The comparison involves analyzing the structural system with three floors, and then the same system but with only one floor. Models with only one storey were expected to yield lower ultimate displacements. Therefore, the displacement increment was reduced from 0.3 mm to 0.2 mm.

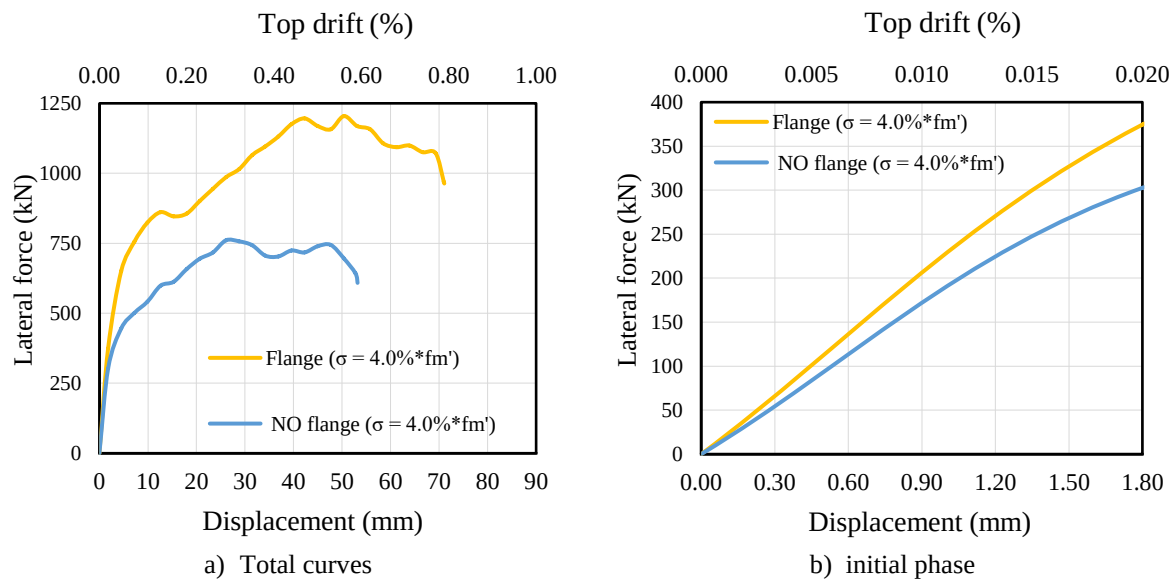
The force-displacement curves for both configurations, with and without orthogonal walls, are depicted in Figure 96 and Figure 97. Note that the complete responses are presented on the left side of these figures, while a zoom of the initial phase of the responses is provided on the right side.

Figure 95: Meshed numerical models with and without orthogonal walls.



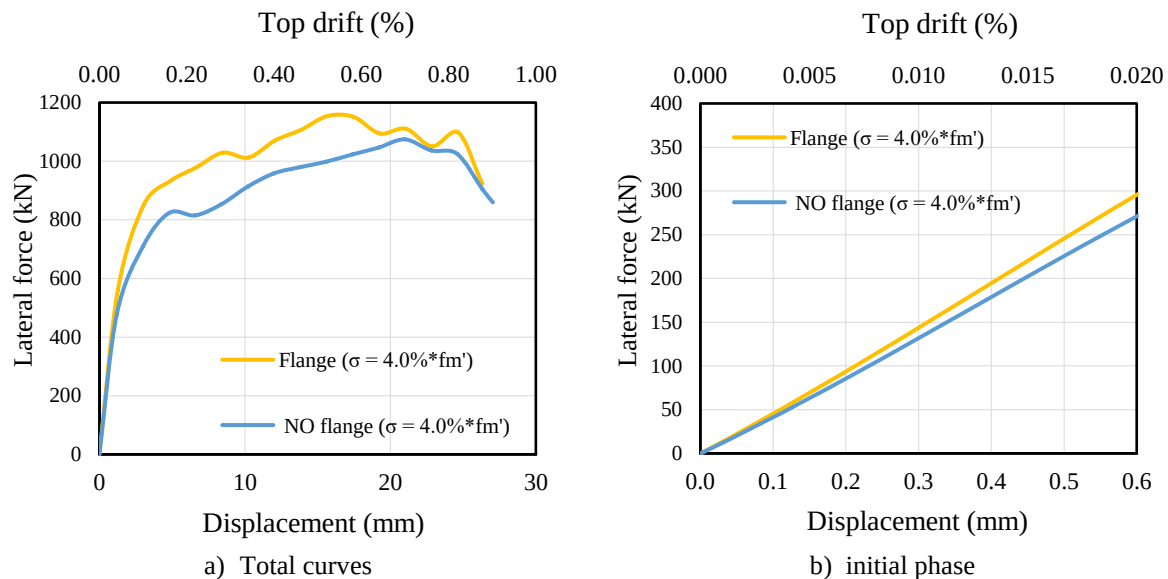
Source: Author.

Figure 96: Influence of orthogonal walls (flanges) on force-displacement response (three storeys).



Source: Author.

Figure 97: Influence of orthogonal walls (flanges) on force-displacement response (one storey).



Source: Author.

From Figure 96, a notable increase in load capacity (approximately 54%) and ultimate displacement (around 33%) is observed for the three-storey configuration when modeled with orthogonal walls (flanges). It is also noticed an increase in initial stiffness, emphasizing the contribution of orthogonal walls to the stiffness of buildings. Conversely, the differences in the system with only one floor are less pronounced, with an increase of 7.0% and a decrease of 3.0% for maximum load and ultimate displacement, respectively (Figure 97). The more significant disparity in the three-storey system may be attributed to its higher aspect ratio, because of a more remarkable influence of the flexural behavior. In contrast, the single-storey

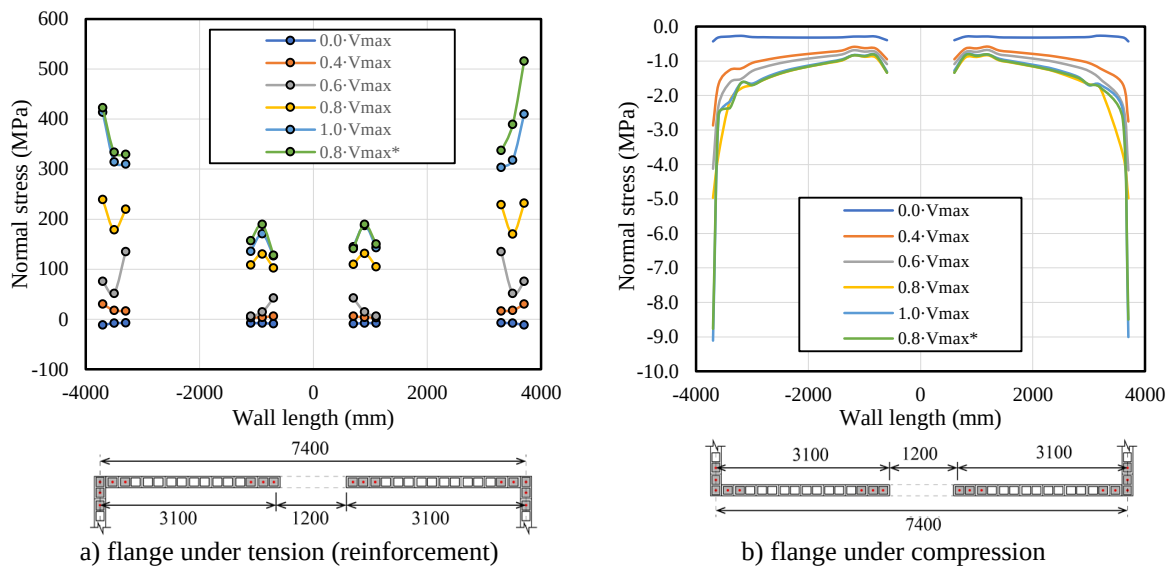
system exhibits more predominant shear behavior, with the orthogonal walls playing a comparatively minor role in its overall structural response.

Regarding failure mechanisms, three primary damage patterns were identified for each structure: (1) interface damage, characterized by stair-stepped cracks forming along the compression struts on all stories; (2) horizontal masonry cracks in the tensioned heel and vertical cracks at the toe edge; and (3) diagonal cracks in the ungrouted regions, as shown in Figure 103. As the lateral load increased, the extent and severity of damage intensified, further emphasizing the influence of structural detailing on failure progression. These failure modes align with observations from previous experimental tests (Fortes, 2017).

To further explore the contribution of the flanges to the overall behavior of structural systems, Figure 98 and Figure 99 present the stresses obtained in the vertical reinforcements of the orthogonal wall under tension and the compressive stresses of the flange under compression for both configurations, considering the force levels of $0.0V_{\max}$, $0.4V_{\max}$, $0.6V_{\max}$, $0.8V_{\max}$, $V_{\max}$ and $0.8V_{\max}^*$. The latter corresponds to a post-peak degradation of 20% of the maximum lateral force capacity.

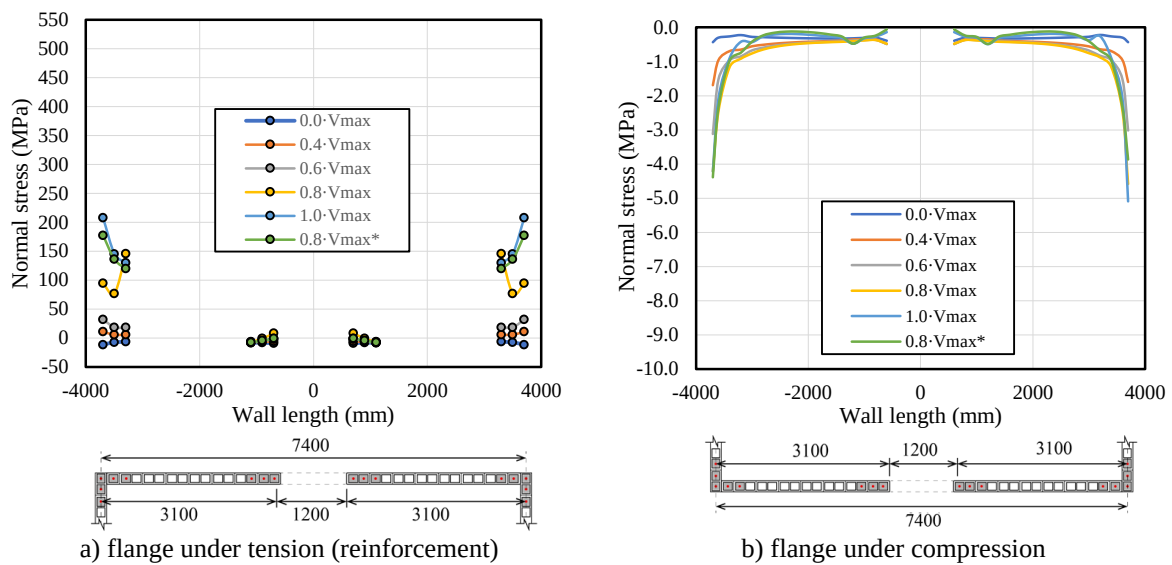
From Figure 98, considerable stress is noticeable in the vertical reinforcements in the orthogonal wall under tension (around 200 to 300 MPa in peak load). Significant compressive stress levels are also taken by flanges under compression. Stress levels are higher at the ends of the orthogonal walls due to stress transfer from the web walls, gradually diminishing further along the length, indicating that the influence of flanges is more pronounced within a certain effective length. The participation of flanges in stress sharing underscores the influence of the flexural behavior in this three-storey structural system. Although the configuration with only one storey exhibits a similar trend for the behavior of the flanges, albeit with lower tension levels (reinforcement stress around 100 MPa; see Figure 98), the prevalence of shear behavior in the one-storey structure compared to the three-storey system is emphasized.

Figure 98: Normal stresses in the orthogonal walls (three storeys).



Source: Author.

Figure 99: Normal stresses in the orthogonal walls (one storey).



Source: Author.

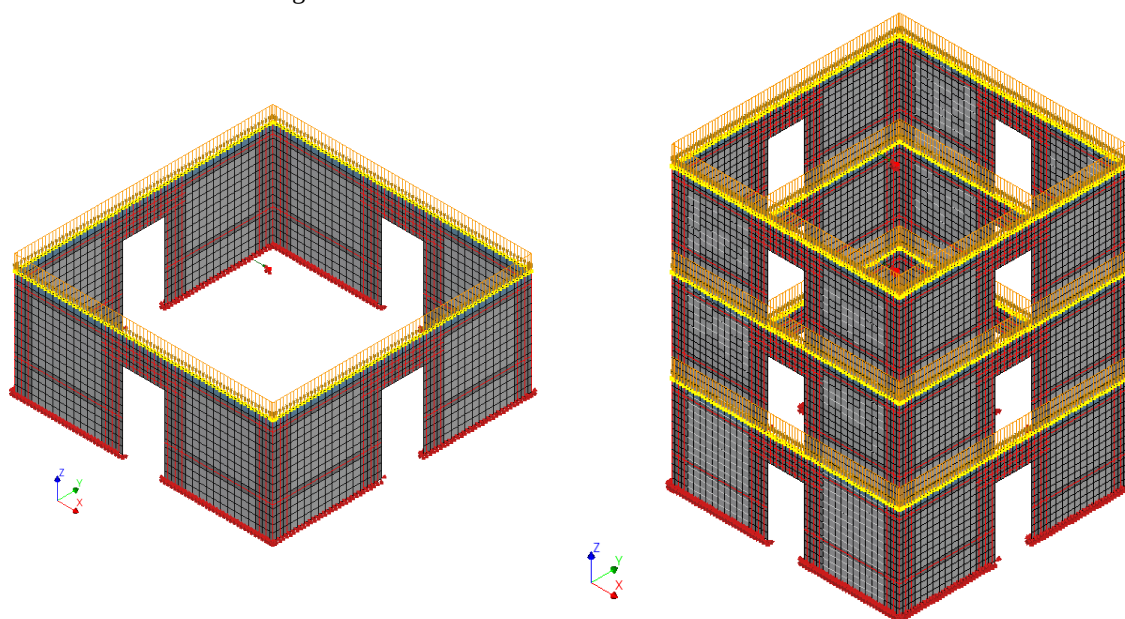
These findings are directly implicit in the force-displacement curves of the structural systems (Figure 96 and Figure 97) and explain the significant impact observed in the three-storey model without flanges and the comparatively minor impact in the one-storey configuration.

5.3.3.2 Axial load and aspect ratio

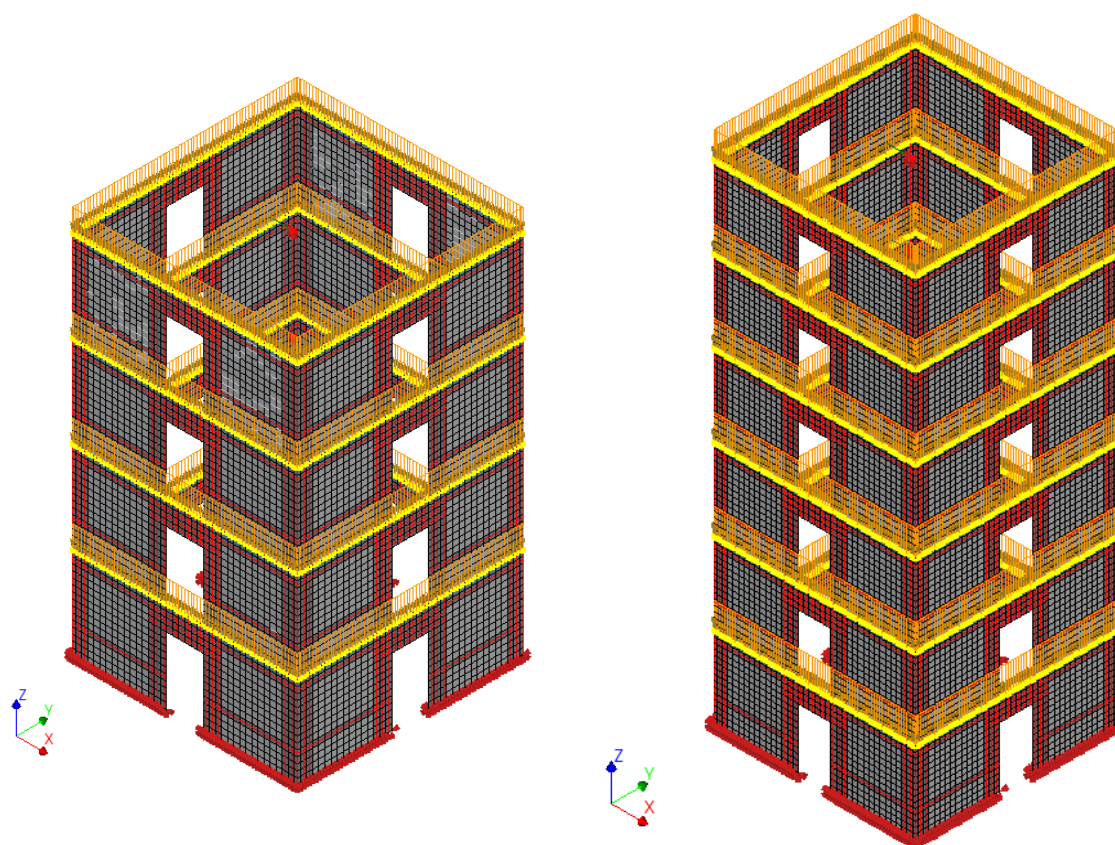
Two distinct scenarios of axial load were investigated in this study. The first scenario comprises an axial load of 4% of the masonry compressive strength (f_m) at the base of the walls. The second scenario involves a higher axial load, resulting in a compressive stress of approximately 20% of f_m . Unlike the models with low axial loads, the models with high axial loads have vertical loads equally distributed along all floors instead of only on the top floor. The compressive stress level of $0.04 \cdot f_m$ reflects experimental testing conditions due to the difficulty of applying higher axial load levels in tests. Conversely, the stress level of $0.2 \cdot f_m$ is more representative of mid and high-rise masonry buildings. Furthermore, this study also investigated the nonlinear behavior of four scenarios in terms of the building aspect ratio, considering one, three, four, and six stories, resulting in aspect ratios of 0.39, 1.18, 1.58, and 2.37, respectively. The number of stories was varied to assess the influence of aspect ratio on the nonlinear behavior of the building. The selected configurations represent typical low- and mid-rise masonry structures, allowing a broader understanding of their seismic performance.

To facilitate the identification of numerical models, a nomenclature based on essential characteristics of the structures was employed as follows: B (indicating a three-dimensional structural system representative of a simple building) – PGRM (referring to the type of walls) – C (indicating vertical reinforcement and grout concentrated at the wall ends) – Aspect ratio (number of storeys in parenthesis) – Percentage of axial stress in relation to the masonry compressive strength. For example, B-PGRM-C-1.18(3)-4 denotes a PGRM wall three-dimensional system, with reinforcement/grout concentrated at the ends, a top height to length ratio of 1.18 (corresponds to three storeys), and under axial stress corresponding to 4% of f_m . The numerical model configurations are shown in Figure 100. The six-storey structural system was not subjected to a low axial load condition because only its self-weight would already induce an axial load larger than 4% of f_m , making this condition physically unrealistic.

Figure 100: Numerical models and their nomenclature.



a) B-PGRM-C-0.39(1)-4 and B-PGRM-C-0.39(1)-20 b) B-PGRM-C-1.18(3)-4 and B-PGRM-C-1.18(3)-20



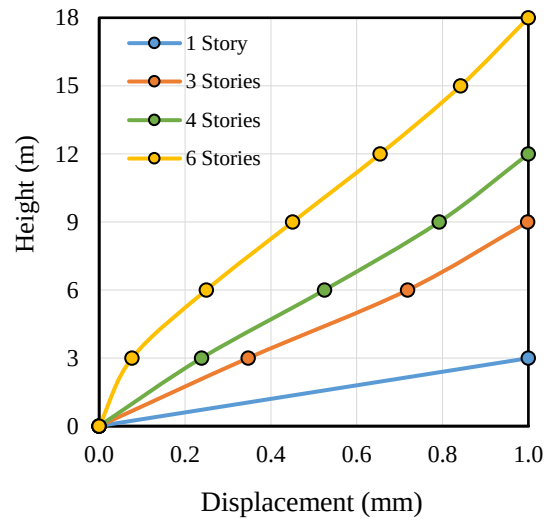
c) B-PGRM-C-1.58(4)-4 and B-PGRM-C-1.58(4)-20

d) B-PGRM-C-2.37(6)-20

Source: Author.

As previously mentioned, the displacement pattern imposed on the various floors follows the modal shape obtained from eigenanalysis in Section 5.3.1. Figure 101 illustrates the displacement pattern imposed for the models with one, three, four, and six storeys.

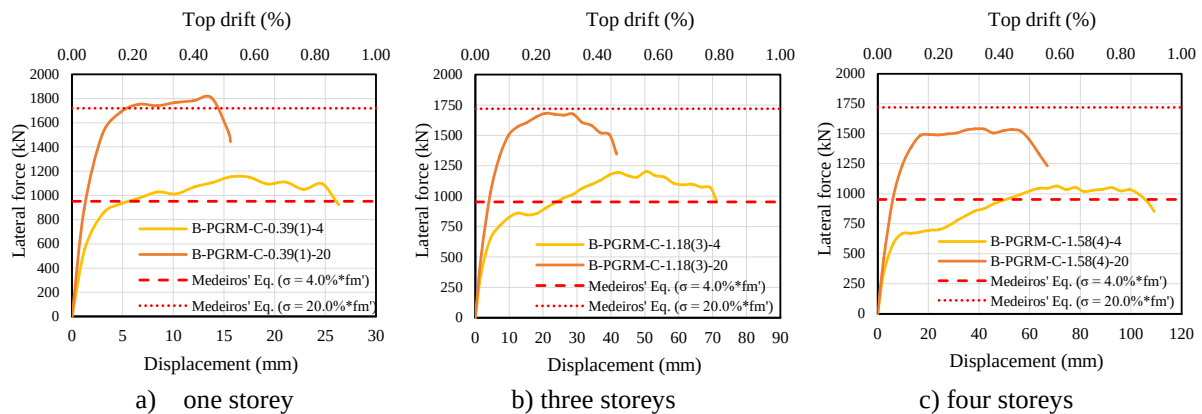
Figure 101: Displacement patterns imposed for pushover analyses.



Source: Author.

The influence of the axial load is demonstrated in Figure 102. In addition to the force-displacement curves, the prediction of maximum force estimated from the equation proposed by Medeiros et al. (2022a) based on fitting of experimental results, adapted for cases with openings, is also plotted in Figure 102. The force-displacement curves are restricted to the stage where the peak load decreases by 20%. The measured displacement corresponds to the top displacement of the system.

Figure 102: Influence of the axial load on the force-displacement curves.



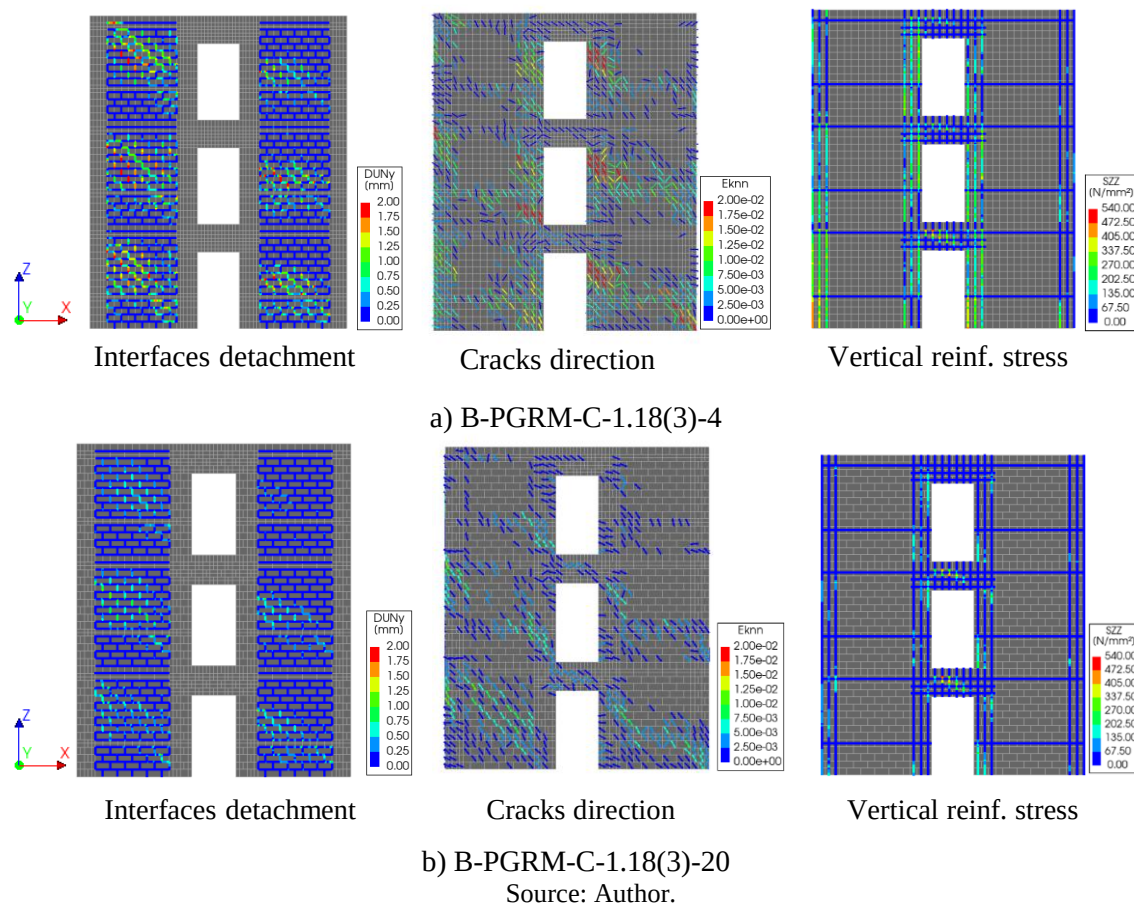
Source: Author.

The axial load level notably influences the behavior of the structural system, particularly regarding lateral load capacity and ductility. The decrease in ductility, evidenced by the force-

displacement curves, indicates that the high vertical pre-compression leads to a more brittle response in the wall systems, although with increased lateral force capacity. The maximum lateral force, V_{max} , increased by 56.4% (1 storey), 39.8% (3 storeys), and 44.7% (4 storeys) when comparing systems with higher axial loads against the same walls with lower axial loads. Furthermore, it is worth noting that the numerical responses are consistent in magnitude with the analytical estimates of the maximum base shear, thus somehow validating the numerical results obtained. The ductility of all models will be quantified and presented later through seismic parameters.

The damage patterns corresponding to the maximum lateral force of the three-storey building for both axial load cases (B-PGRM-C-1-1.18(3)-4 and B-PGRM-C-1.18(3)-20) are displayed in Figure 103. The interface detachment (equivalent to cracks in the mortar joints), crack direction on the masonry and the vertical reinforcements axial stress levels are shown in Figure 103a for the B-PGRM-C-1-1.18(3)-4 model and in Figure 103b for the B-PGRM-C-1.18(3)-20 model.

Figure 103: Damage pattern at V_{max} for the B-PGRM-C-1-1.18(3)-4 and B-PGRM-C-1.18(3)-20 models.



Under low axial load (B-PGRM-C-1-1.18(3)-4), severe interface damage is noticeable, revealing stair-stepped cracks along the compression struts on all storeys. In the masonry, cracks are mainly observed in the reinforced grouted regions. Horizontal cracks were visible in the tensioned heel (left edge), and vertical cracks were observed in the toe edge. Additionally, diagonal cracks are visible in the ungrouted regions. The damage patterns are similar for both axial load scenarios; however, the damage level is significantly reduced as the axial load increases (B-PGRM-C-1.18(3)-20). The vertical reinforcements exhibit reasonable tension stress levels for the B-PGRM-C-1-1.18(3)-4 model (around 416 MPa) in the outermost vertical bars, though they do not reach the yield stress of 540 MPa. The axial stress levels in the vertical reinforcements are significantly reduced under high axial load (B-PGRM-C-1.18(3)-20), which indicates the primarily shear-dominated failure mode. Figure 103 shows raised tension stress levels in the lintel stirrups in both axial load scenarios (up to 500 MPa).

The stiffness degradation, represented as a function of lateral displacements, is presented graphically in Figure 104 according to the number of storeys. The secant stiffness ($K_{s,i} = V_i/d_i$) and the top lateral displacement (d_i) are normalized by the initial secant stiffness (K_{sec}) and the displacement corresponding to the maximum lateral force (d_{Vmax}), respectively. The elastic stiffness was calculated at a level ranging from 5% to 33% of the maximum lateral force, V_{max} , according to the guidelines outlined in TMS 402/602 (2022) and CSA S304 (2014).

Figure 104: Lateral stiffness degradation of the wall structural systems.

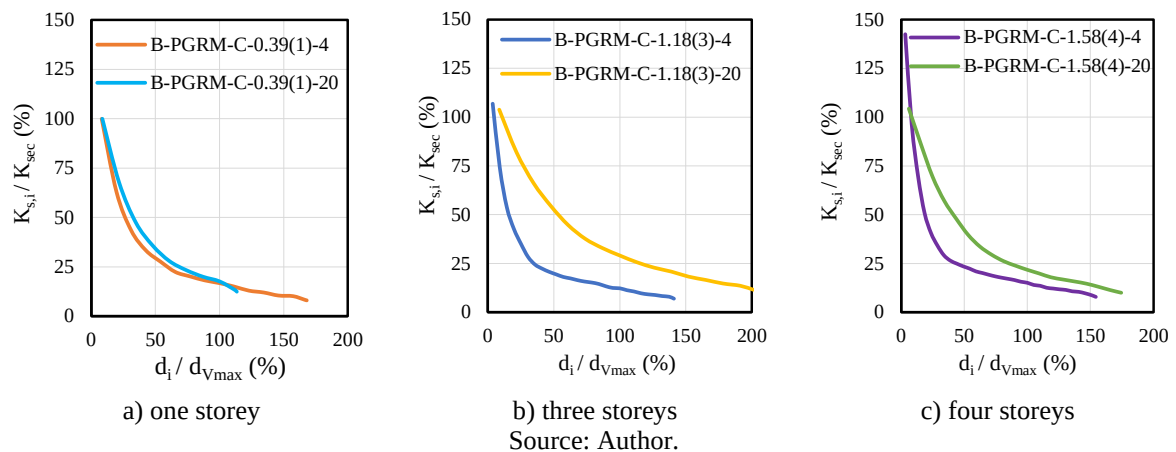


Figure 104 shows that stiffness degradation was more pronounced when the walls experienced lower pre-compression levels, despite this effect being less pronounced for the single-storey structural system than in other cases. Specifically, at 50% of d_{Vmax} , the single-storey structural system exhibited 30% and 35% degradation levels for low and high axial load levels, respectively. For the three- and four-storey configurations, the degradation was

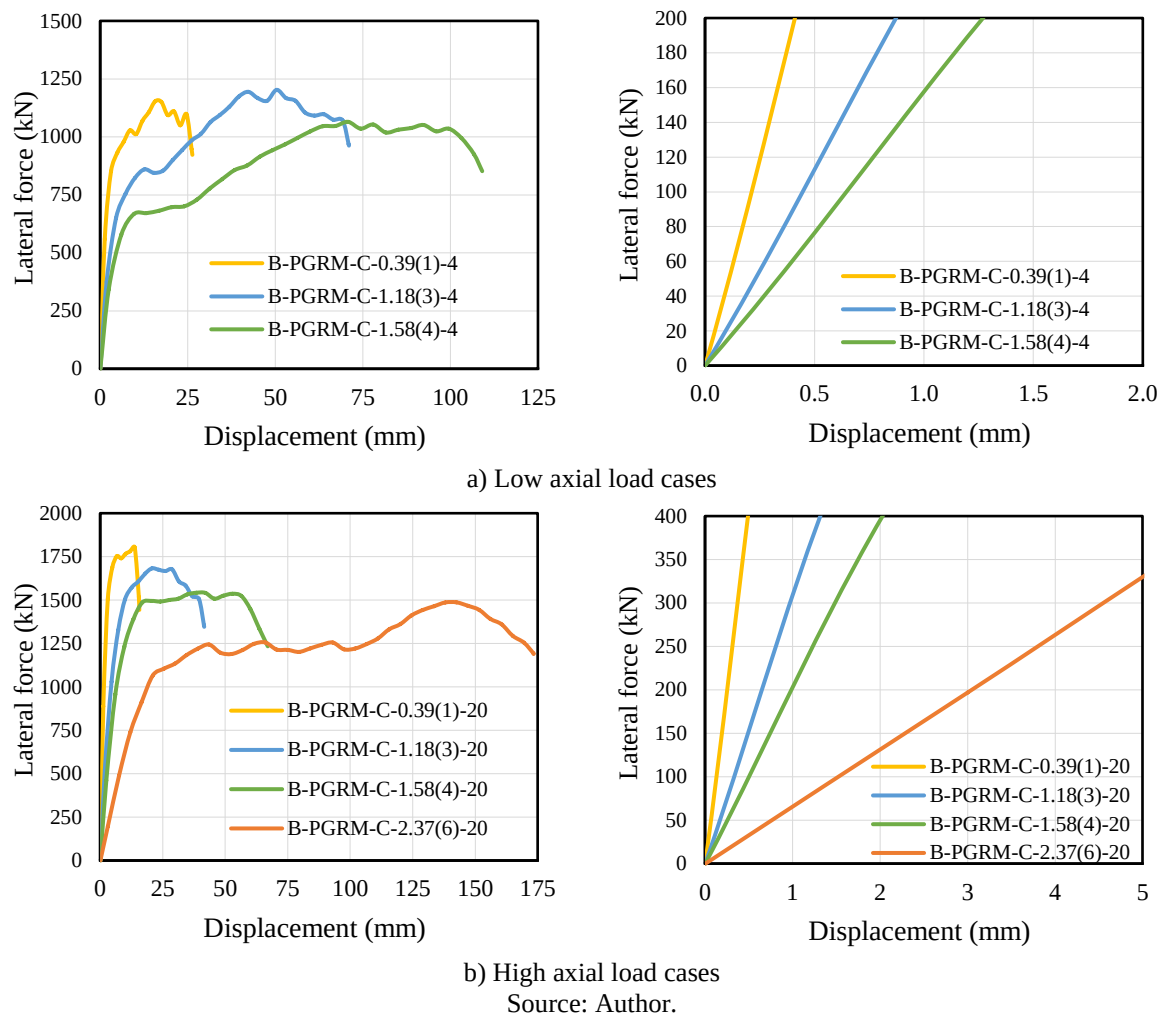
approximately 20% and 22% under low axial load and about 50% and 43% under high axial load, respectively. This behavior is expected since higher levels of pre-compression typically delay the beginning of cracking in the masonry and enhance the shear friction component in the post-cracking, thus mitigating the extent of stiffness degradation.

In addition to illustrating the evolution of stiffness degradation, the graphs in Figure 104 provide insight into each structural system's displacement in the post-peak stage. In general, the wall structural systems could reach displacements of approximately 50% the $d_{V_{max}}$ in the post-peak stage. Noticeable are the B-PGRM-C-0.39(1)-20 system, which showed a minor displacement in the post-peak regime, indicating a sudden drop after V_{max} , and the B-PGRM-C-1.18(3)-20 system, which achieved a significant displacement of approximately 100% the $d_{V_{max}}$ in the post-peak regime, demonstrating considerable ductility even under high axial load levels.

The influence of the aspect ratio (number of storeys) is shown in Figure 105 for the low axial load cases (Figure 105a) and the high axial load cases (Figure 105b). Note that the complete responses are presented on the left side, while a zoom of the initial phase of the responses is provided on the right side to emphasize the substantial influence of the aspect ratio on the stiffness of the structure.

From Figure 105, there is a clear trend that an increase in the number of storeys leads to a reduction in initial stiffness and an increase in the ultimate displacement for both low and high axial load cases. In comparison to the single-storey configuration, as the number of storeys increases, there was a reduction of 55.8% (three storeys) and 68.9% (four storeys) for low axial load cases, and 62.3% (three storeys), 75.1% (four storeys), and 91.2% (six stories) for high axial load cases.

Figure 105: Influence of the aspect ratio (number of storeys) on the force-displacement curves.



Regarding the absolute values of ultimate displacement, d_u , significant increases of 170% (three storeys) and 330% (four storeys) for low axial load cases, and 167% (three storeys), 329.5% (four storeys), and 1011% (six storeys) for high axial load cases were found. In terms of top drift, the percentage differences compared to the single-floor model do not follow a clear trend and are numerically less significant, as follows: -9.9% (three storeys) and 3.8% (four storeys) for low axial load cases; and -10.9% (three storeys), 7.4% (four storeys), and 84.8% (six storeys) for high axial load cases.

Additionally, a slight reduction trend in maximum lateral force is observed as the number of floors increases. However, it is less evident between the models with three and six floors with high axial load and not observed between the models with one and three floors with low axial load. A more practical quantitative analysis for design purposes will be presented later through seismic parameters related to the ductility of each configuration.

5.3.3.3 Vertical reinforcement detailing

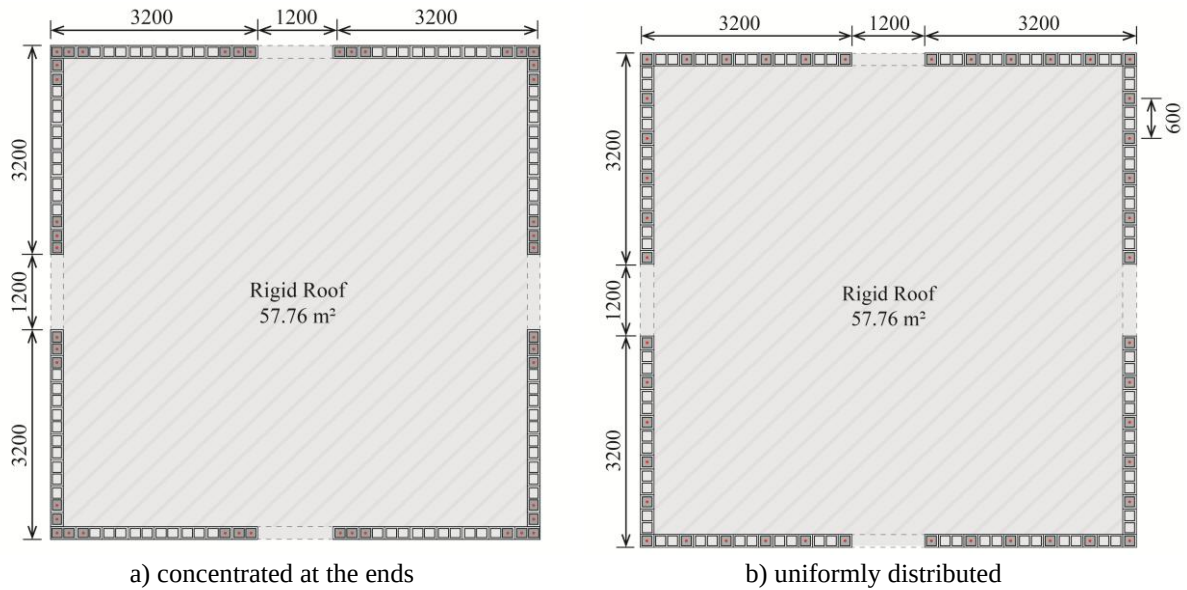
Reinforcement detail variations were introduced to investigate the effect of different vertical grouting and reinforcement spacing configurations while maintaining a constant effective net cross-sectional area and reinforcement ratio. This ensures that observed differences in response are due to the reinforcement distribution rather than to variations in total amount of reinforcement, allowing for direct comparison.

Haider and Dhanasekar (2011; 2004) investigated the influence of vertical reinforcement spacing through experimental tests on PGRM walls built with hollow clay blocks. The walls contained a small amount of vertical reinforcement (approximately 0.14%) placed at large horizontal spacings ranging from 800 mm to 2000 mm and were tested under monotonic in-plane lateral loading with a vertical compressive stress of approximately $0.04 \cdot f'_m$. Their results demonstrated that more uniformly distributed vertical reinforcement led to higher ductility, ultimate displacement, and a lower stiffness degradation rate compared to walls with larger spacing between vertical bars. Single vertical hollow cells were grouted at different spacing and wall positioning. Bolhassani et al. (2016c) further showed that using side-by-side grouted and reinforced cells significantly improves the behavior of PGRM walls by allowing each double-grouted-cells region some flexural capacity. The wall region in between each double-grouted-cells may behave as a diagonal strut. This configuration increased lateral strength, ductility, and stiffness compared to conventional single-cell detailing. Medeiros et al. (2022a) numerically evaluated PGRM walls under cyclic lateral loading and found that reinforcement concentrated at more than one cell at the wall pier ends (in the case, three grouted and reinforced cells were detailed at each pier end) can yield comparable design efficiency to that of uniformly distributed reinforcement, provided the total amount of grout and steel is preserved.

In this study, two types of detailing were analyzed to investigate how different reinforcement detailing approaches influence the behavior of PGRM walls: one scenario involved vertical grouting and reinforcement concentrated (C) at the ends, while the other involved vertical grouting and reinforcement distributed (D) along the wall. This research extends the analysis of these aspects into a three-dimensional context, where the interaction between flange effects, vertical load, and reinforcement detailing is evaluated—beyond the individual PGRM wall studies previously conducted by Haider and Dhanasekar (2004), Bolhassani et al. (2016c) and Medeiros et al. (2022a). The layouts with the concentrated (C)

and distributed (D) vertical reinforcement detailing approaches are shown in Figure 106.

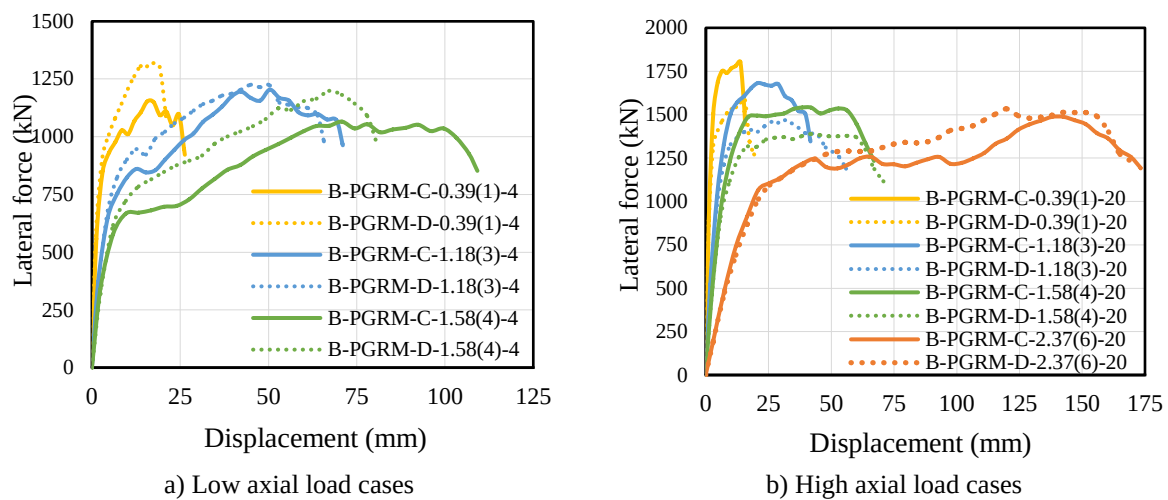
Figure 106: Vertical reinforcement detailing approaches for PGRM Walls.



Source: Author.

The same nomenclature was used based on the essential characteristics explained earlier. The only distinction is that the letter D (indicating vertical reinforcement and grout uniformly distributed along the walls) contrasts with C (indicating vertical reinforcement and grout concentrated at the ends of the walls). The influence of the vertical reinforcement detailing is demonstrated in Figure 107 for the low axial load cases (Figure 107a) and the high axial load cases (Figure 107b).

Figure 107: Influence of the vertical reinforcement detailing on the force-displacement curves.

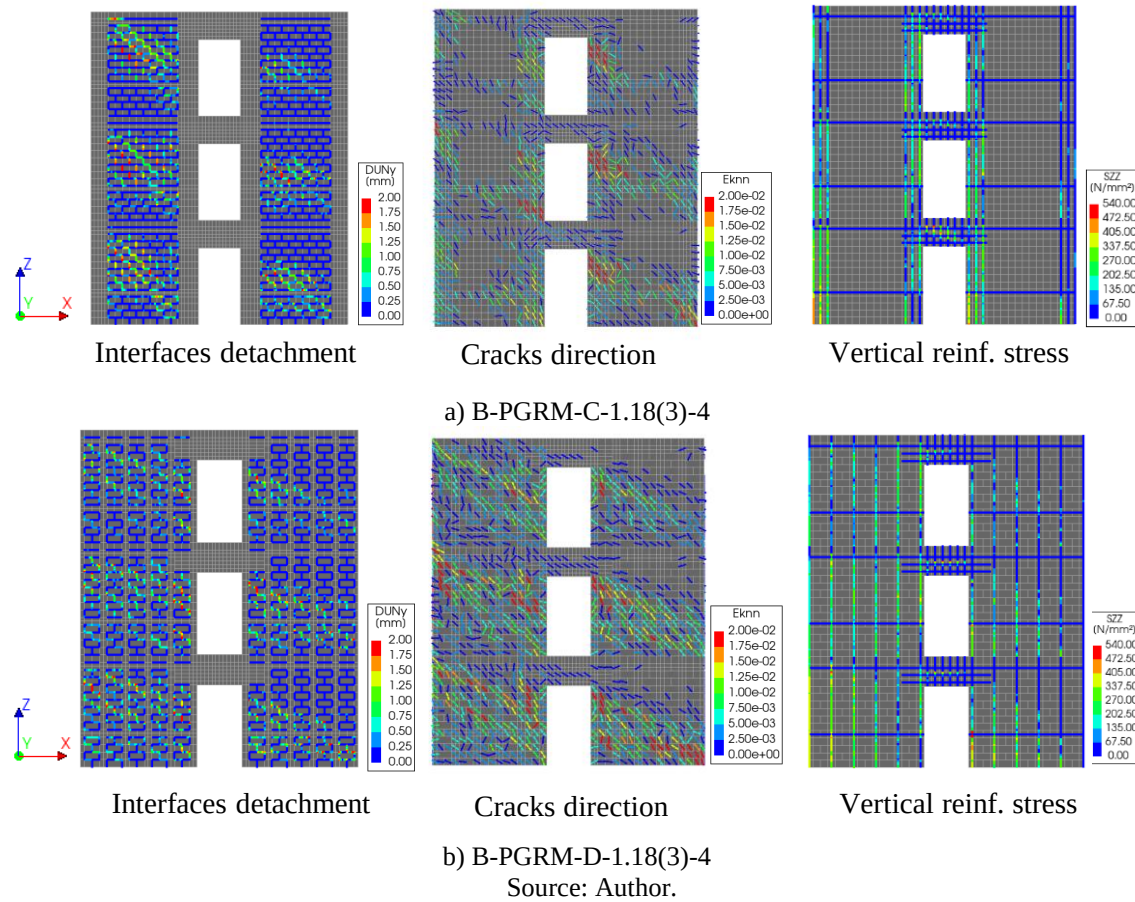


Source: Author.

From Figure 107, it is noted that the models with uniformly distributed reinforcement followed the same trends observed previously for the models with reinforcement concentrated at the ends. The increase in axial load resulted in higher load capacities and smaller ultimate displacements, and the increase in the number of storeys resulted in lower stiffness models with larger ultimate displacements.

Comparing the two approaches of distributing the vertical reinforcement, it is possible to notice that for the cases of low axial load, the uniform distribution of the reinforcement led to slight increases in load capacity, but increases in ductility are not clearly observed. For the cases of high axial load, the opposite occurs, and the models with distributed reinforcement presented lower load capacities, except for the model with six storeys, which reached practically the same load level. A remarkable increase in ductility due to a uniform vertical reinforcement distribution is observed only when comparing the B-PGRM-D-1.18(3)-20 and B-PGRM-C-1.18(3)-20 models.

The crack patterns differ between walls with grouting and reinforcement concentrated at the walls' ends and those with grouting and reinforcement distributed along the walls. Figure 108 compares the damage patterns between models B-PGRM-C-1.18(3)-4 and B-PGRM-D-1.18(3)-4 at V_{max} . When reinforcement is distributed along the walls, diagonal cracks spread throughout the entire wall, with the central reinforcements playing a role in controlling the opening of these diagonal cracks in the masonry and, consequently, the cracks in the mortar joints.

Figure 108: Damage pattern at V_{max} for the B-PGRM-C-1-1.18(3)-4 and B-PGRM-D-1.18(3)-4 models.

5.4 SEISMIC PERFORMANCE EVALUATION

Commonly, the seismic performance assessment relies on idealized lateral force-displacement responses equivalent to actual ones. However, there is no widely accepted method to trace these idealized responses, as several methods have been utilized in the literature (Banting; El-Dakhakhni, 2014; Bolhassani et al., 2016; Calderón; Sandoval; Arnau, 2017; El-Dakhakhni; Banting; Miller, 2013; Morandi et al., 2022; Ramírez; Sandoval; Almazán, 2016; Seif Eldin; Ashour; Galal, 2019). As also previously made by Medeiros et al. (2022a), three approaches (one trilinear and two bilinear) are examined to illustrate their influence on determining seismic performance factors in a three-dimensional numerical model context.

The first approach, depicted in Figure 109a and denominated as the trilinear approach (TL), idealizes the actual response as a trilinear elastic-plastic curve with the same peak load point and d_u corresponding to $0.8 \cdot V_{max}$ in the post-peak phase. The elastic stiffness was calculated according to the guidelines outlined in TMS 402/602 (2022). The idealized linear elastic segment ends at $V_y = K_{el} \cdot d_y$, with d_y calculated using Equation 20 to equalize the energy dissipation (U) under the actual and idealized curves. Seismic factors are then determined using

Equations 21 to 25, with V_{el} and d_{el} corresponding to the point at which a linear response with the secant stiffness, K_{sec} , intersects the actual response. The auxiliary variables, $\eta = V_y/V_{max}$ and $\Delta_d = d_{V_{max}}/d_y$, are used respectively in Equations 23 and 24 to facilitate their presentation.

$$d_y = \frac{2U_t + V_{max}(0.8d_{V_{max}} - 1.8d_u)}{K_{el} \cdot d_{V_{max}} - V_{max}} \quad \text{Eq. 20}$$

$$\mu_d = d_u/d_y \quad \text{Eq. 21}$$

$$\Omega = V_{max}/V_{el} \quad \text{Eq. 22}$$

$$C_d = d_u/d_{el} = \mu_d \cdot \Omega \cdot \eta \quad \text{Eq. 23}$$

$$R_d = V_e/V_{max} = \sqrt{\eta^2 \Delta_d + \eta(1.8\mu_d - 0.8\Delta_d - 1)} \quad \text{Eq. 24}$$

$$R = V_e/V_{el} = R_d \cdot \Omega \quad \text{Eq. 25}$$

The bilinear approach (BL1) is depicted in Figure 108b. This idealized response follows a bilinear elastic-perfectly plastic curve with d_u corresponding to $0.8 \cdot V_{max}$ in the post-peak stage. Like the TL approach, the initial elastic stiffness is determined by the point at which a linear response, with the secant stiffness, K_{sec} , intersects the actual response. The yielding displacement, d_y , is calculated using Equation 26 to equalize the energy under the actual and idealized curves. The calculation of μ_d and R remains consistent with Equations 21 and 25, respectively, while Equations 27 to 29 estimate Ω , C_d , and R_d .

$$d_y = d_u - \sqrt{d_u^2 - 2(U_t/K_{el})} \quad \text{Eq. 26}$$

$$\Omega = V_y/V_{el} \quad \text{Eq. 27}$$

$$C_d = d_u/d_{el} = \mu_d \cdot \Omega \quad \text{Eq. 28}$$

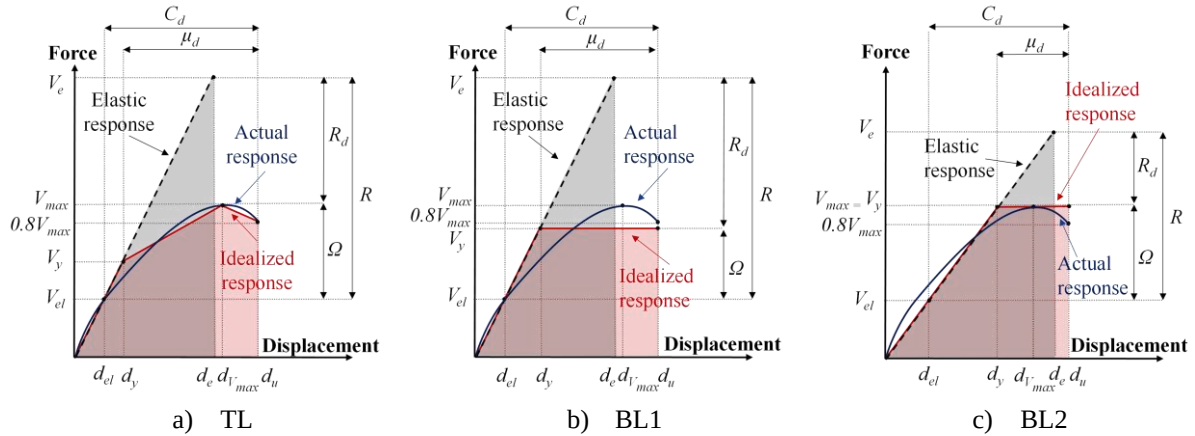
$$R_d = V_e/V_y = \sqrt{2\mu_d - 1} \quad \text{Eq. 29}$$

In the second bilinear approach (BL2), as illustrated in Figure 108c, the actual response is idealized as a bilinear elastic-perfectly plastic curve with a yielding load plateau $V_y = V_{max}$ and a displacement at a load of $0.8 \cdot V_{max}$ in the post-peak stage denoted as d_u . The yield

displacement is determined using Equation 30, which equalizes the energy dissipation (U) under the actual and idealized curves. Like the BL1 approach, the seismic factors are determined using Equations 27 to 29.

$$d_y = 2 \left(d_u - U_t / V_{max} \right) \quad \text{Eq. 30}$$

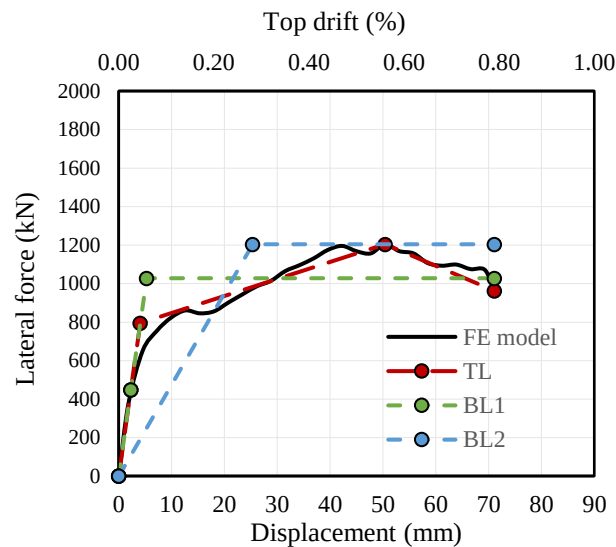
Figure 109: Idealization approaches for equivalent elastoplastic response.



Source: Author.

The idealized curves for the three approaches and the actual response are shown in Figure 110 for B-PGRM-C-1.18(3)-4 only for brevity. The idealized curves for all other wall structural systems are provided in Appendix A.

Figure 110: Actual and idealized responses for B-PGRM-C-1.18(3)-4.

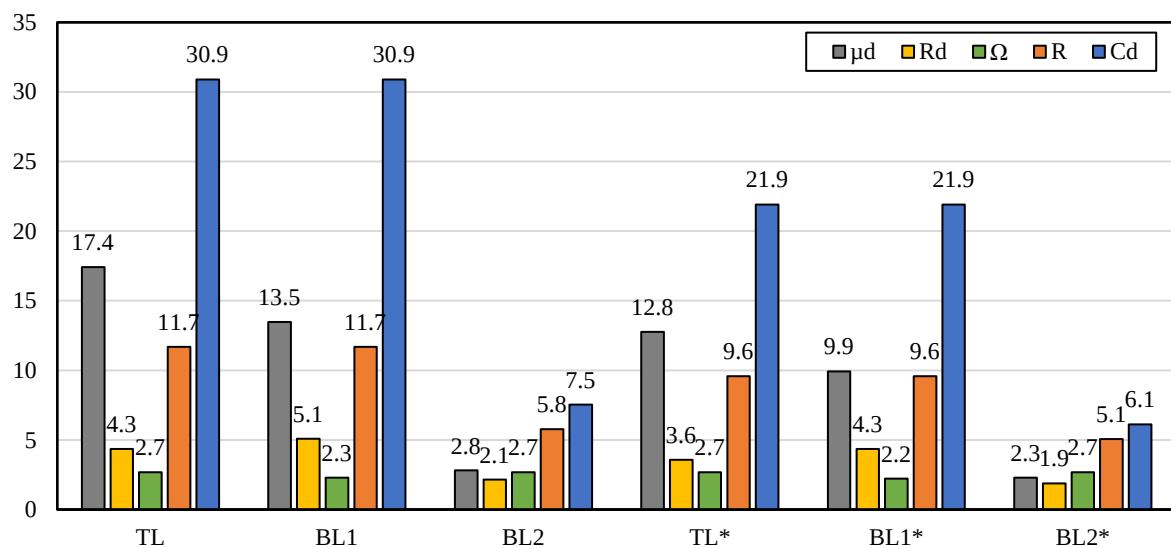


Source: Author.

Significant differences among the idealized responses are highlighted in Figure 110. The TL approach closely matches the actual response, maintaining three key characteristics: the initial elastic stiffness, the lateral force capacity, and the post-peak decay. The BL1 approach retains the same initial elastic stiffness as the actual response but reduces the lateral force capacity of the wall by approximately 20%. In contrast, despite keeping the lateral force capacity, the BL2 approach significantly reduces the initial stiffness and provides a less ductile response than the other approaches.

Disregarding the post-peak phase would still be possible for a more conservative analysis. In this scenario, V_u and d_u are set equal to V_{max} and d_{Vmax} , respectively, and the energy dissipation under the actual and idealized curves is limited to the peak load point. To quantitatively compare the seismic parameters provided by each approach, the results for B-PGRM-C-1.18(3)-4 are presented in Figure 111, considering each approach with and without considering the post-peak stage. Approaches without considering the post-peak are indicated by the * symbol superscript, e.g., TL*, BL1*, and BL2*.

Figure 111: Seismic performance factors of B-PGRM-C-1.18(3)-4.



Source: Author.

A clear trend emerges upon comparing the seismic performance factors in Figure 110: the BL2 approach exhibits the most conservative results. R_d and R are approximately two times smaller, μ_d around five times smaller, and C_d four times smaller compared to the other two approaches. The TL and BL1 approaches yield identical R and C_d values but differ in μ_d , R_d , and Ω . Specifically, while the BL1 approach shows a higher R_d , the TL approach demonstrates higher μ_d and Ω . When comparing the approaches with and without consideration of the post-

peak response, a notable reduction emerges in the seismic factors. The seismic factor R decreases by 18% in the TL and BL1 approaches and 12% in the BL2 approach. Similarly, the seismic factor R_d demonstrates a decrease of approximately 16% in the TL and BL1 approaches and 9% in the BL2 approach. Additionally, μ_d and C_d show reductions of around 26% and 29%, respectively, concerning the TL and BL1 approaches and approximately 18% in the BL2 approach. Hence, disregarding the post-peak phase implies an overly conservative strategy for PGRM systems. Nevertheless, it is crucial to acknowledge that this approach overlooks the additional ductility of the wall resulting from post-peak behavior. These observations align with findings reported by Medeiros et al. (2022a) for isolated planar PGRM walls.

The main responses of all numerical models are summarized in Table 17, including peak load and displacement, V_{max} and d_{Vmax} ; ultimate displacement, d_u ; the stress in the outermost rebar at peak load, $\sigma_{s,Vmax}$; the energy (area under the curve), U ; the natural period from eigenanalysis, T_n ; and seismic parameters obtained from the trilinear idealization. The trilinear approach was considered more appropriate because it closely matched the actual response, maintaining three crucial characteristics of the structural systems: initial elastic stiffness, lateral force capacity, and post-peak phase behavior.

Table 17: Summary of structural responses parameters and seismic performance factors.

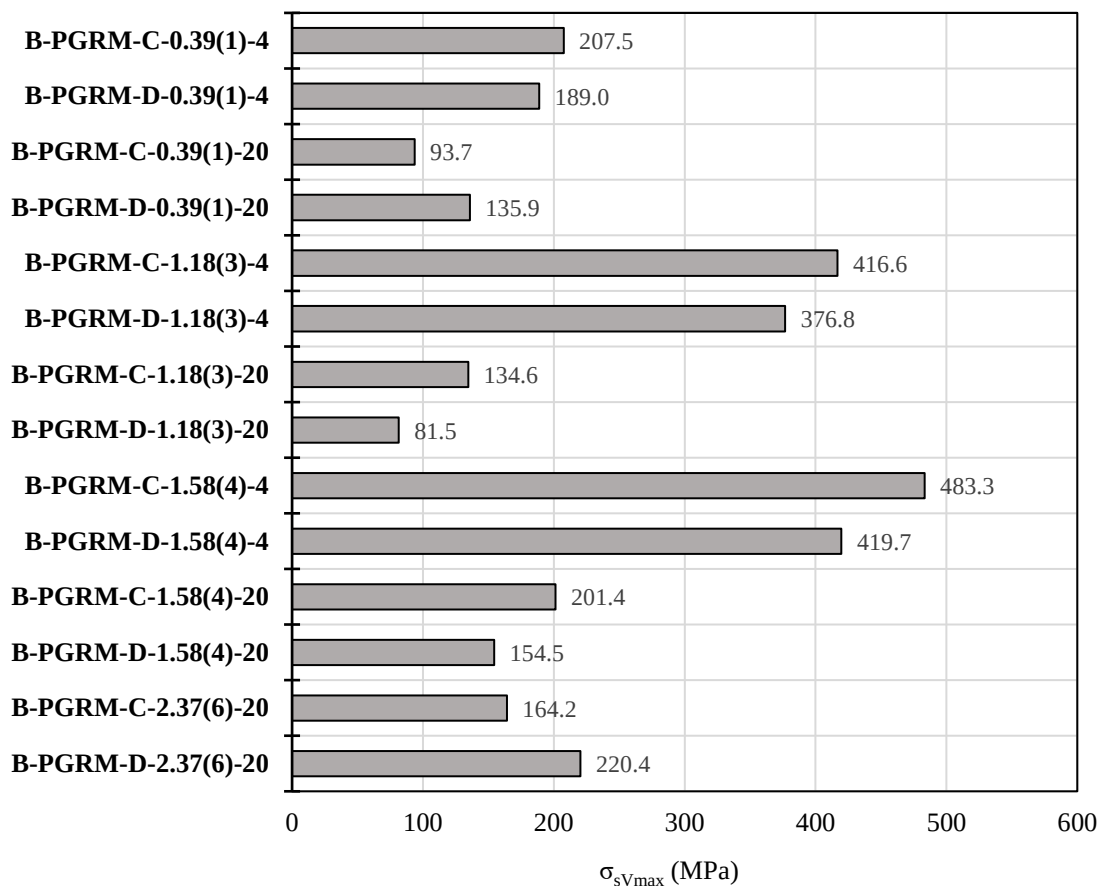
Model	V_{max} (kN)	$\sigma_{s,Vmax}$ (MPa)	d_{Vmax} (mm)	d_u (mm)	U (kN·mm)	K_{sec} (kN/mm)	T_n (s)	μ_d	R_d	Ω	R	C_d
B-PGRM-C-0.39(1)-4	1152.9	207.5	15.7	26.3	26086.8	441.0	0.05	12.7	4.2	2.0	8.4	20.3
B-PGRM-D-0.39(1)-4	1318.6	189.0	17.4	21.4	23653.5	497.4	0.05	10.6	3.7	2.0	7.5	16.5
B-PGRM-C-0.39(1)-20	1803.9	93.7	13.8	15.6	24407.2	740.8	0.05	7.3	3.3	2.0	6.8	13.0
B-PGRM-D-0.39(1)-20	1584.1	135.9	15.6	19.2	26398.0	717.8	0.05	9.0	4.0	1.7	6.8	14.8
B-PGRM-C-1.18(3)-4	1203.5	416.6	50.4	71.1	70308.8	194.7	0.12	17.4	4.3	2.7	11.7	30.9
B-PGRM-D-1.18(3)-4	1225.1	376.8	45.0	65.7	67909.4	184.0	0.12	13.2	4.1	2.7	10.9	26.3
B-PGRM-C-1.18(3)-20	1683.1	134.6	20.7	41.7	60461.3	279.1	0.12	7.6	3.5	2.7	9.5	18.9
B-PGRM-D-1.18(3)-20	1469.0	81.5	31.5	56.2	72866.9	284.3	0.12	12.3	4.4	2.9	12.6	31.2
B-PGRM-C-1.58(4)-4	1065.1	483.3	70.8	109.2	96006.2	137.3	0.17	21.9	4.8	2.9	13.9	40.4
B-PGRM-D-1.58(4)-4	1198.8	419.7	67.2	81.0	76803.4	117.4	0.17	12.5	3.5	2.4	8.6	19.3
B-PGRM-C-1.58(4)-20	1541.0	201.4	38.4	67.0	91635.6	184.6	0.17	8.3	3.8	2.6	9.8	20.9
B-PGRM-D-1.58(4)-20	1389.0	154.5	42.0	71.0	88281.0	191.0	0.17	10.3	4.2	2.9	12.2	28.4
B-PGRM-C-2.37(6)-20	1487.4	164.2	142.5	173.3	208397.9	65.3	0.26	11.1	3.5	3.0	10.7	23.1
B-PGRM-D-2.37(6)-20	1533.4	220.4	120.0	170.3	215156.9	61.8	0.27	10.9	3.3	2.9	9.4	19.6

Source: Author.

The stress analysis in the outermost rebar at peak load is shown in Figure 112. The increase of axial load level tends to decrease $\sigma_{s,Vmax}$, which means that the response becomes

more governed by shear than bending. Also, it can be observed that as the number of floors increases, the $\sigma_{s,Vmax}$ also increases, indicating a greater demand for flexural behavior in the structural system. However, this was not observed between cases B-PGRM-C-1.58(4)-20 and B-PGRM-C-2.37(6)-20, as $\sigma_{s,Vmax}$ slightly decreased from 201.0 MPa to 164.2 MPa. Regarding the influence of the distribution of vertical reinforcement, the stress levels are similar, and a defined pattern of influence on $\sigma_{s,Vmax}$ due to the distribution of reinforcement was not observed. Nevertheless, despite reaching high-stress levels, especially for the low axial load cases of three and four floors for both types of reinforcement distribution, it is noted that $\sigma_{s,Vmax}$ did not achieve the reinforcement yield stress ($f_y = 540$ MPa), indicating a mixed mode of failure involving both flexural and shear, but with shear being preponderant.

Figure 112: Stress values in the outermost rebar at peak load, $\sigma_{s,Vmax}$.

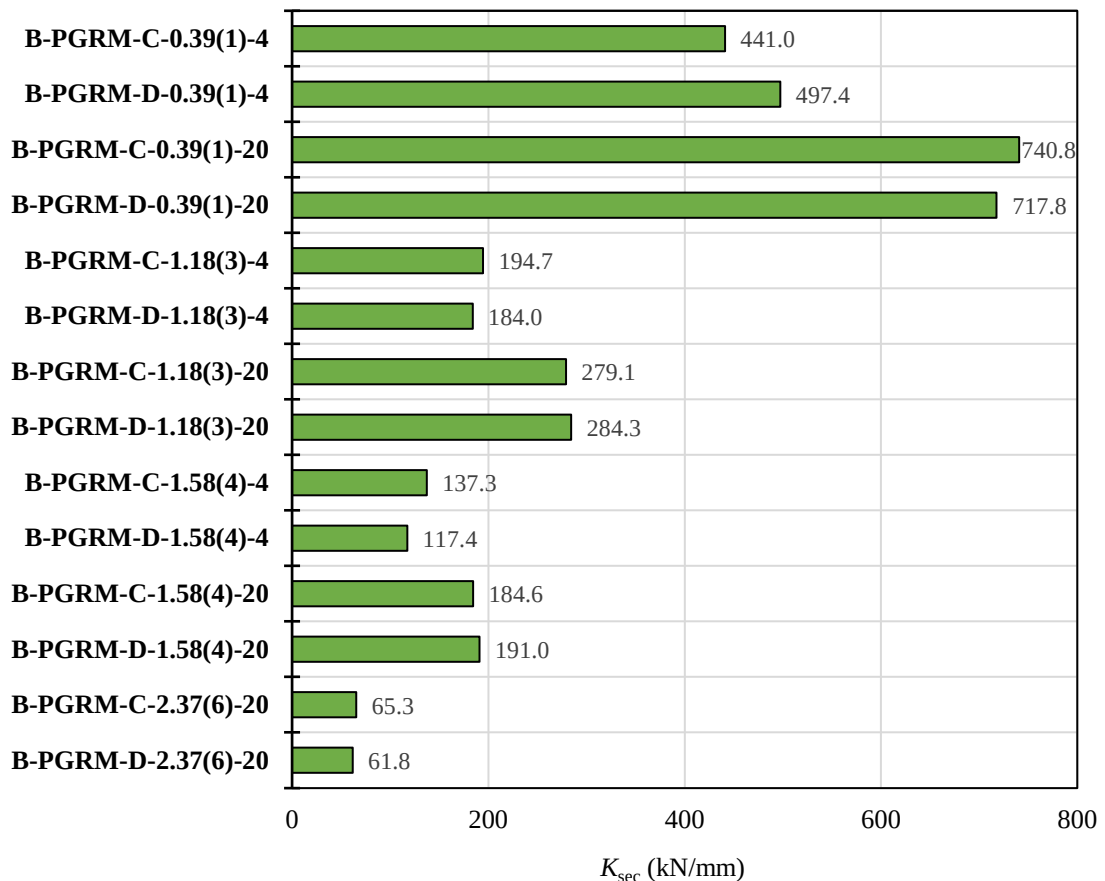


Source: Author.

The numerical values of the secant stiffness for each model are presented in Figure 113. As the value of the secant stiffness, K_{sec} , evaluated in this study depends on the ultimate load, an increase in K_{sec} is observed for models with higher axial loads. Additionally, the previous observation regarding the decrease in stiffness with an increase in the number of storeys is evident. Regarding the detailing, minor numerical variations are observed but not significant,

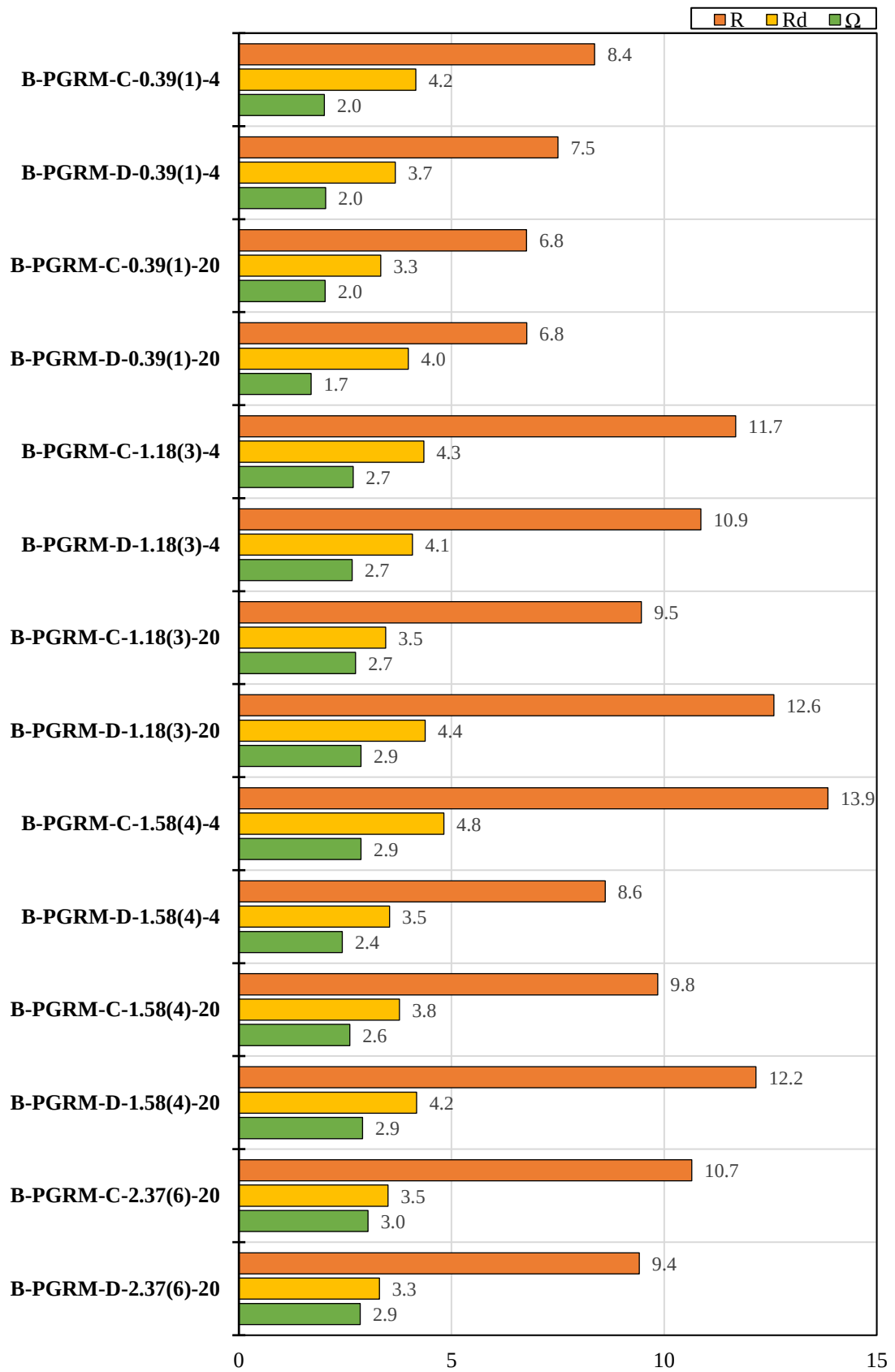
suggesting that the distribution type of reinforcement did not influence the initial stiffness. The values of the natural periods, T , obtained from eigenanalysis indicate no difference in the models' elastic behavior depending on the axial load level and the vertical reinforcement distribution. However, the increase in T with the increase in the number of storeys indirectly indicates a reduction in elastic stiffness.

Figure 113: Predicted values of secant stiffnesses, K_{sec} .



Source: Author.

This study focuses on elastic force reduction factors for seismic design. The values of R , R_d , and Ω (see Figure 109) are presented in Figure 114, again, considering the trilinear approach. The results from considering the bilinear approaches and also those that disregard the post-peak phase are presented in Appendix B.

Figure 114: Estimated values of seismic parameters R , R_d , and Ω .

Source: Author.

Considerable values of R were obtained for all cases presented here, corresponding to different design conditions. These findings demonstrate that the seismic performance of PGRM wall structural systems without irregularities and with in-plane rigid floors outperforms the expectations presented in current design standards, such as TMS 402/602 (2022) and the CSA S304 (2014).

For the detailing with reinforcements concentrated at the ends (C), the highest R values are obtained for cases with low axial load. However, this was not a trend for the detailing with uniformly distributed vertical reinforcement (D). Note that while for models B-PGRM-D-1.18(3)-4 and B-PGRM-D-1.58(4)-4, R values of 10.9 and 8.6 were obtained, respectively, models B-PGRM-D-1.18(3)-20 and B-PGRM-D-1.58(4)-20 have R values of 12.6 and 12.2, respectively. Similarly, for detailing with reinforcements concentrated at the ends, R values increase with the number of storeys. However, this was not a trend for detailing with uniformly distributed vertical reinforcement. Note that while models B-PGRM-D-1.18(3)-4 and B-PGRM-D-1.58(4)-20 comprise R values of 10.9 and 12.2, models B-PGRM-D-1.58(4)-4 and B-PGRM-D-2.23(6)-20 have R values of 8.6 and 9.4, respectively. Notably, C presents higher R values than D at low axial loads, while D exhibits higher values than C at high axial loads.

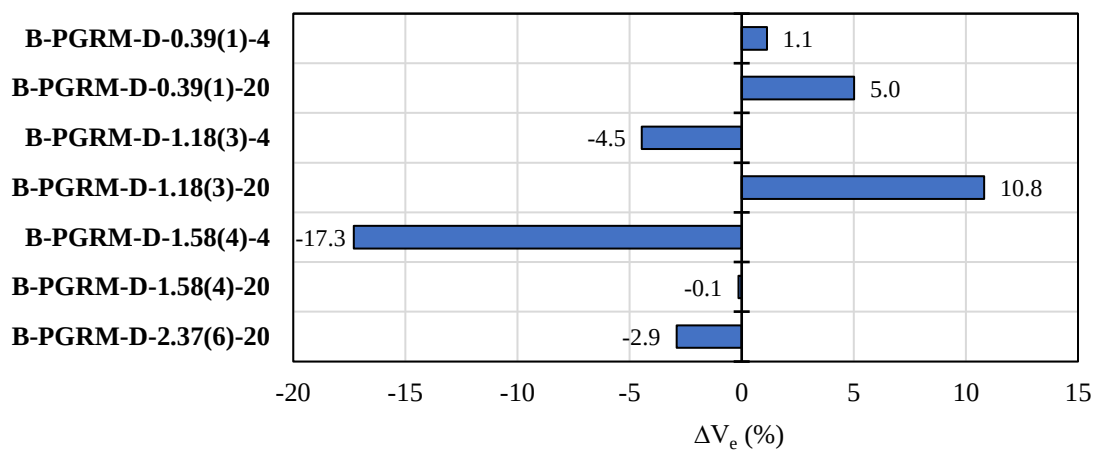
The influence of the axial load and number of storeys on the R values suggests that the design standards' approach of considering only one R factor to cover all design scenarios should be reconsidered. Furthermore, the values found in current design standards tend to be more conservative than those obtained in this study using a trilinear approach. The R factor values closest to the normative recommendations are those obtained from the most conservative method, the BL2* approach (see Appendix B).

No significant gains in lateral capacity were observed for the wall's structural system with distributed vertical reinforcement compared to those with vertical reinforcement concentrated at the wall ends. For the low axial load cases, B-PGRM-D-0.39(1)-4, B-PGRM-D-1.18(3)-4, and B-PGRM-D-1.58(4)-4 models exhibited lateral capacities 14.4%, 1.8%, and 12.6% greater, respectively, than their counterparts with reinforcement concentrated at the ends. However, for the high axial load cases, only the B-PGRM-D-2.37(6)-20 model shows a slightly higher V_{max} (3.1%) compared to the B-PGRM-C-2.37(6)-20 model.

In addition, a straightforward comparison is made here to assess the effectiveness of the vertical reinforcement detailing, using the parameter $V_e = V_{max} \cdot R_d$, which represents the maximum load the structural system will be subjected to under linear elastic behavior, as

depicted in Figure 109. Since the ductility parameters are integrated into the calculation of R_d (Equation 24), using V_e as a comparison index entails simultaneous consideration of lateral force capacity and ductility. As demonstrated in Figure 115, structural systems with uniformly distributed vertical grout/reinforcement along the wall demonstrated detailing efficiency (measured by the product $V_{max} \cdot R$) ranging from -17.3% (four storeys) to 1.1% (one storey) for low axial load cases and from -2.9% (six storeys) to 10.8% (three storeys) for high axial load scenarios. These results suggest an efficiency comparable to that of structural systems with vertical reinforcement concentrated at the wall ends.

Figure 115: Efficiency related to distribution of vertical reinforcement.



Source: Author.

These findings are in line with the work by Medeiros et al. (2022a), in which it is suggested that the reinforcement spacing limits for shear walls specified by TMS 402/602 (2022) and CSA S304 (2014) can be reduced in the center of the walls, provided that the total amount of grout and reinforcement is concentrated at the wall ends.

5.5 CONCLUSIONS

This chapter addresses several key points: it discusses the numerical challenges faced in modeling three-dimensional PGRM wall structural systems, introduces a displacement-controlled pushover analysis strategy, examines the impact of orthogonal walls (flanges) on masonry buildings' behavior, and quantitatively assesses the influence of aspect ratio (number of storeys), axial load level, and vertical reinforcement arrangement (concentrated at the wall's end or distributed along the wall's length) through seismic performance parameters.

The issues encountered during the force-controlled pushover analysis highlight the difficulty in achieving convergence, indicating limitations in accurately capturing the complete nonlinear response. However, a displacement-controlled pushover analysis with a displacement distribution following the modal shape obtained by eigenanalysis provided a viable solution for overcoming the convergence issues associated with force-based analysis.

The comparison of fundamental periods, T , calculated through eigenanalysis and predicted by design standard equations, revealed consistent trends: taller buildings exhibit longer natural periods, reflecting reduced initial stiffness with increasing height. Notably, the equation from ASCE SEI 7 (2022) demonstrates the closest agreement with the eigenanalysis predictions across different building heights than NBCC (2020) and Eurocode 8 Part 1 (2004) equations.

The results of comparative analyses between configurations with and without the inclusion of orthogonal walls (flanges) reveal that flanges enhance the overall stiffness and significantly improve the lateral capacity, especially for multi-storey structural systems where flexural behavior has a significant role in the overall behavior. Pushover analyses showed that configurations with flanges exhibited up to 35% greater lateral strength compared to those without flanges, confirming the beneficial effects of interlocking walls for improved seismic performance.

The results show that higher axial load levels increase capacity but decrease ductility. In fact, reducing ductility may be undesirable. Incorporating an adequate number of load-bearing walls is beneficial to maintaining ductile behavior in the overall structural system. This approach helps keep pre-compression levels low, ensuring better structural performance. Stiffness degradation is more pronounced when the wall structural systems are subjected to lower pre-compression levels, rapidly decreasing in the initial stages and gradually declining. Besides, increasing the number of stories results in a reduction in the secant stiffness, K_{sec} , and an increase in the ultimate displacement, d_u . However, as this analysis is limited to buildings with up to six storeys, further research is needed to investigate potential height restrictions for PGRM buildings to prevent brittle failures.

The comparison between reinforcement concentrated at the wall's ends and uniformly distributed reinforcement shows that similar load capacity and stiffness are obtained in both cases. The percentage difference in efficiency detailing (measured by the product $V_{max} \cdot R$) between cases where the vertical reinforcement is distributed compared to the cases where the

vertical reinforcement is concentrated at the walls' ends ranges from -17.3% to 10.8%, indicating that the distinct details examined here exhibit similar efficiency. These findings indicate a potential for the detailing with vertical reinforcement concentrated at the wall ends to be included in design standards such as TMS 402/602 (2022) and CSA S304 (2014). Experimental comparisons of walls with identical dimensions and materials, differing only in vertical reinforcement distribution, would be pertinent to confirm these numerical results.

Comparisons among different approaches revealed significant differences in seismic performance parameters, highlighting the importance of accurate modeling for seismic design. The trilinear approach (TL) stands out for approximating the actual response of the walls, preserving three crucial characteristics: initial elastic stiffness, lateral force capacity, and post-peak phase behavior. In contrast, the bilinear approaches, although simpler, tend to be more conservative. Disregarding the post-peak stage when calculating the seismic performance factors (SPFs) results in a conservative design, but the structural systems' actual ductility is too undervalued.

Significant R values were obtained across all cases examined in this study, reflecting the good performance of PGRM wall structural systems under varied design conditions. These results underscore the suitability of PGRM walls for use in multi-storey buildings without irregularities and featuring in-plane rigid floors, particularly in regions with low to moderate seismic activity. However, it is essential to clarify that the SPFs obtained here are specific to the structural systems examined and do not directly reflect the overall response of actual buildings. Nonetheless, the findings presented herein represent a significant step toward achieving that goal. Further analyses considering different building archetypes are needed to establish reliable SPFs.

6. SEISMIC PERFORMANCE EVALUATION OF CONCRETE MASONRY BUILDINGS IN DIFFERENT REGIONAL CONTEXTS: CANADA VS. BRAZIL

6.1 INTRODUCTION

A critical consideration in seismic design is ensuring that the seismic force-resisting system (SFRS) can effectively dissipate energy, allowing structures to undergo ductile deformation under lateral loads well into the inelastic (plastic) range without experiencing sudden collapse (Brzev; Anderson, 2018). Concrete masonry walls, widely used in low to high-rise buildings (Parsekian; Medeiros; Sipp, 2018), are essential components of SFRS, resisting vertical loads from gravity and lateral forces from seismic and wind loads. Their widespread application underscores the importance of understanding their seismic performance at the system level to improve building design.

Masonry buildings can vary significantly in their structural layout configurations, incorporating walls with uniform or varied characteristics (e.g., cross-section, lengths, aspect ratios, axial load levels, and reinforcement detailing). These variations introduce complexity in seismic response, as the overall behavior of a building with interlocked walls often deviates from the simple sum of individual wall responses. Research by Heerema et al. (2015a; 2015b) and Noor-E-Khuda (2022) have demonstrated that the disparity in wall characteristics poses challenges in accurately quantifying the overall system-level inelastic response of the buildings, including lateral load capacity, ductility, stiffness degradation, and energy dissipation.

While substantial progress has been developed in understanding the nonlinear behavior of individual walls (Bolhassani et al., 2016a; 2016b; Calderón; Sandoval; Arnau, 2017; El-Dakhakhni; Banting; Miller, 2013; Elmapruk; ElGawady; Hassanli, 2020; Janaraj; Dhanasekar, 2016; Kasparik; Tait; El-Dakhakhni, 2014; Minaie et al., 2010; Pérez-Pinedo et al., 2024; Rahim et al., 2023; Sandoval; Calderón; Almazán, 2018; Voon; Ingham, 2006), the seismic response at the system level remains less explored, mainly due to the challenges of full-scale testing and computational modeling, despite significant contributions (Heerema et al., 2015a; Koutras; Shing, 2020; Minaie; Moon; Hamid, 2014; NIST, 2010; Noor-E-Khuda, 2022;

Palhares et al., 2025). At the individual wall level, factors such as cross-sectional area, material properties, reinforcement detailing, aspect ratios, and axial loads significantly influence the nonlinear behavior. When scaled to the building level, additional aspects—including layout configurations, wall-to-wall connections, slab diaphragm behavior, and axial load distribution—introduce further complexity, underscoring the limitations of summing isolated wall responses to assess the overall building performance.

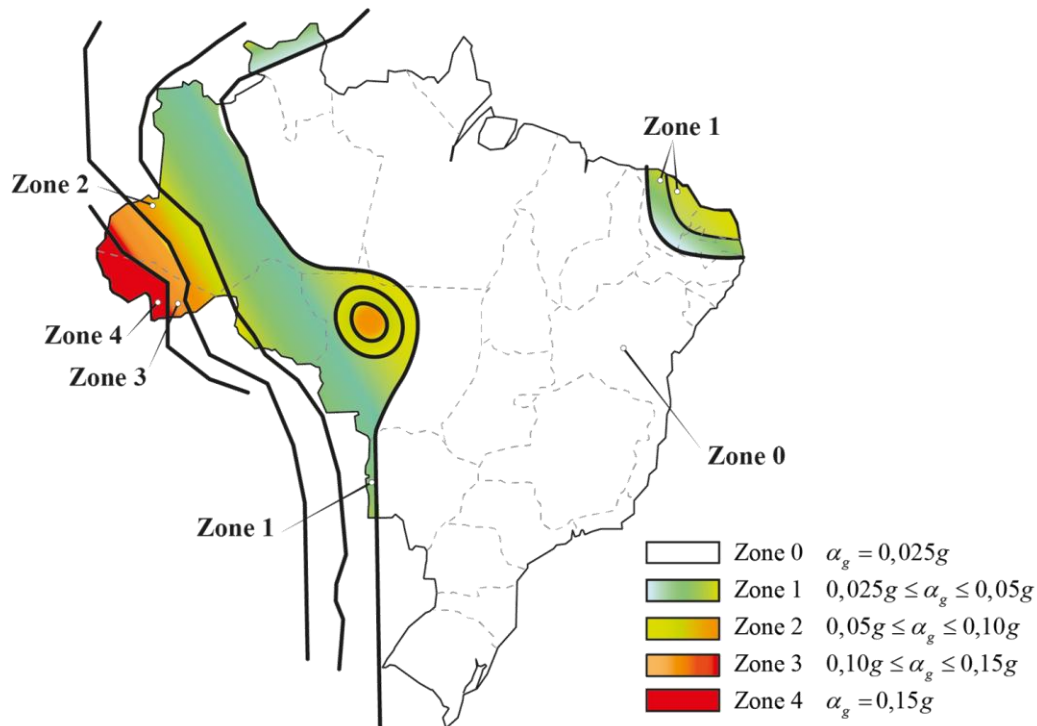
Due to the importance of the influence of structural layouts on seismic performance in real-world design scenarios, research on this topic requires further exploration. Morandi et al. (2022) investigated unreinforced masonry (URM) buildings in Italy and Germany, using a three-dimensional equivalent frame approach to derive the response modification factor, R . This modeling approach has limitations as it may oversimplify the complex and variable inelastic behavior of masonry structures under seismic loads by relying on predefined ultimate deformation limits. Three-dimensional effects related to wall-to-wall connections and the out-of-plane response of the walls are disregarded. This highlights the need for advanced modeling approaches to capture building seismic responses and allow accurate performance assessments.

This study addresses these research gaps by numerically assessing the seismic performance of two concrete masonry buildings, considering structural layouts from low- to mid-rise for each building. The two buildings selected represent typical cases of Canadian and Brazilian construction. A unified numerical modeling approach conceived and validated by Palhares et al. (2025) was employed.

In Brazilian standards, such as NBR 16868 (2020) and NBR 15421 (2023), explicit seismic detailing requirements for masonry walls are not specified. Furthermore, NBR 15421 (2023) exempts structures in Brazil's seismic Zone 0 — covering around 80% of the national territory (Figure 116) — from its seismic design requirements. This assumption is justified mainly by Brazil's location within a stable intraplate zone with low seismic activity compared to tectonically active regions. However, further studies and advanced analyses are essential to evaluate potential risks and improve the Brazilian standard's seismic design and detailing provisions, particularly for masonry structures and regions classified as higher seismic zones, to ensure sufficient safety margins. In contrast, the Canadian standard NBCC (2020) establishes specific usage restrictions for masonry buildings based on the seismic hazard of the region, which is determined by the spectral acceleration value, $S_a(0.2)$. Complementing these provisions, CSA S304 (2024) prescribes detailed reinforcement schemes and design requirements for different categories of masonry structures. Instead of a zoned seismic map like

Brazil's NBR 15421 (2023), the NBCC (2020) provides site-specific seismic hazard values for Canada based on latitude, longitude, and site classification.

Figure 116: Seismic zone map of Brazil based on NBR 15421 (2023).



Source: Author.

The two buildings evaluated in this study share a four-storey configuration but differ significantly in layout characteristics, reinforcement detailing, material properties, and design assumptions. Furthermore, these two building configurations are particularly significant because low- to mid-rise buildings generally experience higher seismic demands (spectral accelerations) associated with their short fundamental periods. Moreover, as noted by Palhares et al. (2025), increasing the number of stories tends to enhance the ductility of the structural system, thereby improving seismic performance. Additionally, taller buildings exhibit longer fundamental periods, reducing seismic demands.

This research employs a multi-step methodology to compare the seismic performance of these buildings comprehensively. First, key structural building characteristics, including slenderness ratios, wall densities, and axial load stress, are quantified to highlight structural layout differences. Eigenvalue analyses are performed for building heights ranging from one to ten stories, with their calculated fundamental periods compared to predictions from design standards (ASCE/SEI 7 (2022), NBCC (2020), and Eurocode 8 Part 1 (2004)).

Secondly, nonlinear static (pushover) analyses were carried out to evaluate lateral stiffness, ultimate force capacity, ductility, and damage mechanisms. Nonlinear analyses, whether static or dynamic, are essential for assessing the seismic capacity and, consequently, the building's performance when subjected to a specific hazard (demand). Pushover analyses are widely used to evaluate the nonlinear behavior of structures subjected to lateral loads, such as those induced by seismic events (Aşikoğlu; Vasconcelos; Lourenço, 2021; Cattari et al., 2022; Easa; Yan, 2019; Endo; Pelà; Roca, 2017; Kuria; Kegyes-Brassai, 2023; Rosin et al., 2018; Wang; Martinez-Vazquez; Zhao, 2020). This type of analysis is covered by several building standards, such as Eurocode 8 (2004) and NTC (2018), as well as seismic guidelines like FEMA 440 (2013).

In the sequence, this study focuses on the evaluation of the force reduction factors for linear seismic design: overstrength factor, Ω (denoted as R_o by the NBCC (2020)); ductility-related force modification factor, R_d ; and response force modification factor, R (defined as the product of R_d and Ω). By using appropriate values of R , it is possible to adopt the results of elastic analyses for linear seismic design that consider, in a simplified manner, the inelastic dissipative capacity of structural systems, as prescribed in design standards (ASCE, 2022; EN 1998-1, 2004; NBCC, 2020; NTC, 2018). The main challenge is to accurately quantify the R values for balancing safety and economic efficiency in building design. Using energy-equivalent idealized responses derived from pushover curves, response modification factors, R , overstrength factors, Ω , and ductility-related force modification factors, R_d , are estimated.

The maximum attainable spectral accelerations, $S(T)_{max}$, derived from pushover curves, are compared against design spectra for seismic zones in Canada and Brazil. To further investigate system-level seismic performance, the Capacity Spectrum Method (CSM), originally developed by Freeman (1998), is applied to provide an additional perspective for comparing the seismic capacity and demand of each building in different regions.

By integrating deterministic and probabilistic analyses, this research aims to advance the understanding of the behavior of concrete masonry buildings, providing valuable insights into how variations in structural layout configurations influence seismic performance.

6.2 CHARACTERISTICS OF THE BENCHMARK BUILDINGS

Brazil and Canada have different seismic conditions. Brazil is characterized by relatively low seismic activity, while Canada experiences moderate to high seismicity. Thus, notable differences in construction practices, detailing, and building structural layouts are

expected. The two buildings selected for this study, B1 in Canada and B2 in Brazil, are actual concrete masonry buildings that represent the typical construction practices in their respective regions. The Canadian building, B1, incorporates reinforcement detailing in compliance with stricter seismic design requirements and features a structural layout with low density of walls, primarily aligned in one direction, not interlocked, and large-span one-way precast concrete slabs. In contrast, the Brazilian building, B2, does not include seismic-specific detailing. However, it features a structural layout with high density of walls in both principal directions, orthogonally interlocked, and small-span two-way reinforced concrete slabs.

6.2.1 Building located in Canada (B1)

The selected structural layout is a typical concrete masonry building in Atlantic Canada, previously studied by Smith et al. (2020) for cost-analysis purposes. The details of the structural layout are presented in Figure 117. The exterior and corridor walls are non-loadbearing (typically not constructed of masonry) and were not represented in the structural layout. The analyses included their self-weight as mass and gravity load.

The building has four storeys, each storey comprising 14 masonry courses plus a 200 mm slab, with the 14th course as a bond beam. This configuration results in a total building height, H , of 12 m. The base floor area, A_b , is 1815.2 m², and the external perimeter, P_e , is 218.4 m. The maximum and minimum floor plan widths, $B_{\max}$ and $B_{\min}$, are 84.8 m and 22.0 m, respectively, corresponding to maximum and minimum slenderness ratios, $\lambda_{\max}$ and $\lambda_{\min}$, of 0.55 and 0.14.

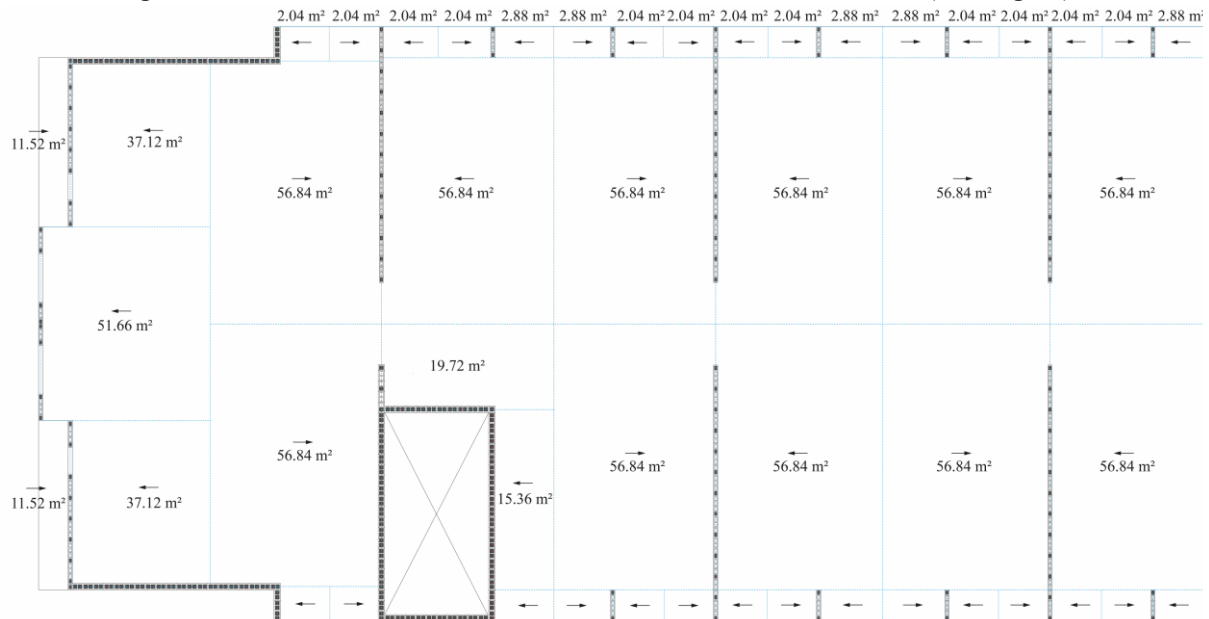
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wall W6 (comprising four corner walls) contributes to the in-plane shear resistance of the building. In contrast, the Y direction features a greater number of walls, which likely provides stronger resistance to lateral forces along that axis. The total lengths of shear walls in the X and Y directions, L_x and L_y , are 32 m and 184 m, respectively. The web area of the walls (net effective cross-section) in the two directions is $A_{w,x} = 9.6 \text{ m}^2$ and $A_{w,y} = 20.93 \text{ m}^2$, resulting in a total shear wall web area of $A_{w,total} = 30.53 \text{ m}^2$.

Two types of concrete masonry units (CMUs) were used in this building: $200 \times 190 \times 390 \text{ mm}^3$ (width x height x length) with a block nominal compressive strength, f_b , of 25 MPa (walls W1 and W2) and $300 \times 190 \times 390 \text{ mm}^3$ units with f_b of 30 MPa (walls W6 and W7).

The seismic weight per storey, W , is 13703 kN, corresponding to a total building weight, W_{total} , of 54812 kN. The maximum and minimum structural wall gravity load per floor, g_{max} , and g_{min} , are 78.30 kN/m (W3) and 17.14 kN/m (W7), respectively, corresponding to maximum and minimum axial load stresses, $\sigma_{v,max}$ and $\sigma_{v,min}$, of 3.0 MPa and 0.23 MPa. The vertical loads from the slabs were calculated according to the influence area shown in Figure 118. Note that the one-way slabs are in the same direction for the entire floor plan (Figure 117). Only the dead load (DL) specified in the design, $DL = 6.4 \text{ kN/m}^2$ (including the self-weight of the slabs and a partition load allowance, such as that for non-loadbearing walls), along with the self-weight of the structural walls, is considered. The flooring system comprises precast prestressed Hollow Core Slabs (HCS), considered rigid diaphragms. This hypothesis simplifies the model by considering the constraints of the top walls (rigid diaphragm model).

Figure 118: Influence area for vertical load distribution on walls from slabs (building B1).

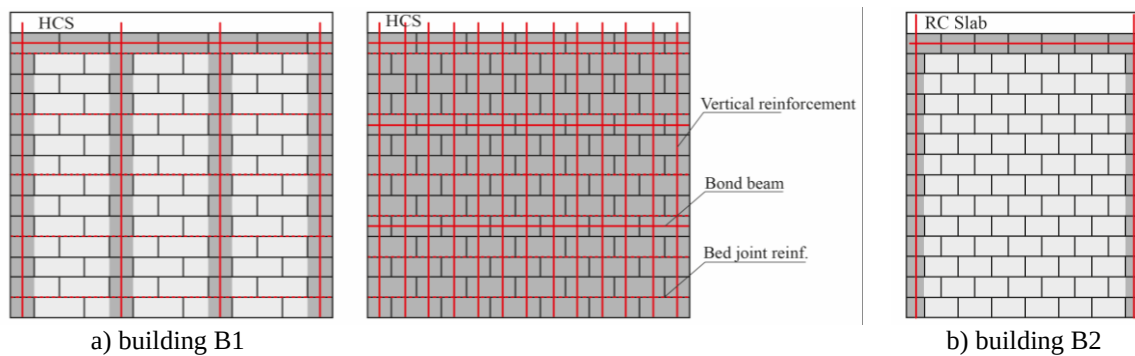


Source: Author.

The building was designed following the requirements of NBCC (2010), which was the applicable building standard at the time. The equivalent static force method was adopted for the seismic design, using force modification factors of $R_d = 1.5$ and $R_0 = 1.5$, as specified in Table 4.1.8.9 of NBCC (2010) for conventional wall detailing. It should be noted that these seismic parameters are retained in the latest version of the standard, NBCC (2020). The building location was Moncton, New Brunswick, with an assumed site class of C, corresponding to dense soil or soft rock conditions. The vertical reinforcement ratios in the X and Y directions, $\rho_{\text{reinf},x}$ and $\rho_{\text{reinf},y}$, are 0.82% and 0.30%, respectively. Vertical reinforcement consisted of 25 mm steel bars (W6, W7) and 16 mm bars (W1, W2, W3, W4). Horizontal reinforcement (bond beams) consists of 25 mm, 19.5 mm, and 16 mm bars for W6 and W7, with only 16 mm bars for W1, W2, W3, and W4. Bed joint reinforcement of 4.75 and 3.7 mm is used for W6 and W7, while 3.7 mm is adopted for W1, W2, W3 and W4. The lintel detailing above the door openings in walls W1 and W2 includes stirrups with a diameter of 11.3 mm.

The complete detailing of all the walls is provided in Figure C.1 to C.7 (Appendix C). A schematic illustration showing the pattern of wall detailing is shown in Figure 119. It is important to note that, in building B1, some walls are partially grouted reinforced masonry (PGRM), while others are fully grouted reinforced masonry (FGRM); for instance, walls W6 and W7 are FGRM in the first three storeys and PGRM in the top storey (see Appendix C).

Figure 119: Schematic illustration of reinforcement detailing on the walls of the buildings.



Source: Author.

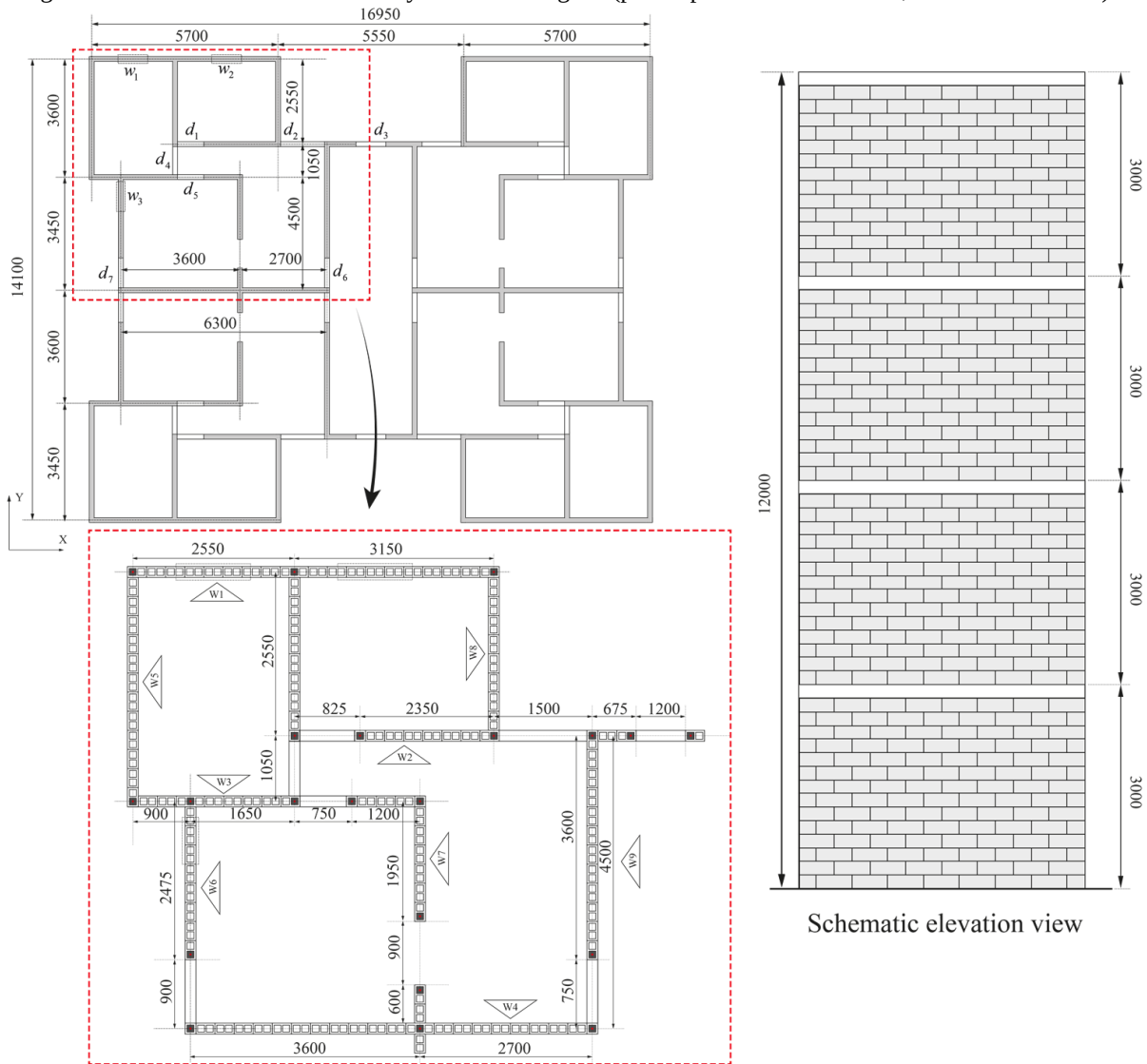
6.2.2 Building located in Brazil (B2)

The selected structural layout is a typical concrete masonry building in Brazil. The details of the structural layout are presented in Figure 120.

The building has four storeys, each consisting of 14 masonry courses, with the 14th course as a bond beam and an assumed 200 mm RC slab contributing to the total height, H , of 12 m, similar to building B1. A slab thickness of 100 mm was used for self-weight calculations. The base floor area, A_b , is 207.2 m², and the external perimeter, P_e , is 62.4 m. The maximum and minimum floor plan widths, $B_{\max}$ and $B_{\min}$, are 17.1 m and 14.2 m, respectively, corresponding to maximum and minimum slenderness ratios, $\lambda_{\max}$ and $\lambda_{\min}$, of 0.84 and 0.70.

As shown in Figure 120, building B2 has an “H” shape with well-distributed masonry walls in both principal directions, a structural layout that is advantageous for resisting lateral loads. This uniform distribution of walls in both directions is beneficial, providing balanced resistance to lateral forces. The total lengths of shear walls in the X and Y directions, L_x and L_y , are 63.6 m and 69.6 m, respectively. The web area of the walls (net effective cross-section) in each direction is $A_{w,x} = 6.6$ m² and $A_{w,y} = 4.1$ m², resulting in a total shear wall web area of $A_{w,total} = 10.66$ m².

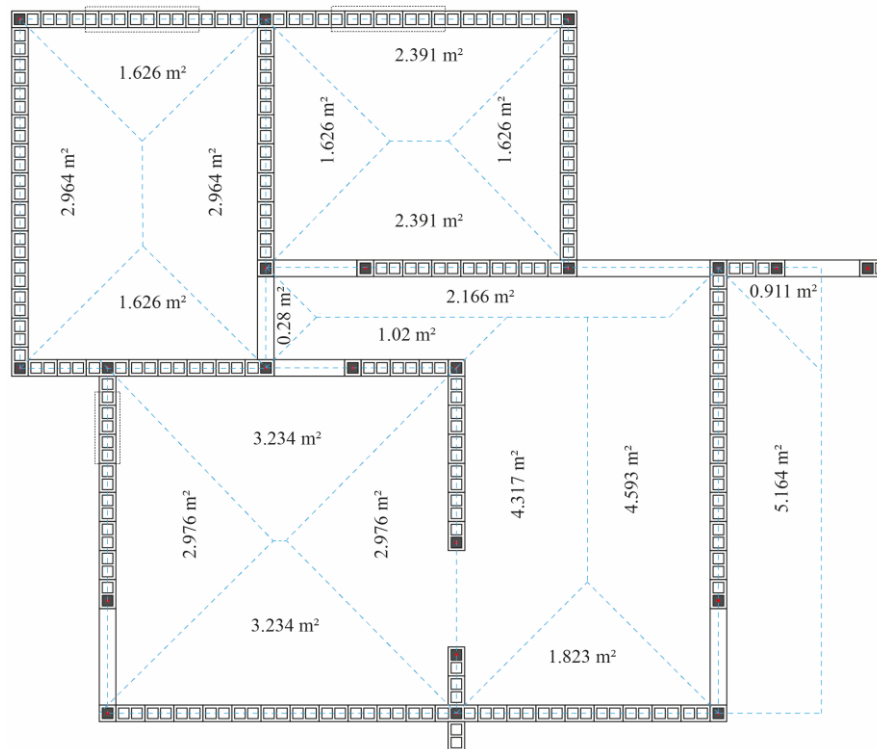
Figure 120: Details of the structural layout of building B2 (plan repeated on four floors, dimensions in mm).



Source: Author.

The concrete masonry units (CMUs) measure $140 \times 190 \times 290 \text{ mm}^3$ (width x height x length), which is typical in masonry buildings in Brazil. The seismic weight per storey, W , is 1326 kN, corresponding to a total building weight, W_{total} , of 5304 kN. The maximum and minimum structural wall gravity load per floor, g_{max} , and g_{min} , are 14.15 kN/m (W7) and 7.86 kN/m (W1), respectively, corresponding to maximum and minimum axial load stresses, $\sigma_{v,max}$ and $\sigma_{v,min}$, of 0.86 MPa and 0.58 MPa. The flooring system comprises RC slabs, and the vertical loads from the slabs were calculated according to the influence area shown in Figure 121 following the yield line method. The inclination of the yield lines can vary between 30° and 60° , depending on the boundary conditions of each slab panel. For simplification, an inclination of 45° was adopted for all slabs. A dead load (DL) of 3.5 kN/m^2 is considered, comprising the self-weight of the slab plus floor covering, in addition to the self-weight of the walls.

Figure 121: Influence area for vertical load distribution on walls from slabs (building B2).



Source: Author.

The building B2 does not have specified seismic design considerations and detailing, representing a building located in Zone 0 of Brazil's seismic regions, which covers most of the country and exempts buildings from seismic design according to NBR 15421 (2023). Vertical rebars at wall ends, intersections, tops, and around openings, is commonly used as a good construction practice. The building has very low vertical reinforcement ratios in the X and Y directions, $\rho_{\text{reinf},x}$ and $\rho_{\text{reinf},y}$, at approximately 0.03%. Vertical and horizontal reinforcements are composed of 10 mm steel bars. Although this building could be classified as unreinforced masonry (URM), it is defined here as partially grouted reinforced masonry (PGRM) due to the inclusion of reinforcement in the analyses. The complete wall detailing is provided in Figure C.7 to C.16 (Appendix C). A schematic illustrating the pattern of wall detailing is shown in Figure 119b.

6.2.3 Comparison between the buildings

This section compares the structural layouts of the four-storey (mid-rise) buildings in Canada (B1) and Brazil (B2), focusing on building layout parameters. These include slenderness ratio (H/B), λ ; compactness ratio (or form factor), C_r , calculated by Equation 31; percentage of web wall area (net effective cross-section) relative to the floor area (A_w/A_b), ρ_w ; average wall length per floor area (L/A_b), ρl_w ; average gravity load per storey floor area (W/A_b),

g ; average axial stress ($W_{total}/A_{w,total}$), σ ; and vertical reinforcement ratio ($A_s/A_{w,total}$), ρ_{reinf} . The main objective is to identify and quantify more clearly the differences between the two buildings through percentage comparison to understand each structural layout's characteristics.

$$C_r = \frac{2 \cdot \sqrt{A_b \cdot \pi}}{P_e} \quad \text{Eq. 31}$$

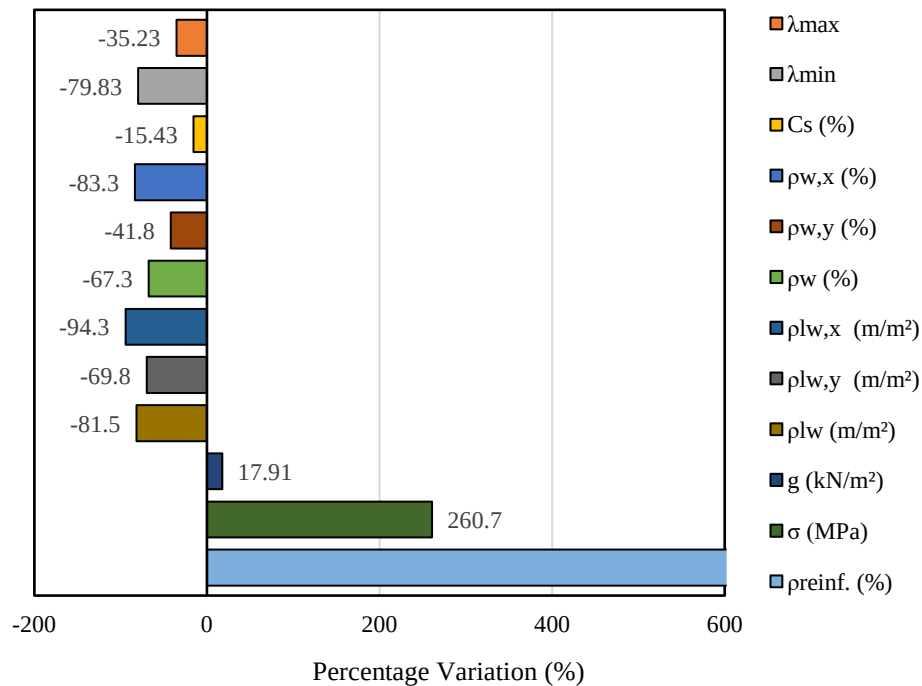
A summary of the buildings' characteristics is described in Table 18, and a percentage variation in layout parameters comparing B1 versus B2 is shown in Figure 122.

Table 18: Comparison of layout parameters between buildings B1 and B2.

Build.	λ_{max}	λ_{min}	C_r (%)	$\rho_{w,x}$ (%)	$\rho_{w,y}$ (%)	ρ_w (%)	$\rho l_{w,x}$ (m/m ²)	$\rho l_{w,y}$ (m/m ²)	ρl_w (m/m ²)	g (kN/m ²)	σ (MPa)	$\rho_{reinf.}$ (%)
B1	0.55	0.14	69.1	0.53	1.15	1.68	0.02	0.10	0.12	7.55	1.80	1.11
B2	0.84	0.70	81.7	3.17	1.98	5.15	0.31	0.34	0.64	6.40	0.50	0.05

Source: Author.

Figure 122: Percentage variation of layout parameters for building B1 versus building B2.



Source: Author.

The comparison between the Canadian (B1) and Brazilian (B2) buildings reveals significant differences in layout parameters. The slenderness ratios for B1 are lower than those for B2, with a percentage variation of -35.23% and -79.83% for λ_{max} and λ_{min} , respectively. The compactness ratio indicates how far the building layout is from a circular area, which could establish a better balance between the external perimeter and the floor area. The closer the factor is to 100%, the closer the floor is to a circular shape. In B1, this factor is lower than in B2 (-15.43%), reflecting a more elongated shape. The more compact structural layout of B2 suggests

better floor area utilization, which could contribute to material efficiency, considering both structural and environmental aspects.

In building B1, the shear walls are longer in the Y direction (184.0 m) than in the X direction (32.0 m), whereas building B2 has a more balanced wall length in the two principal directions (63.6 m in the X-axis and 69.6 m in the Y-axis).

The walls in building B1 are notably longer than those in building B2. However, B1 featured a remarkably lower average wall length per floor square area, p_{lw} , (-81.5%). This suggests that building B1 must be designed to support a higher gravity load, as confirmed by the 17.91% higher average gravity load per floor area and 260.7% higher average axial load stress for B1 than B2. Consequently, building B1 requires higher axial (compression) loaded walls and masonry compressive strength, f_m , to meet the compressive design requirements.

Building B2 has a higher percentage of web wall area (net area) related to the floor area, ρ_w , (5.15% for B2 versus 1.68% for B1). This substantial difference reflects the higher density of walls in building B2 (structural layout with greater internal compartmentalization), compensating for the lateral stability given its higher slenderness ratio. From a structural point of view, this has the advantage of distributing gravity loads more evenly across the walls, keeping axial loads lower compared to building B1. Walls with higher axial load are usually less ductile (Davidson, 1996; Medeiros et al., 2022a; Palhares et al., 2023, 2024, 2025). However, the layout of B1 offers significant architectural flexibility, as it minimizes the number of internal walls and allows for larger open spaces.

The vertical reinforcement rate is significantly higher in B1 than in B2, with a difference of 2035%. The building B1 is designed to meet strict specifications for seismic detailing. In contrast, the building B2 does not consider seismic design; reinforcement is only included to meet good construction practices.

Interlocked walls are stiffer than non-interlocked walls, as part of the flange can contribute to the wall stiffness (Palhares et al., 2025). Usually, only the wall web is considered when calculating its stiffness in non-interlocked walls. Also, the distributed vertical load is lower in interlocked walls than in non-interlocked ones. For the same layout, interlocked walls (as in B2) should provide greater lateral stability to the building than non-interlocked walls (as in B1).

The comparison reveals that for building B1, designed for moderate seismic activity, the structural conception is with fewer shear walls but with a heavy reinforcement ratio. In

contrast, building B2, where seismic detailing is not the primary concern, features densely distributed masonry walls in both principal directions. This structural layout comparison emphasizes the potential for learning when considering a cross-regional approach; for example, incorporating some aspects of the Brazilian structural layout into Canadian designs could improve building stability and material efficiency.

6.3 DESCRIPTION OF THE NUMERICAL MODELING

A unified modeling approach validated by Palhares et al. (2024) for straight concrete masonry walls is adopted here to model the actual buildings previously presented. The approach was already extended to flanged walls using a three-dimensional model and applied in three-dimensional structural systems by Palhares et al. (2025).

Following the strategy outlined by Palhares et al. (2025), the numerical building models were developed using the finite element analysis software DIANA[®] (DIANA FEA, 2022). For the ungrouted masonry (UGM) and grouted masonry (GM) regions, the CQ40S elements were used. The mortar joints in UGM regions were modeled considering the interface elements (CL24I – DIANA (DIANA FEA, 2022)), with interface elements considered only in the walls aligned with the loading direction, where displacements are imposed.

A rigid diaphragm assumption was adopted for the slabs by constraining the walls at each floor level. A master node was defined at the centroid of each floor at the slab level, which was tied to the top nodes of the walls at the corresponding floor level, ensuring they shared the same displacement in the X-Y plane to simulate the rigid diaphragm. The building base nodes were constrained horizontally and vertically, excluding any soil-structure interaction effects in the building response (Figure 123 and Figure 124). For large structures located in soft soil locations, the deformation of the soil may influence the structure response (Brzev; Anderson, 2018).

The material properties and corresponding references, including those for UGM, GM, and interfaces, used in both building models, are outlined in Table 19. For the building B1, the masonry compressive strength, f_m , for UGM and GM regions, was determined from CSA S304 (2014) according to the block strength f_b , specified in the design. For the building B2, the compressive strength, f_m , of both UGM and GM was specified as 6.4 MPa, derived from a practical rule commonly used in the conception of Brazilian masonry buildings, which estimates 1.6 MPa per storey in the net cross-sectional area. The Young's modulus, E , tensile strength, f_t , and fracture energies in compression, G_c , and tension, G_f , were calculated as

functions of the compressive strength. The Total Strain Crack Model (TSCM) in DIANA (DIANA FEA, 2022) was adopted with a compressive strength reduction model due to lateral cracking applied according to the Vecchio and Collins 1993 model (Vecchio; Collins, 1993), using a lower bound reduction curve set at 0.6.

Table 19: Material properties adopted for building models.

Parameter	Building						Reference for the adopted value
	CAN build. (B1)			BRA build. (B2)			
	UGM*	GM*	Interf.	UGM	GM	Interf.	
Compressive strength (f_m) N/mm ²	15.0 / 17.5	12.5 / 13.5	—	6.4	6.4	—	CSA S304 (2014) (B1) Specified (B2)
Young's modulus (E) N/mm ²	13500 / 15750	10625 / 11475	—	5120	5120	—	$E = 900 \cdot f_m$ (B1) (TMS, 2022) $E = 800 \cdot f_m$ (B2) (ABNT, 2020)
Compressive fracture energy (G_c) N/mm	20.6 / 21.4	21.1 / 22.8	—	17.6	10.8	—	$G_c = 15 + 0.43 \cdot f_m - 0.0036 \cdot f_m^2$ (UGM) (Magenes et al., 2008) $G_c = 1.69 \cdot f_m$ (GM) (Calderón et al., 2021d)
Tensile strength (f_t) N/mm ²	1.28 / 1.38	1.17 / 1.21	—	0.84	0.84	—	$f_t = 0.33 \cdot \sqrt{f_m}$ (Wong et al., 2013)
Tensile fracture energy (G_f) N/mm	0118 / 0.122	0.115 / 0.117	—	0.102	0.102	—	$G_f = 73 \cdot f_m^{0.18}$ (fib Model Code 2010, 2013)
Normal stiffness (k_{nn}) N/mm ²	—	—	1350	—	—	512	$k_n = \frac{E}{h_{int}}$ (Arnau et al., 2015; Calderón et al., 2021d)
Shear stiffness (k_{tt}) N/mm ²	—	—	548.8	—	—	234.15	$k_t = \frac{E}{2 \cdot (1 + \mu) \cdot h_{int}}$ (Arnau et al., 2015; Calderón et al., 2021d)
Cohesion (c) N/mm ²	—	—	0.15	—	—	0.15	Test (Pettit, 2023; Rahim, 2023)
Friction angle (φ) deg.	—	—	44.7	—	—	44.7	Test (Pettit, 2023; Rahim, 2023)
Tensile strength (f_{ti}) N/mm ²	—	—	0.10	—	—	0.10	Recommendation (Lourenço; Gaetani, 2022)

* Values are based on CMU specifications: 200 mm thickness ($f_b = 25\text{MPa}$) / 300 mm thickness ($f_b = 30\text{MPa}$)

Source: Author.

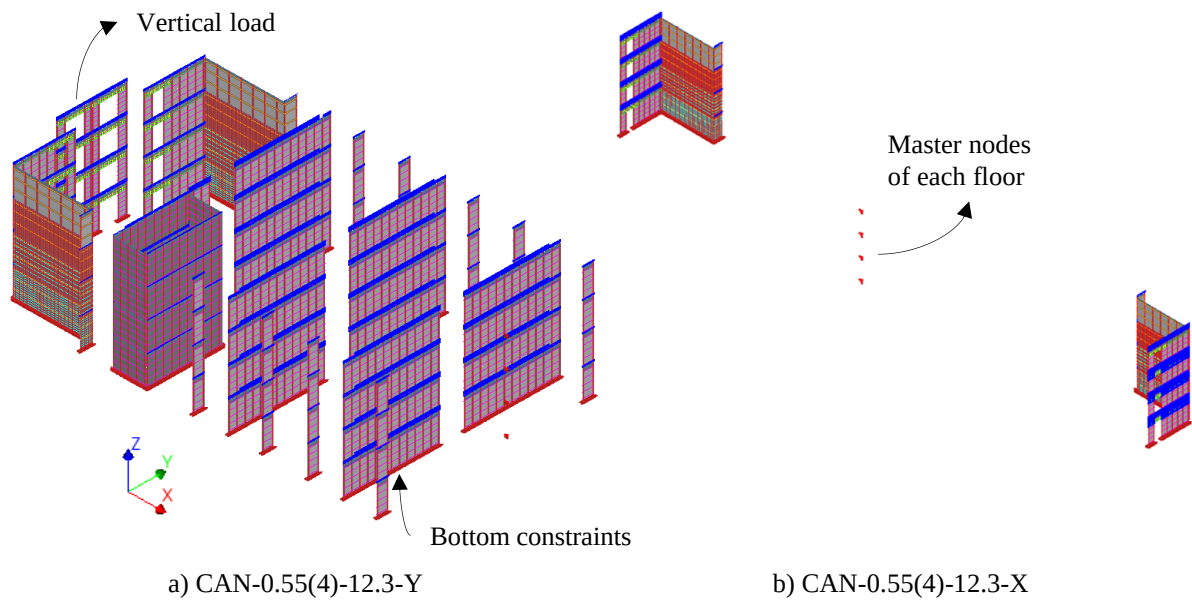
Cohesion, c , and friction angle, ϕ , for the interfaces were defined based on shear triplet test results on hollow concrete blocks (Pettit, 2023; Rahim, 2023). The tensile strength of interfaces, f_{ti} , was assumed to be 0.10 MPa, according to Lourenço and Gaetani (2022). The normal and shear stiffness components were calculated based on the elastic properties of UGM and a mortar joint height, h_{int} , assumed to be equal to 10 mm. A Poisson's ratio, ν , of 0.23 was adopted based on experimental tests on hollow concrete blocks (Oan, 2013). Steel reinforcement was defined with yield strength, f_y , of 400 MPa and Young's modulus, E , of 210 GPa. The slabs are not directly modeled in the numerical models, as a rigid diaphragm assumption is adopted; only the portion between the walls of each storey is included, with a thickness equal to the block thickness of the walls and defined as a linear elastic material with $E = 28000$ MPa.

To facilitate the identification of numerical models, a nomenclature based on essential characteristics of the building was employed as follows: location of the building (CAN for Canada and BRA for Brazil) – building slenderness ratio (number of storeys in parenthesis) – the percentage of average axial stress related to the average masonry compressive strength – direction of the analyses (X or Y direction). For example, CAN-0.55(4)-12.3-Y denotes building B1, located in Canada, with a λ_{max} of 0.55 (corresponds to four storeys), under average axial stress corresponding to 12.3% of average f_m , and analyzed in the Y direction.

6.3.1 Canadian building (B1)

To optimize computational efficiency, only half of the building was modeled to take advantage of the symmetry of the structural layout. Two models were created: one for the Y direction and the other for the X direction. In the X-axis, the seismic design was primarily governed by the couple of walls W6. The numerical model also included the wall flanges (composing the "Z" cross-section). The numerical models of the building B1 for each direction are depicted in Figure 123.

Figure 123: Numerical models of building B1.



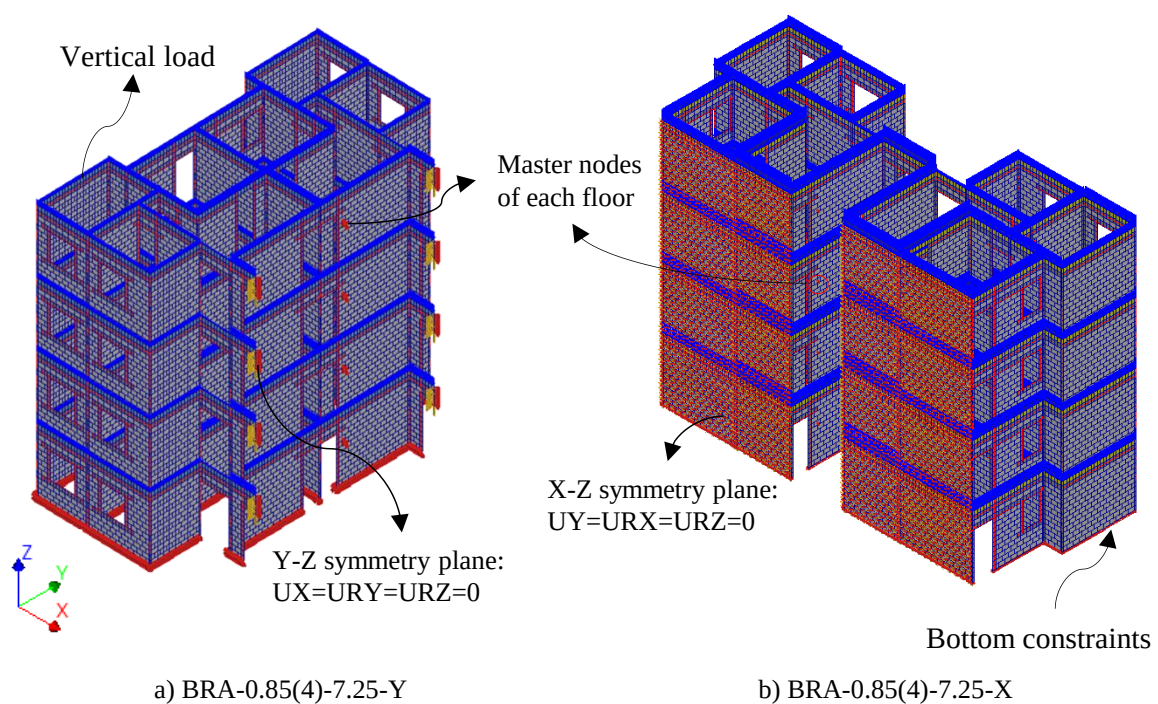
Source: Author.

The mesh was set to half-block size by shell elements measuring $200 \times 200 \text{ mm}^2$. The elements represent the net effective cross-section of the corresponding wall regions. Based on the block type, a thickness of 200 mm or 300 mm was assigned for the GM regions, while a thickness of 70 mm was used for the UGM regions, equivalent to the thickness of the face shells added together. The final 3D numerical models of the building consisted of 60,697 elements and 201,280 nodes for the Y direction and 10,572 elements and 42,224 nodes for the X direction.

6.3.2 Brazilian building (B2)

The same symmetry approach was used to model only half of the building. Two models were created: one for the Y direction and the other for the X direction. The numerical models of the building B2 for each direction are depicted in Figure 124. Symmetry conditions were imposed on the Y-Z symmetry plane ($U_X=U_{RY}=U_{RZ}=0$) for the model simulating the translational behavior in the Y direction, and on the X-Z symmetry plane ($U_Y=U_{RX}=U_{RZ}=0$) for the model simulating the translational behavior in the X direction.

Figure 124: Numerical models of building B2.



Source: Author.

The mesh was set to half-block size, with each cell of the blocks represented by shell elements measuring $150 \times 200 \text{ mm}^2$. The elements represent the net effective cross-section of the corresponding wall regions. A thickness of 140 mm was assigned for the GM regions, while a thickness of 50 mm was used for the UGM regions, equivalent to the thickness of the face shells added together. The final 3D numerical model of the building B2 consisted of 47,292 elements and 137,840 nodes for the Y direction and 49,252 elements and 144,188 nodes for the X direction.

6.4 NUMERICAL ANALYSES

This section presents the numerical analyses conducted on the Canadian (B1) and Brazilian (B2) buildings, including eigenanalysis and nonlinear static (pushover) analysis. The eigenanalysis examines the fundamental periods, T , and mode shapes for building heights ranging from one to ten stories, with calculated fundamental periods compared to predictions from design standards (ASCE/SEI 7 (2022), NBCC (2020), and Eurocode 8 (2004) (2004)). The pushover analysis evaluates key parameters such as lateral force-resisting capacity, V_{max} , ultimate displacement, d_u , lateral stiffness, K_{sec} , energy dissipation, U , lateral stress distribution, τ , and base shear coefficient, C_h . Together, these analyses provide a comprehensive understanding of the buildings' lateral behavior, emphasizing the impact of structural layout.

6.4.1 Eigenanalyses

The natural period, T , is a crucial parameter for seismic analysis of buildings, as it provides insights into their dynamic characteristics. In linear elastic force-based seismic design, accurately predicting T is essential, as it directly influences the calculation of seismic forces derived from the spectral acceleration, which is used to verify the building's capacity against earthquakes. Research has shown that T is influenced by factors such as building height, material properties, and structural layout, as demonstrated in extensive studies on RC and URM buildings (Crowley; Pinho, 2004; Crowley; Pinho, 2010; Gallipoli et al., 2023; Kaplan; Guney; Dogangun, 2021; Michel et al., 2010; Shakya et al., 2016). However, the literature on the topic of concrete reinforced masonry (RM) buildings is comparatively limited, highlighting a knowledge gap that warrants further investigation.

In this study, a series of eigenvalue analyses were carried out for both buildings (B1 and B2), varying the number of storeys (one, two, four, six, eight, and ten storeys) to compute the global translational vibration modes and the respective T .

For taller buildings, higher compressive strengths, f_m , are expected at the lower stories, reflecting the increased gravity load demands. The Young's modulus, E , a key factor in eigenanalysis, was calculated as a function of the compressive strength, f_m , with $E = 900 \cdot f_m$, according to TMS 402/602 (2022), for the building B1, and $E = 800 \cdot f_m$, as specified by ABNT NBR 16868 (2020), for building B2. To account for varying stiffness along the building height, E values were incrementally adjusted to reflect changes in stiffness as the number of stories increased.

For building B1, the adjustments were based on the properties of the four-storey building design. For buildings exceeding four stories, E was adjusted every two stories, proportionally to the total number of stories relative to the four-storey model. For instance, in the ten-storey configuration, the first two stories had E values scaled by a factor of 10/4. The next two stories (3–4), the E was adjusted by 8/4, and the following two stories (5–6) a factor of 6/4 was used. The upper stories (7–10) retained the original E values from the four-storey building design.

For building B2, f_m was derived using a practical rule commonly applied in Brazilian masonry design, which correlates f_m to the number of stories. Accordingly, E was varied along the building height based on incremental increases in f_m . The f_m values were increased by 1.6 MPa per storey, with adjustments made every two storeys. For instance, in the ten-storey

building, the first two floors have $f_m = 16$ MPa and $E = 12800$ MPa; the third and fourth floors have f_m equal to 12.8 MPa and E equal to 10240 MPa; the fifth and sixth floors have $f_m = 9.6$ MPa and $E = 7680$ MPa; and the top four floors have $f_m = 6.4$ MPa and $E = 5120$ MPa. Table 20 summarizes the E values used for both buildings.

Table 20: Young's Modulus, E , distribution across stories for buildings B1 and B2.

Build.	Masonry	E (MPa) by storey range			
		1-2	3-4	5-6	7-10
B1	UGM	33750	27000	20250	13500 / 15750 *
	GM*	26563 / 28688 *	21250 / 22590 *	15938 / 17213 *	10625 / 11475 *
B2	UGM	12800	10240	7680	5120
	GM	12800	10240	7680	5120

* Values are based on CMU specifications: 200 mm thickness ($f_b = 25$ MPa) / 300 mm thickness ($f_b = 30$ MPa).

Source: Author.

To determine the vibration modes, the modeling strategy that concentrates each storey's mass on its respective master node was defined. The mass comprised the self-weight of walls and the vertical load from the slabs (half of the building), resulting in an estimated mass of 698,670 kg (698.67 tons) per floor for building B1 and an estimated mass of 67,630 kg (67.63 tons) per floor for building B2.

The results were compared against the values predicted by analytical equations from ASCE/SEI 7 (2022), NBCC (2020), and Eurocode 8 Part 1 (2004) seismic design standards, which are presented in Equations 32, 34, and 35, respectively. The equations are intended to predict the fundamental period of buildings with masonry walls. Notably, the equation presented in NBR 15421 (2023) aligns with that of NBCC (2020), reflecting similar methodologies for estimating the fundamental period. The standard-based equations are intended to offer quick estimates that align with more detailed methods, such as eigenvalue analysis of finite element models (FEM). However, discrepancies between the analytical estimates and FEM results can significantly impact seismic design outcomes: an overestimated T may lead to underestimated seismic forces, potentially compromising structural safety.

$$T = \frac{C_q}{\sqrt{C_w}} \cdot h \quad \text{Eq. 32}$$

where h is the height (m) of the structure above the base, $C_q = 0.006233$ m, and C_w is calculated from Equation 33.

$$C_w = \frac{100}{A_B} \sum_{i=1}^x \frac{A_i}{\left[1 + 0.83 \left(\frac{h}{l_{wi}}\right)^2\right]} \quad \text{Eq. 33}$$

where A_B is the area of the base of the structure (m²); A_i is the web area of the shear wall i (m²); l_{wi} is the length of the shear wall i (m); x is the number of shear walls in the building effective in resisting lateral forces in the direction under consideration.

$$T = 0.05 \cdot h^{0.75} \quad \text{Eq. 34}$$

$$T = C_t \cdot h^{3/4} \quad \text{Eq. 35}$$

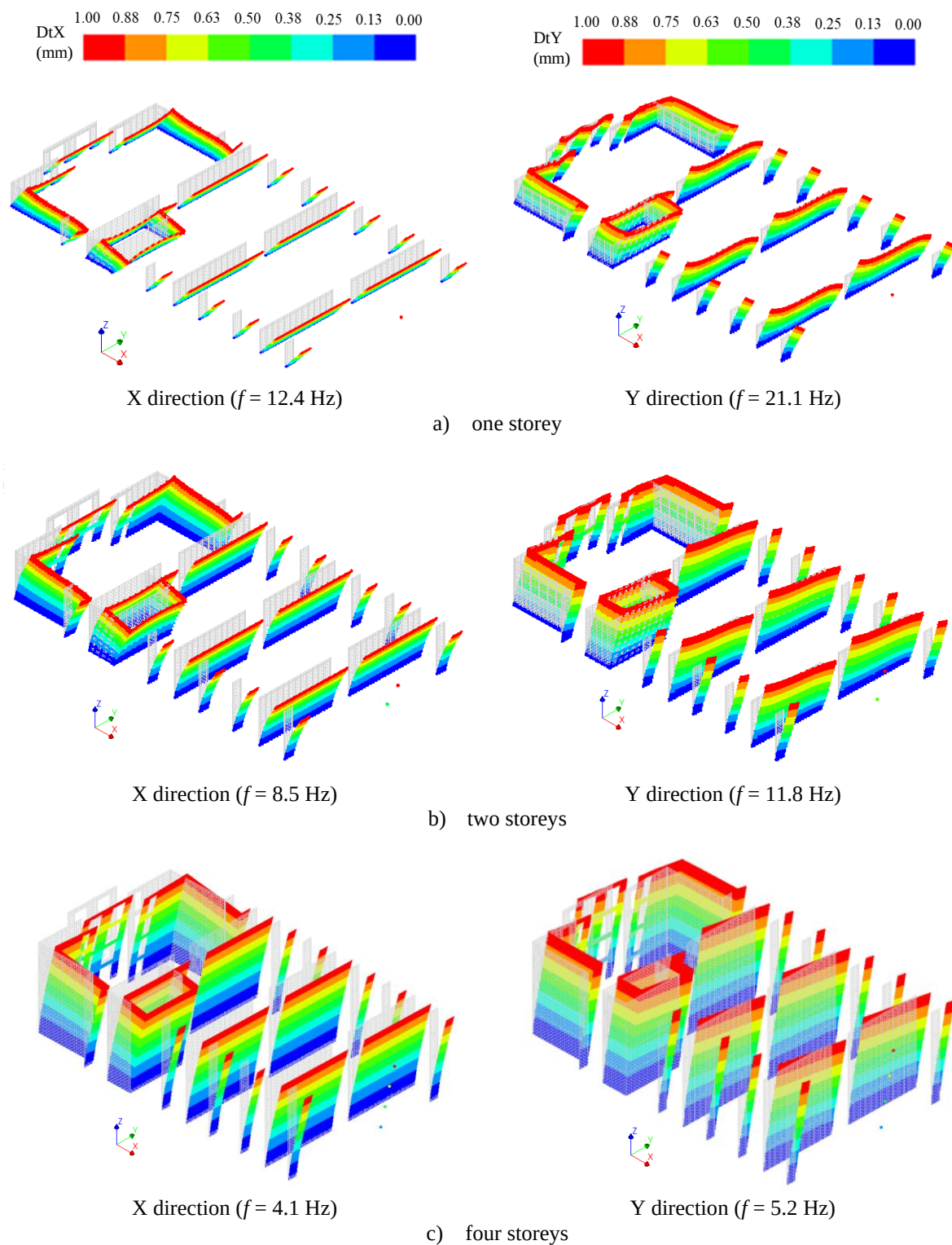
where $C_t = 0.075/\sqrt{A_c}$ and A_c is calculated from Equation 36.

$$A_c = \sum \left[A_i \cdot \left(0.2 + \frac{l_{wi}}{h}\right)^2 \right] \quad \text{Eq. 36}$$

where: A_c is the total effective area of the shear walls in the first storey of the building (m²). Also, l_{wi}/h should not exceed 0.9.

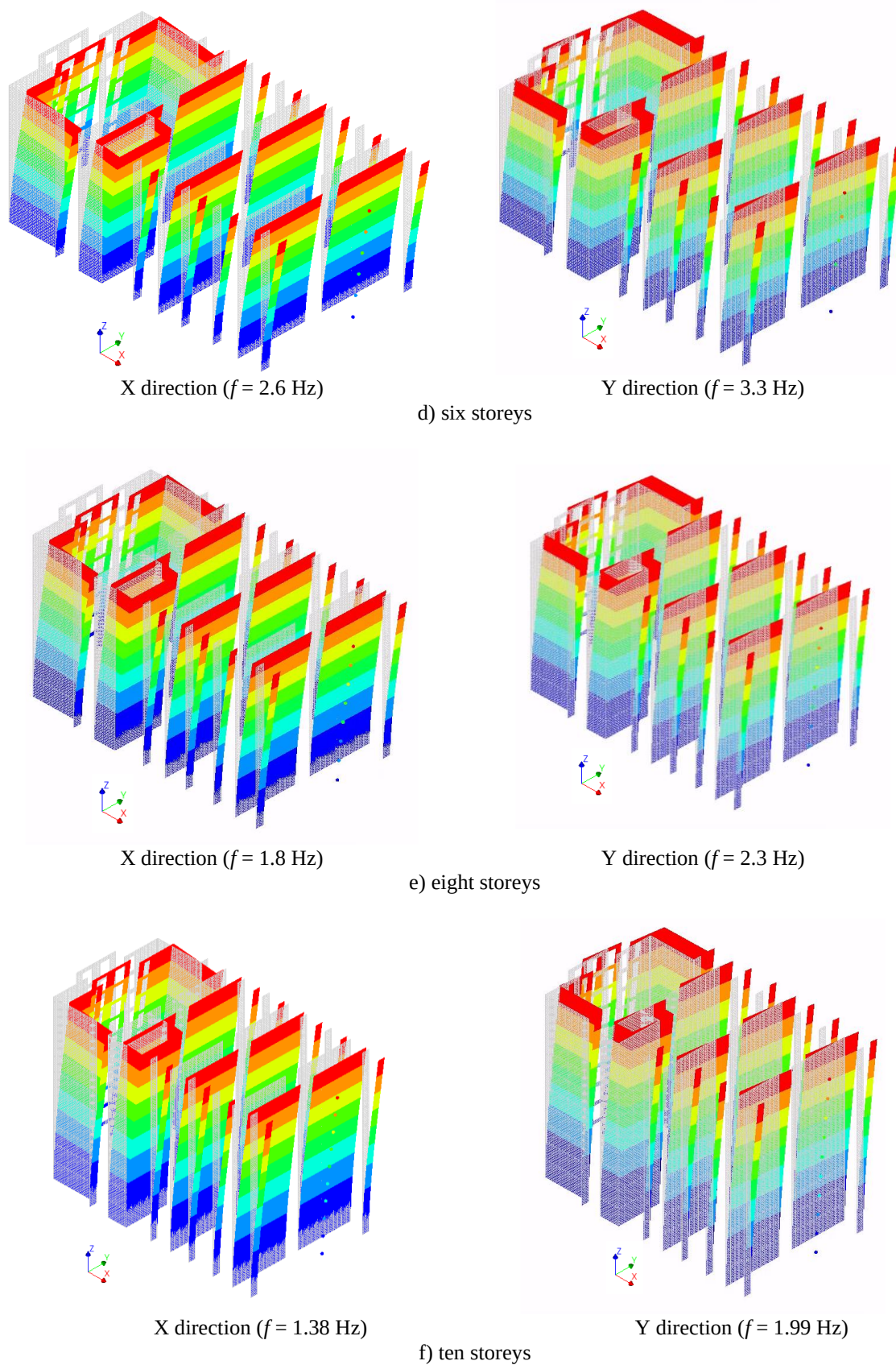
The eigenmodes and corresponding eigenfrequencies obtained from the analyses are illustrated in Figure 125 and Figure 126 for buildings B1 and B2, respectively. A comparison between the T values calculated from eigenanalyses and that predicted by the formulas in design standards is presented in Figure 127 and Figure 128, respectively for the buildings B1 and B2 and summarized in Table 21. Additionally, a comparison of T values from eigenanalyses of both buildings is presented in Figure 129. Also shown are a curve and the corresponding formula and R^2 that fits the FEM predictions.

Figure 125: Eigenmodes and corresponding eigenfrequencies of building B1.



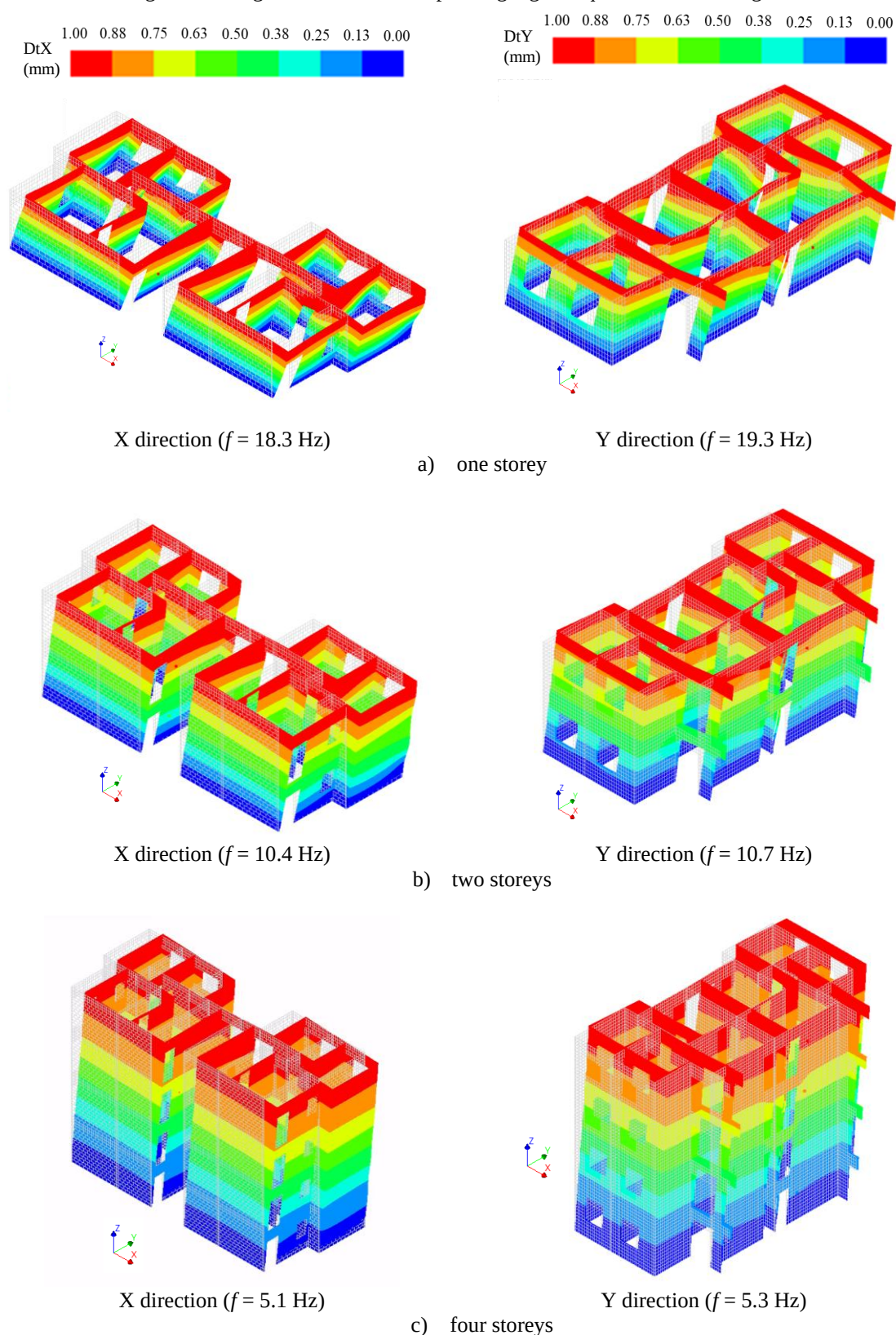
Source: Author.

Figure 124: Eigenmodes and corresponding eigenfrequencies of building B1 (continued).



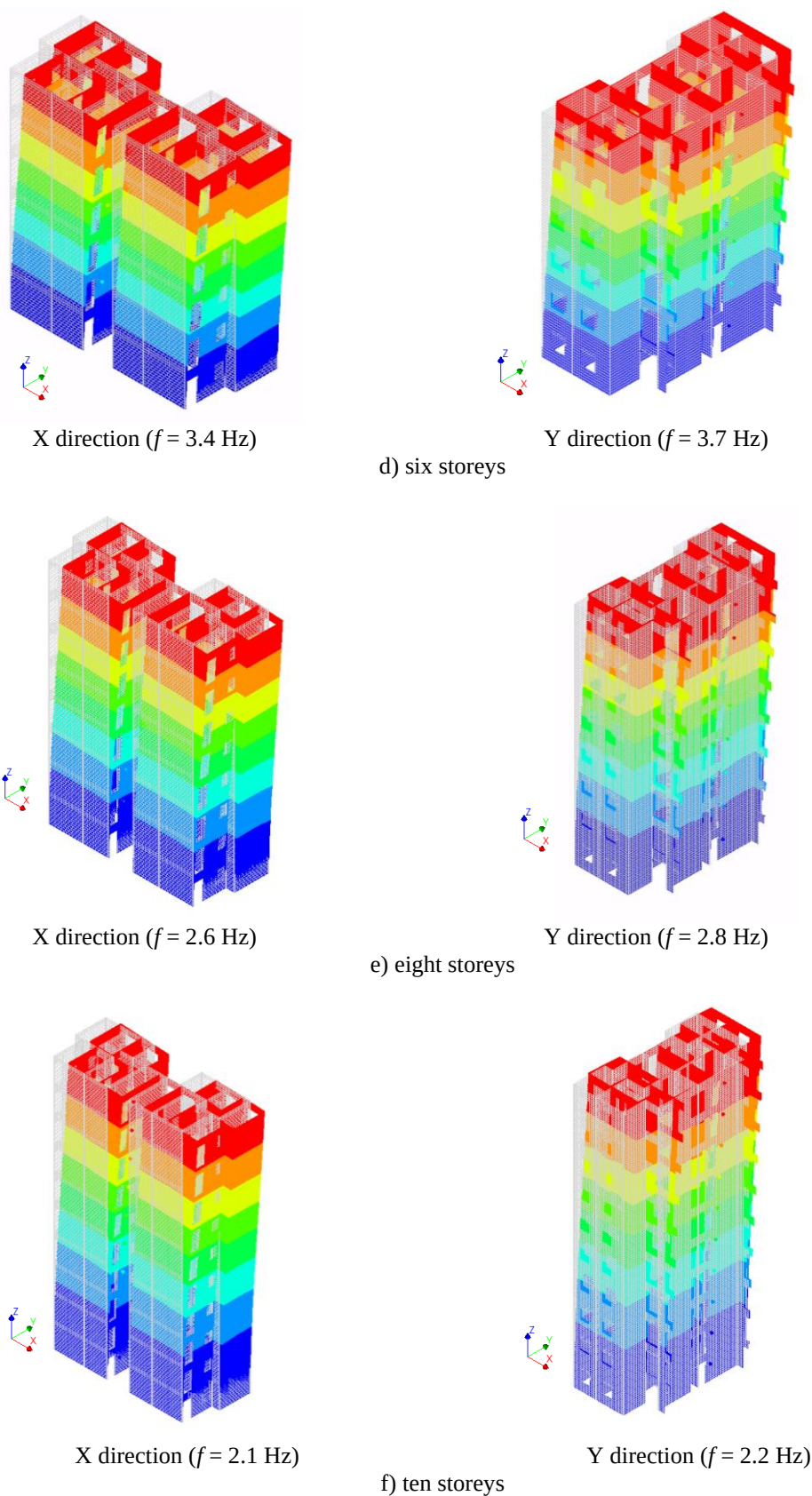
Source: Author.

Figure 126: Eigenmodes and corresponding eigenfrequencies of building B2.



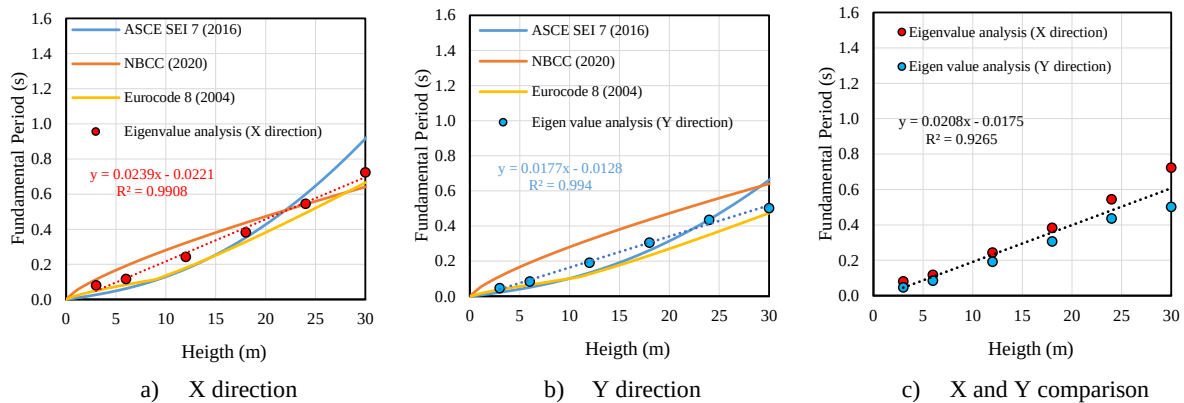
Source: Author.

Figure 125: Eigenmodes and corresponding eigenfrequencies of building B2 (continued).



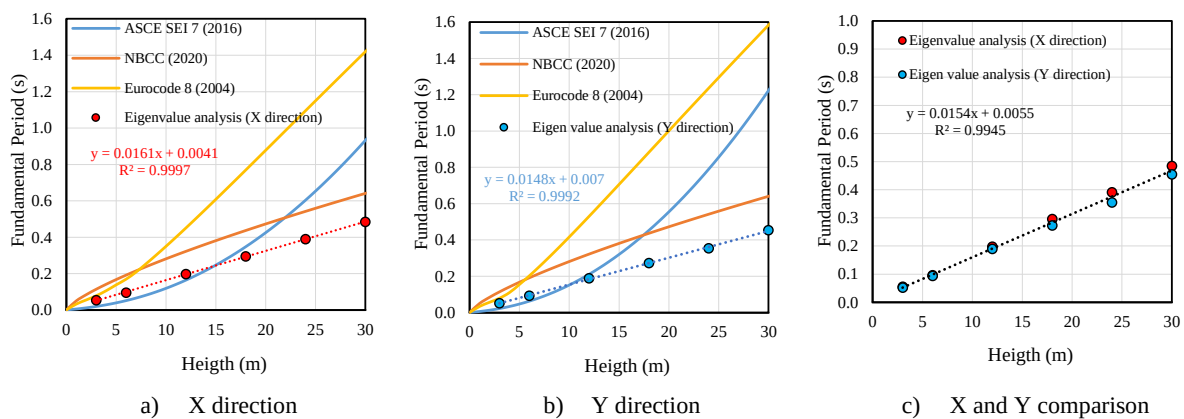
Source: Author.

Figure 127: Comparison of the fundamental period, T , from eigenanalyses and equations in design standards for building B1.



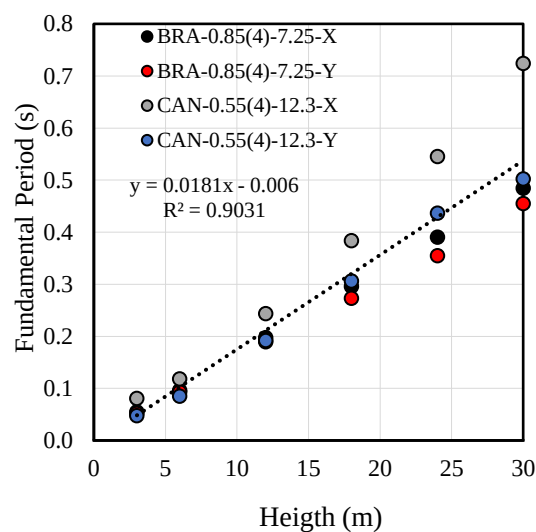
Source: Author.

Figure 128: Comparison of the fundamental period, T , from eigenanalyses and equations in design standards for building B2.



Source: Author.

Figure 129: Comparison of the fundamental period, T , from eigenanalyses of both buildings.



Source: Author.

Table 21: Summary of the fundamental period, T , comparison for the two buildings.

Method	Height (m)	No. of storeys	Canadian Building (B1)				Brazilian Building (B2)			
			X direction		Y direction		X direction		Y direction	
			T (s)	ΔT (%)	T (s)	ΔT (%)	T (s)	ΔT (%)	T (s)	ΔT (%)
Eigenvalue analysis	3	1	0.081	—	0.047	—	0.055	—	0.052	—
NBCC (2020)			0.114	41.4	0.114	140.3	0.114	108.8	0.114	119.5
Eurocode 8 (2004)			0.050	-37.8	0.038	-19.1	0.078	43.1	0.079	51.8
ASCE/SEI 7 (2022)			0.027	-66.3	0.023	-51.6	0.019	-64.4	0.022	-57.3
Eigenvalue analysis	6	2	0.118	—	0.085	—	0.096	—	0.093	—
NBCC (2020)			0.192	62.8	0.192	126.0	0.192	99.8	0.192	105.8
Eurocode 8 (2004)			0.084	-28.3	0.066	-22.2	0.169	76.4	0.202	117.4
ASCE/SEI 7 (2022)			0.062	-47.1	0.051	-39.5	0.051	-46.3	0.063	-32.4
Eigenvalue analysis	12	4	0.243	—	0.192	—	0.197	—	0.189	—
NBCC (2020)			0.322	32.4	0.322	68.1	0.322	63.4	0.322	70.2
Eurocode 8 (2004)			0.180	-26.0	0.127	-33.8	0.451	128.5	0.531	180.2
ASCE/SEI 7 (2022)			0.174	-28.4	0.134	-29.9	0.164	-16.8	0.211	11.5
Eigenvalue analysis	18	6	0.383	—	0.306	—	0.296	—	0.273	—
NBCC (2020)			0.437	14.0	0.437	42.8	0.437	47.8	0.437	60.1
Eurocode 8 (2004)			0.328	-14.4	0.232	-24.0	0.769	160.2	0.883	223.7
ASCE/SEI 7 (2022)			0.352	-8.2	0.262	-14.4	0.348	17.6	0.453	66.1
Eigenvalue analysis	24	8	0.545	—	0.436	—	0.390	—	0.355	—
NBCC (2020)			0.542	-0.5	0.542	24.3	0.542	38.9	0.542	52.9
Eurocode 8 (2004)			0.492	-9.7	0.350	-19.8	1.095	180.6	1.237	248.8
ASCE/SEI 7 (2022)			0.599	9.8	0.438	0.3	0.604	54.6	0.790	123.0
Eigenvalue analysis	30	10	0.724	—	0.502	—	0.484	—	0.455	—
NBCC (2020)			0.641	-11.5	0.641	27.6	0.641	32.4	0.641	41.0
Eurocode 8 (2004)			0.665	-8.1	0.474	-5.6	1.420	193.4	1.584	248.3
ASCE/SEI 7 (2022)			0.915	26.5	0.662	31.8	0.932	92.6	1.224	169.2

Source: Author.

All of the approaches for calculating T indicate that taller buildings exhibit longer natural periods. This trend confirms expectations as buildings reduce their initial stiffness with increasing height. Brzev and Anderson (2018) pointed out that many low-rise masonry buildings usually have T around 0.2 to 0.3 s. For both structural layouts, the natural periods calculated by eigenvalue analyses are around 0.2 s for the four-storey buildings. At six-storey, building B2 and building B1 in the Y direction still have T values around 0.3 s, while building B1 in the X direction exceeds this threshold. The natural period in the X direction for building B1 increases more rapidly with height than in the Y direction, reflecting reduced stiffness due to a smaller web wall area (net effective cross-section) in this direction.

For the Canadian building (B1), the NBCC (2020) estimates higher T values than those calculated by eigenvalue analyses for up to six stories in the X direction and for all stories in the Y direction. For example, at ten stories, NBCC (2020) predicts $T = 0.64$ s for both directions,

while the eigenvalue analysis calculates $T = 0.724$ s in the X direction, representing an underestimation of -11.5% . In contrast, in the Y direction, the eigenvalue analysis gives $T = 0.502$ s, leading to an overestimation of 27.6% . For the four-storey configurations, the percentage differences are 32.4% in the X direction and 68.1% in the Y direction. These findings underscore the limitations of the NBCC (2020) equation in accurately estimating T values for buildings with walls primarily distributed in a single direction, such as B1, as the equation relies solely on building height. ASCE/SEI 7 (2022) predicts lower T values than eigenvalue analyses for up to six-storey in the X direction and eight-storey in the Y direction. Beyond these heights, ASCE/SEI 7 equation tends to overestimate T values for both directions. The predictions range from -66.3% (X direction) and -51.6% (Y direction) for a single storey to 26.5% (X direction) and 31.8% (Y direction) for ten stories. Eurocode 8 (2004) better captures the trend of increasing periods while predicting lower T values than eigenvalue analyses for all storey configurations.

For the building B2, NBCC (2020) and Eurocode 8 (2004) formulas estimate higher T values in both principal directions across all heights. Although ASCE/SEI 7 (2022) generally aligns well with eigenvalue analysis results up to mid-rise heights (up to four storeys); for example, for the four-storey buildings, the differences are -16.8% in X direction and 11.5% in Y direction. For taller buildings, differences become more pronounced, with deviations of 54.6% and 123.0% (X and Y directions, respectively) for eight storeys, and 92.6% and 169.2% for ten storeys. Similarly, Eurocode 8 (2004) initially aligns reasonably well for shorter buildings, but as height increases, it underestimates the periods significantly. At ten storeys, Eurocode 8 (2004) formula predicts a period of 1.58 s in the Y direction, compared to the eigenvalue result of 0.45 s, a 248.3% difference. This discrepancy is due to the low l_{wi}/h ratio, initially limited to 0.9 for wall-heights up to six meters. Beyond six meters, the l_{wi}/h ratio values decrease, causing an increase in the estimated value of T , so resulting in a significant deviation from the eigenvalue analyses and the predictions of other design standards.

The results here suggest that Eurocode 8 (2004) and ASCE/SEI 7 (2022) formulas, may not adequately capture the fundamental period trend for high-rise buildings with structural layouts like building B2. It is worth noting that the Eurocode 8 formula is primarily targeted at low- to mid-rise buildings, as tall masonry buildings are uncommon in Europe, which may explain the unfitting predictions for high-rise structures. The trend predicted by the NBCC (2020) is more similar to the results of the eigenvalue analyses, despite the Canadian standard estimating higher values, with a percentage variation ranging from 63.4% (X direction) and

70.2% (Y direction) for four storeys to 32.4% (X direction) and 41% (Y direction).

For the building B1 (Figure 127c), the X direction exhibits higher fundamental periods than the Y direction as height increases, attributed to varying stiffness in the two directions. The walls in building B1 are predominantly distributed along the Y direction, resulting in greater stiffness and shorter fundamental periods in that axis. A linear trend line with $R^2 = 0.93$ suggests a reasonably accurate prediction, although some variation is present due to the structural asymmetry. In contrast, for building B2 (Figure 128c), the trend lines show nearly identical fundamental periods in both directions, reflecting the balanced wall distribution across the principal axes. The linear trend line predicts the T values for building B2 from one up to ten stories with higher accuracy ($R^2 = 0.99$). This uniformity leads to a consistent structural response in the building B2, with similar stiffness and fundamental periods in both directions.

Seismic standards lead to estimated T values that, for safety, should generally be lower than those from eigenanalysis. However, results here show this is not always true. While ASCE/SEI 7 (2022) and Eurocode 8 (2004) are conservative in estimating lower T values for structural layouts like B1, NBCC (2020) is less conservative for low- to mid-rise buildings, showing higher values (32.4% in the X direction and 68.1% in the Y direction for four storeys), potentially leading to lower seismic forces in force-based design. For structural layouts like B2, equations from the standards often provide higher values, particularly in ASCE/SEI 7 (2022) and Eurocode 8 (2004). In taller buildings, these unconservative estimates could lead to calculated lower seismic forces, resulting in lower safety margins.

Comparing both buildings (Figure 129), the building B2, particularly in the X direction, exhibits lower fundamental periods. This difference is likely attributed to the higher percentage of web wall area relative to the floor area for B2 compared to B1. A linear trend line represents the general increase in natural periods with building height, with an $R^2 = 0.90$, suggesting reasonable accuracy. However, as the trend line is based on only two distinct structural layouts with specific characteristics, it should not be generalized to all scenarios. While useful within this dataset, it does not capture the variability that would arise across a broader range of building designs and structural layouts. Furthermore, the results emphasize that height is a key factor in determining the fundamental period, but the architectural layout significantly influences the period values. Predictions such as those of NBCC (2020) and NBR 15421 (2023), which depend solely on building height, can be oversimplified and are not always conservative.

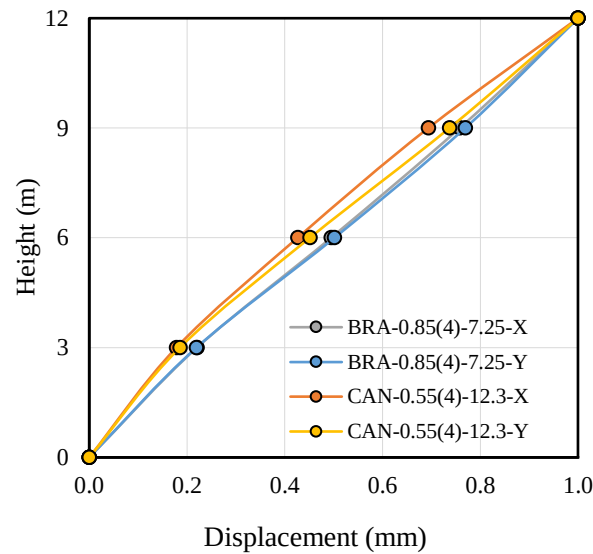
6.4.2 Pushover analyses of the four-storey buildings

Nonlinear static (pushover) analyses were performed for each building layout to evaluate the lateral stiffness, lateral force capacity, and ductility. Furthermore, the results of the pushover analyses are also used to investigate the seismic performance of the studied buildings.

In the numerical analyses presented here, the solution methods outlined by Palhares et al. (2024, 2025) were employed. The Secant (Quasi-Newton) iterative method with the Broyden-inverse updated method (Broyden, 1970) was adopted with a sparse Cholesky-based direct solver. The convergence criterion was defined based on the procedure reported by Hendriks et al. (2017), also adopted by Mahmoudimotlagh (2020), where load increments are converged if either the force norm or the energy norm is satisfied within the maximum number of 100 iterations. Error tolerance for energy and force norms were set at 0.1% and 1%, respectively. Load increments that do not fully meet these criteria may still be accepted if convergence is achieved in the following step. This approach allows the computation to continue even if a specific step fails to converge within 100 iterations, accommodating localized failures.

Following the effective approach outlined by Palhares et al. (2025), a displacement-controlled pushover analysis strategy was adopted, where displacements with increments of 0.20 mm are imposed at each floor level following the modal displacement shape obtained from eigenanalysis, as shown in Figure 130. The gravity load was distributed at the upper nodes of the wall, and a masonry mass density of 2250 kg/m³ was adopted to consider for self-weight, resulting in an axial load level of 12.3% of the average masonry compressive strength ($f_m = 14.6$ MPa) for building B1 and 7.25% of the average masonry compressive strength ($f_m = 6.4$ MPa) for building B2.

Figure 130: Displacement profile patterns imposed for the pushover analysis of the buildings.



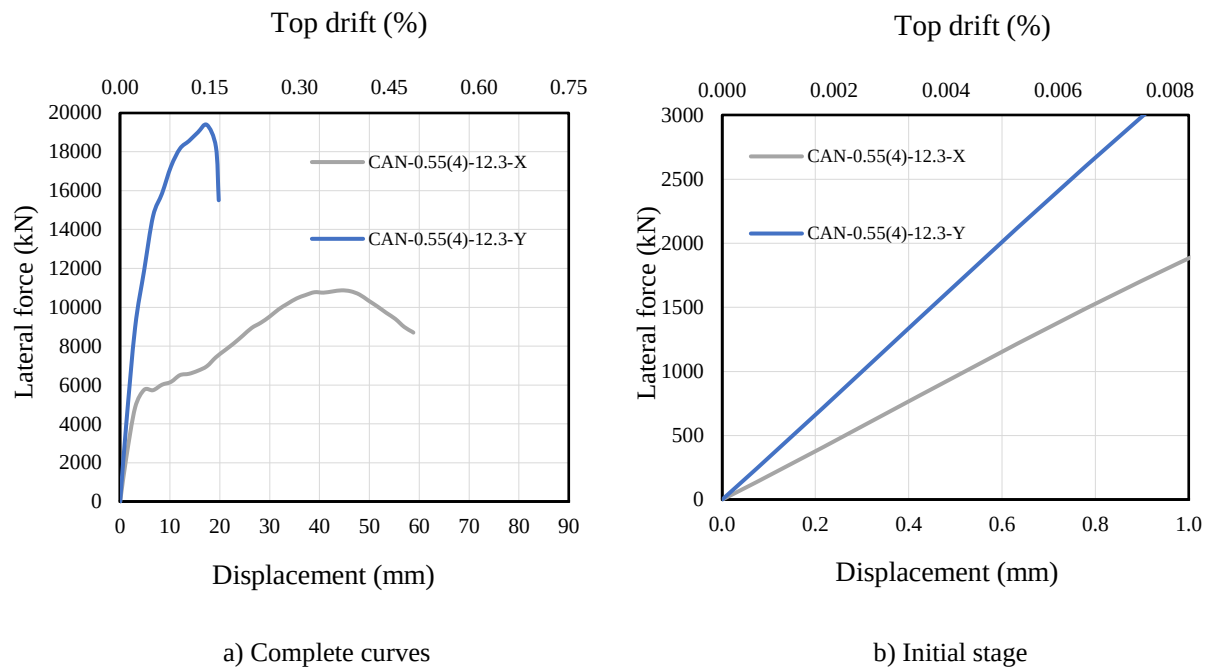
Source: Author.

The three-dimensional models, developed using high-resolution meshes to catch localized effects, effectively captured key phenomena, such as masonry cracking, interface detachment, and reinforcement yielding. However, high computational demands due to the high number of elements and nodes posed significant challenges, requiring advanced processing resources. The analyses were performed on a high-performance server with an Intel Core i9-7900X 3.30 GHz CPU (10 cores, 20 threads), 128 GB RAM, and a 512 GB NVMe SSD (RAID 1).

6.4.2.1 Canadian building (B1)

The force–displacement graphs (capacity curves) for the building B1 are depicted in Figure 131. Since only half of each building was modeled to reduce the computational cost, the lateral forces obtained from the analyses were doubled to reflect the behavior of the entire building. The complete responses are shown on the left side, while a zoomed-in view of the initial stage of the responses is provided on the right side. These curves represent the sum of base shear reactions plotted against the top displacement of the building. The analyses were carried out until a 20% degradation of the structure's lateral capacity. Additionally, the top drift values, expressed as the displacement-to-height ratio relative to the building base, are indicated along the upper horizontal axis.

Figure 131: Force–displacement curves of building B1.



Source: Author.

The complete capacity curves (Figure 131) indicate that the building B1 exhibits distinctly different behavior in the X and Y directions. The lateral force capacity in the Y direction is significantly higher than in the X direction (+78.1%), reaching a peak load of 19,380 kN. This disparity is likely due to the greater density of walls aligned along the Y-axis. Despite the higher lateral force capacity in the Y direction, its curve shows a peak load followed by a relatively rapid degradation in the load-bearing capacity as lateral displacements increase, with the top drift limited to approximately 0.17%. In contrast, a more flexible response is observed in the X direction, with a gradual reduction in capacity after reaching peak load and a higher top drift of around 0.50%. This suggests that while the Y direction is stiffer and stronger, it loses capacity quickly after post-peak, whereas the X direction provides a more ductile response.

The damage evolution in terms of masonry cracking, interface detachment (equivalent to cracks in the mortar joints), and vertical reinforcement stress is shown in Figure 132 and Figure 133 for the CAN-0.55(4)-12.3-X and CAN-0.55(4)-12.3-Y building models, respectively. This damage evolution is analyzed as a function of the total energy dissipated, U , which corresponds to the area under the capacity curve. The increments considered are: $0.25U$, $0.50U$, $0.80U$, and $1.0U$. For CAN-0.55(4)-12.3-X, only one of the W6 walls is presented for a clearer visualization.

Figure 132: Damage pattern evolution at $0.25U$, $0.50U$, $0.80U$, and $1.0U$ dissipated energy levels for the CAN-0.55(4)-12.3-X model (only one of the W6 walls is shown).

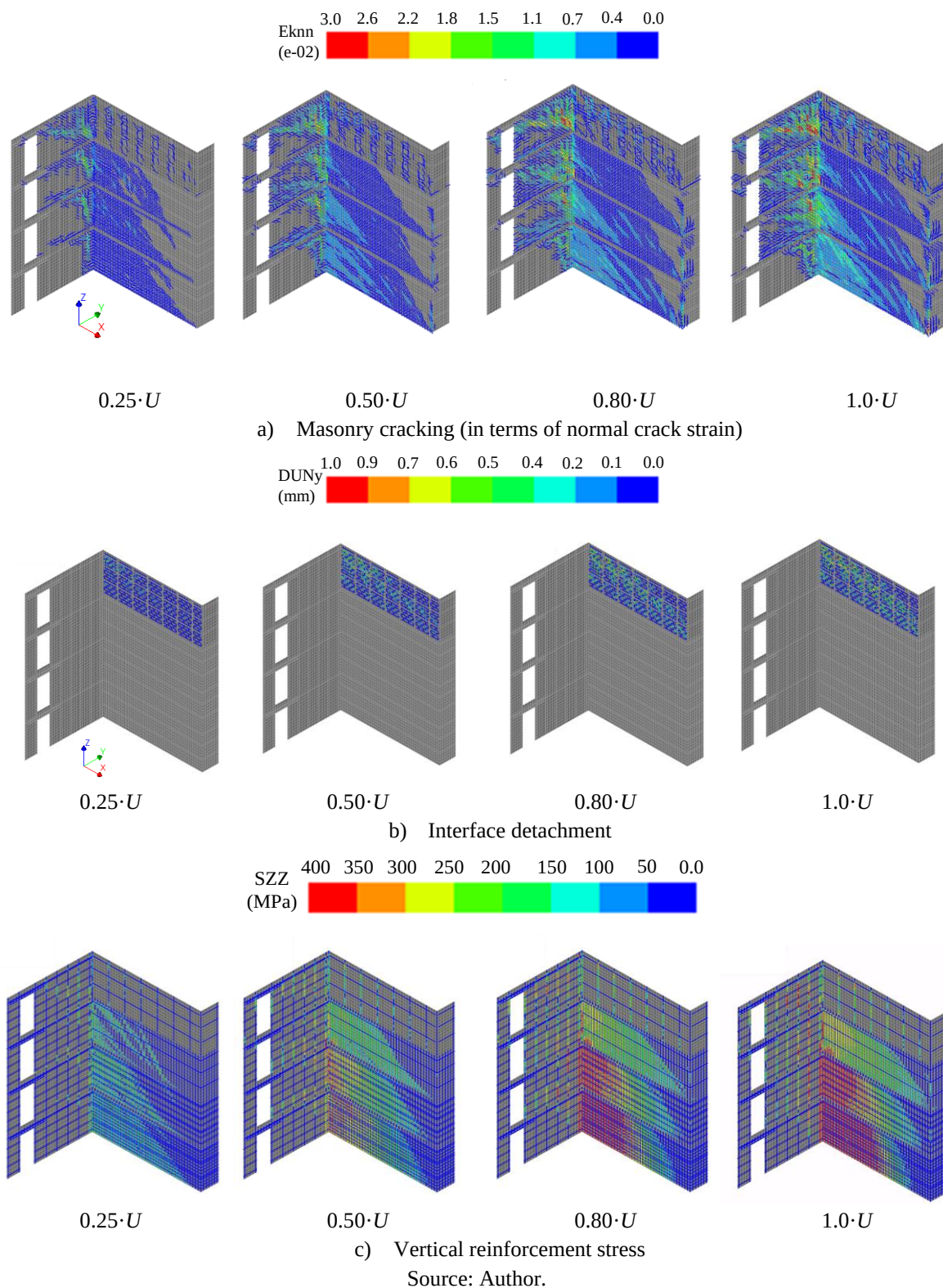
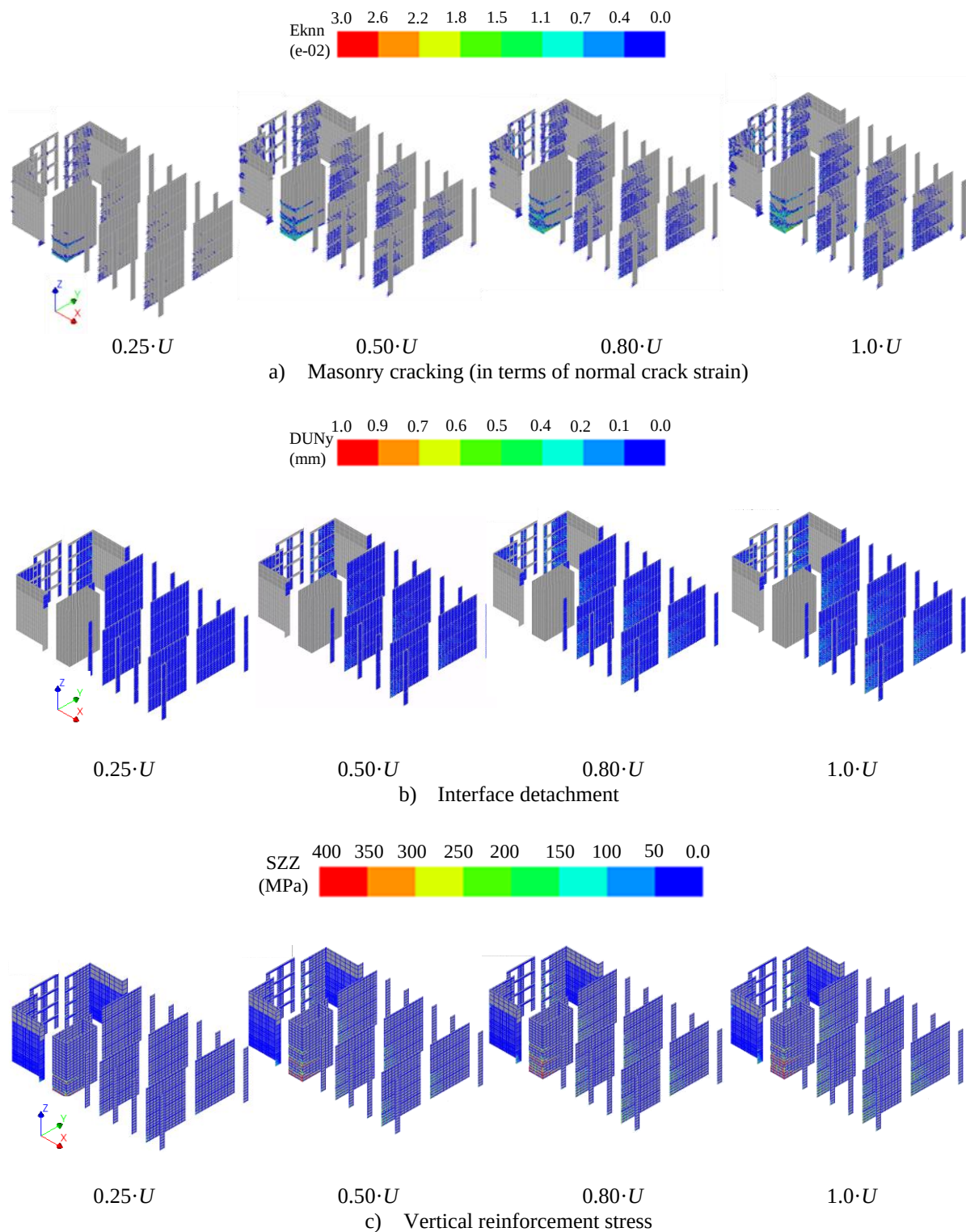


Figure 133: Damage pattern evolution at $0.25U$, $0.50U$, $0.80U$ and $1.0U$ dissipated energy levels for the CAN-0.55(4)-12.3-Y model.

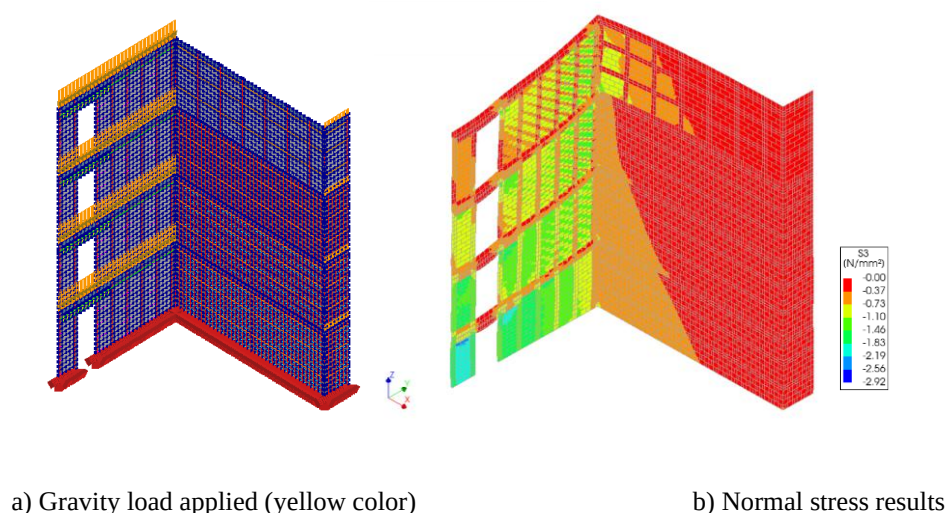


Source: Author.

The damage pattern intensifies through each dissipated energy increment: minor damage at $0.25U$, moderate at $0.50U$, severe at $0.80U$, and collapse at $1.0U$, where a 20% reduction in lateral force capacity is observed. A complex damage pattern emerges in wall W6, consisting of FGRM walls on the first three storeys and a PGRM wall on the top. At $0.25U$, initial masonry cracks develop diagonally in the FGRM walls, while cracks in the top PGRM wall occur concentrated in the grouted regions. Notable cracks appear along the flanges, highlighting the contribution of orthogonal walls in W6. Mortar joint detachment in the top PGRM wall is consistent with partial grouting, with diagonal cracks appearing in ungrouted panels. Vertical reinforcement stresses are high and correlate with the diagonal cracking in the masonry. Many rebars achieve the yield stress of 400 MPa, particularly on the first and second storeys at $0.8U$ and $1.0U$.

An illustration in Figure 134 depicts how the normal stresses from the gravity load applied to the wall W2 are transferred and distributed to the orthogonal wall, W6, which includes three storeys with FGRM walls and a top-storey PGRM. The stress distribution shows a clearly inclined path, where the vertical forces from gravity loads spread downward through the walls and are dispersed across the intersecting walls. Differences between FGRM and PGRM stress paths are evident, likely due to the stiffness variations and cross section-area discontinuity of the PGRM walls, highlighting the complex behavior due to the gravity load distribution.

Figure 134: Normal stresses distributed to the orthogonal walls.

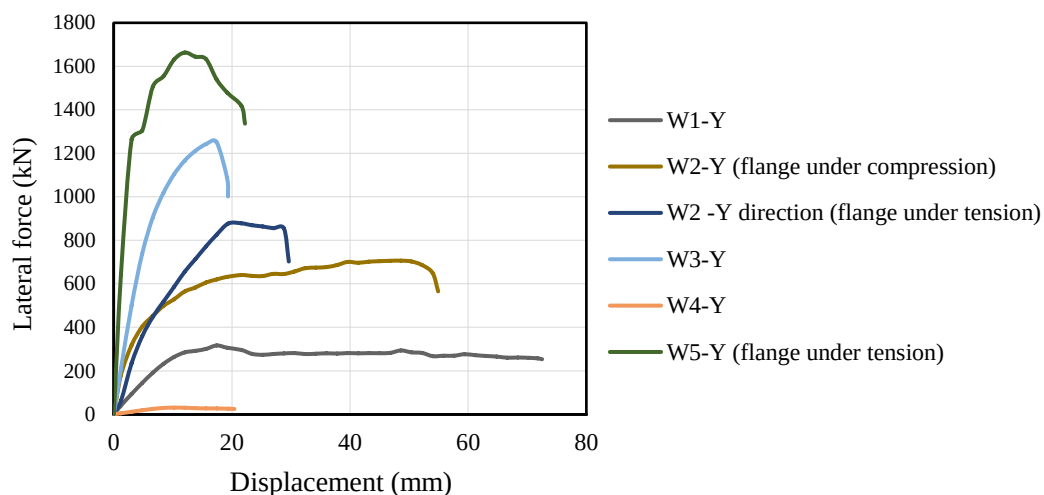


Source: Author.

For the CAN-0.55(4)-12.3-Y model, masonry cracks are widely distributed due to the arrangement of reinforcement and grouted regions within the walls, with pronounced damage concentrated as horizontal cracks along the FGRM flange of wall W5. Some vertical rebars, located where horizontal cracks are concentrated, reach the yield stress of 400 MPa even at 0.2U, and this damage pattern progressively intensifies up to failure.

The nonlinear behavior of the individual walls in the CAN-0.55(4)-12.3-Y model, acting independently without the support of an integrated structural system, is depicted in Figure 135. Each wall's unique lateral force–displacement response illustrates the variation in strengths, stiffnesses and ductility characteristics within the building's seismic force-resisting system.

Figure 135: Lateral force–displacement response of individual walls in the Y direction.



Source: Author.

Walls W1, W2, W3, W4 and W5 exhibit different lateral force–displacement responses. For instance, W5, with its flange under tension, reaches the highest lateral force capacity, around 1,700 kN, but undergoes relatively rapid strength degradation as displacement increases, similarly to the response of the entire building. The behavior of W5 contrasts with that of W1, which has a lower lateral force capacity (321 kN) and exhibits higher ductility and more gradual strength degradation up to the ultimate displacement, d_{Vu} , equal to 72.5 mm. Walls with flanges, such as W2 and W5, exhibit notable differences when their flanges are under compression versus tension; configurations with the flange under tension present higher lateral load capacity but are less ductile.

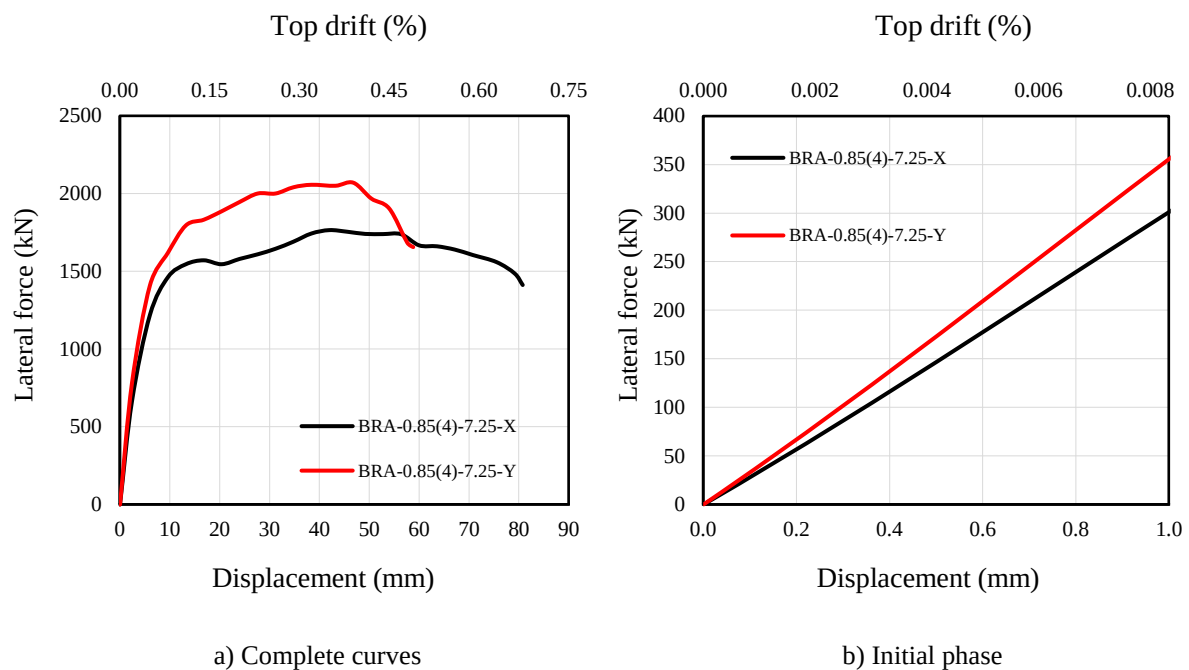
These variations in individual wall responses highlight the complexity of concrete masonry buildings composed of walls with diverse cross-sectional geometries and reinforcement configurations, highlighting the limitations of analyzing isolated walls for

system-level seismic performance assessments. By forming part of an integrated system, these walls would interact, redistributing loads and engaging in different stages of loading to collectively contribute to the global lateral resistance of the building (Heerema et al., 2015a; Heerema; Shedid; El-Dakhakhni, 2015b). This emphasizes the need for a comprehensive analysis that considers the combined behavior of walls in concrete masonry buildings, rather than relying on single-wall response as a predictor of overall building performance.

6.4.2.2 Brazilian building (B2)

The force–displacement curves for the building B2 are depicted in Figure 136. Since only half of each building was modeled to reduce the computational cost, the lateral forces obtained from the analyses were doubled to reflect the behavior of the entire building. The complete responses are shown on the left side, while a zoomed-in view of the initial phase of the responses is provided on the right side. These curves represent the sum of base shear reactions plotted against the top displacement of the building. The analyses were carried out until a 20% degradation of the structure’s lateral capacity. Additionally, the top drift values, expressed as the displacement-to-height ratio relative to the building base, are indicated along the upper horizontal axis.

Figure 136: Force–displacement curves of building B2.



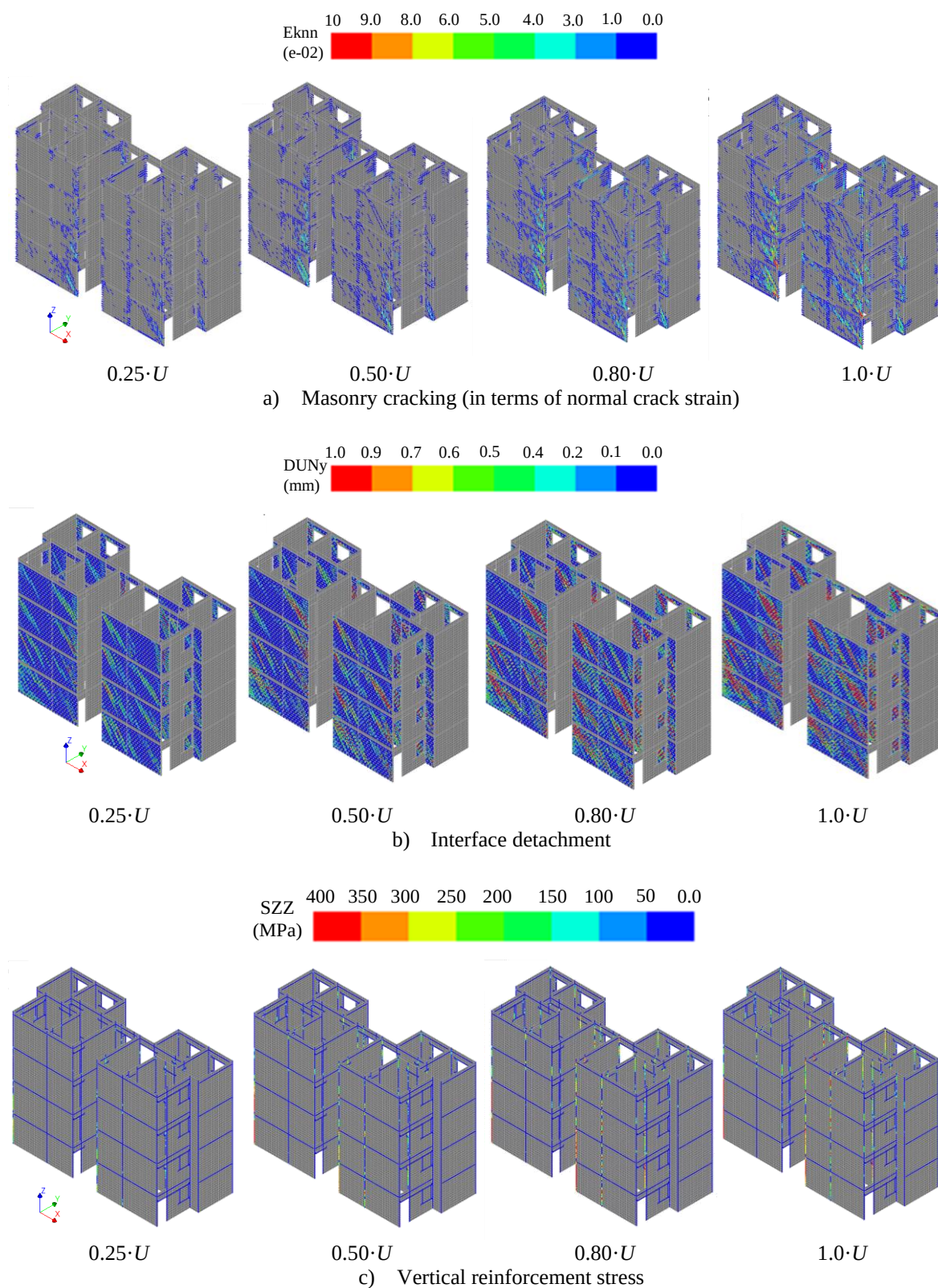
Source: Author.

The capacity curves show that building B2 exhibits relatively similar behavior in the X and Y directions, with some differences. In the Y direction, the building reaches a slightly higher lateral force capacity than in the X direction (+17.3%), reaching a peak load of 2,070 kN. The response in the X direction demonstrates higher ductility, with an ultimate top drift of around 0.68% compared to 0.49% in the Y direction. These ultimate drift values align with established benchmarks, with the X-direction value consistent with the three-storey PGRM system's ultimate drift of 0.72%, as found by Palhares et al. (2025), while the Y-direction value matches the experimental ultimate drift of 0.52% for a three-storey PGRM wall, as found by Fortes et al. (2016). Such ultimate drift values are associated with shear failure, contrasting with higher values observed for flexural failure, for example 2.14% for a 20-storey FGRM building, which walls are designed having confined boundary tie-elements and exhibiting a flexural failure mode, as outlined by Aly and Galal (2019). Despite this, both directions demonstrate a similar strength degradation pattern after reaching peak lateral force, with a gradual decline in the load-bearing capacity as displacement increases.

The initial phase of the force–displacement curves (Figure 136b) highlights that the Y-direction response has a slightly steeper initial slope, indicating an initial stiffness marginally higher than in the X direction. The secant stiffness in the Y direction is 325.7 kN/mm compared to 276.8 kN/mm in the X direction, therefore with about +17.7% in the Y direction.

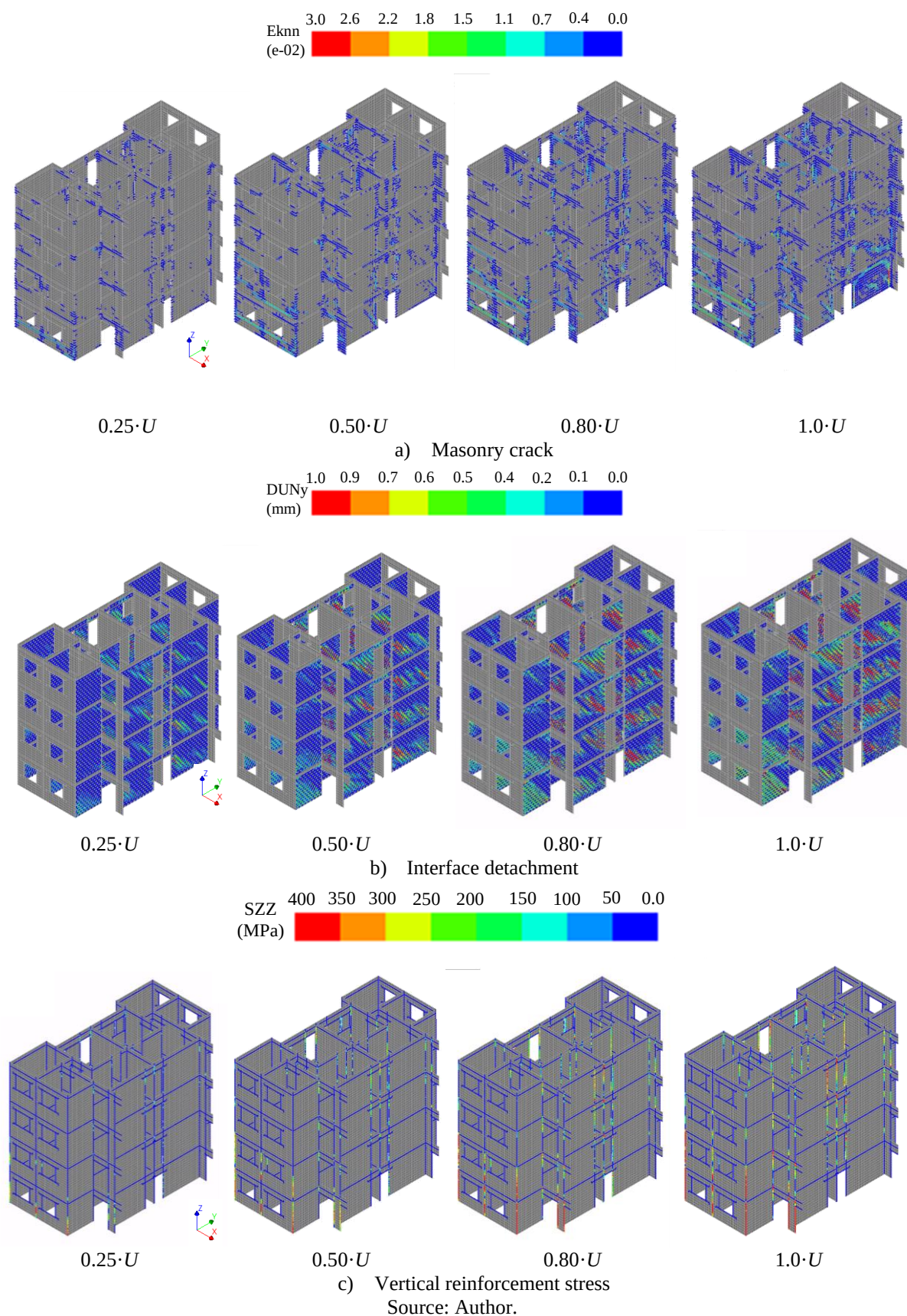
The damage evolution in terms of masonry cracking, interface detachment (equivalent to cracks in the mortar joints), and vertical reinforcement stress is illustrated in Figure 137 and Figure 138 for the BRA-0.85(4)-7.25-X and BRA-0.85(4)-7.25-Y models, respectively. Damage progression was analyzed at different levels of dissipated energy, U : $0.25U$, $0.50U$, $0.80U$ and $1.0U$.

Figure 137: Damage pattern evolution at $0.25U$, $0.50U$, $0.80U$, and $1.0U$ dissipated energy levels for the BRA-0.85(4)-7.25-X model.



Source: Author.

Figure 138: Damage pattern evolution at $0.25U$, $0.50U$, $0.80U$ and $1.0U$ dissipated energy level for the BRA-0.85(4)-7.25-Y model.



The observed damage patterns in both directions reveal a clear and pronounced diagonal shear failure mode, evidenced by the development of diagonal cracks in the masonry and damage at the mortar joints (interfaces detachment) along the ungrouted panels. This crack pattern aligns with the expected shear failure behavior for PGRM walls with vertical reinforcement concentrated at the ends (Fortes et al., 2016; Medeiros et al., 2022a; Palhares et al., 2023, 2024).

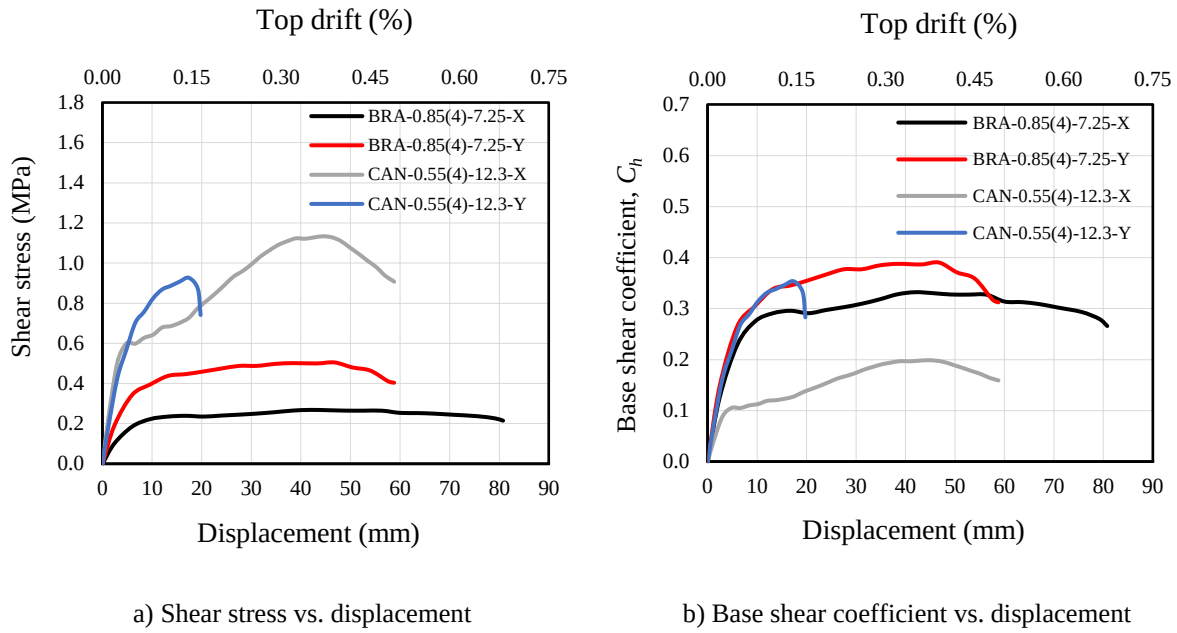
The diagonal shear damage pattern is noticeable early in the analysis, with low magnitude cracking emerging at $0.25U$. As energy dissipation increases, the damage intensity increases, resulting in progressive cracking and interface detachment. By the final energy level ($1.0U$), diagonal shear cracks are fully developed, indicating structural failure in the numerical model.

The vertical reinforcement stress on the tensioned side of the building reaches the yield level (400 MPa) as early as $0.25U$, beginning on the first storey and progressively involving the vertical reinforcement across approximately half the building's height (two storeys) at the failure. As shear cracks propagate diagonally across the structure, additional points of vertical reinforcement along these paths also reach yield stress due to the concentrated stresses induced by the diagonal shear cracking. The numerical results underscore that the diagonal shear failure mode strongly characterizes buildings with vertical reinforcement concentrated only at the walls' ends.

6.4.2.3 Comparison between the buildings

To directly compare both structural layouts, the nonlinear responses of the two buildings are plotted in terms of shear stress (defined as V/A_w) in Figure 139a and base shear coefficient C_h (defined as V/W) in Figure 139b. The shear stress reflects the capacity relative to the effective wall area resisting lateral forces, while the base shear coefficient provides insight into the building's capacity relative to its weight. Additionally, a summary of the pushover analysis results for both buildings is presented in Table 22, which includes key response measures: the maximum lateral force, V_{max} , the displacement at the maximum lateral force, d_{Vmax} , the ultimate displacement, d_{Vu} , the secant stiffness, K_{sec} , and the dissipated energy, U . The d_{Vu} was defined as that corresponding to a 20% reduction in V_{max} . The K_{sec} was calculated over a range of 5% to 33% of V_{max} , following a common procedure in design standards (e.g., TMS 402/602 (2022) and CSA S304 (2024)) for calculating the Young's modulus to adopt in design.

Figure 139: Force–displacement response for the different building structural layouts.



Source: Author.

The secant stiffness, K_{sec} , values presented in Table 22 reveal a substantial difference in stiffness between the building structural layouts. Building B1 exhibits significantly higher K_{sec} values in both directions. Despite these notable stiffness differences, the natural period, T , values obtained from the eigenanalyses for both buildings are similar. This similarity of T values is likely due to the significantly higher seismic weight of building B1, which compensates for the initial stiffness difference.

Table 22: Summary of key pushover results for the building structural layouts.

Building	V_{max} (kN)	d, V_{max} (mm)	d, V_u (mm)	K_{sec} (kN/mm)	$C_{h,max}$	τ_{max} (MPa)	U (kN·mm)	T (s)
CAN-B-0.55(4)-12.3-X	10877.4	44.4	58.8	1649.6	0.2	1.1	496171.2	0.243
CAN-B-0.55(4)-12.3-Y	19380.0	17.4	19.8	3032.9	0.4	0.9	290430.5	0.192
BRA-0.85(4)-7.25-X	1764.2	42.0	80.5	276.8	0.3	0.3	125303.4	0.197
BRA-0.85(4)-7.25-Y	2069.7	46.8	58.8	325.7	0.4	0.5	105517.0	0.189

Source: Author.

From Figure 139a and Table 22, it is evident that the building B1 achieves a significantly higher maximum shear stress, τ_{max} , than the building B2 in both directions. Specifically, CAN-B-0.55(4)-12.3-X reaches a τ_{max} of 1.1 MPa, while CAN-B-0.55(4)-12.3-Y reaches 0.9 MPa. In contrast, the building B2 exhibits lower τ_{max} values of 0.3 MPa and 0.5 MPa in the X and Y directions, respectively. This increased shear stress capacity for Building B1 aligns with the higher lateral force capacity, V_{max} , observed for this building in previous analyses. However,

for CAN-B-0.55(4)-12.3-Y in particular, this higher stress level corresponds to a more rapid reduction in lateral capacity in the post-peak, indicating lower ductility. Building B2, despite its lower shear stress values, demonstrates higher ductility, likely resulting from a more uniform axial load distribution between its walls.

Although the building B1 exhibits a significantly higher lateral force capacity and τ_{max} in both directions compared to the building B2, the graph in Figure 138b shows that the building B2 performs better in terms of the base shear coefficient C_h . The model BRA-0.85(4)-7.25-X reaches a $C_{h,max}$ of 0.33 while BRA-0.85(4)-7.25-Y achieves a $C_{h,max}$ of 0.39, which are both higher than the values obtained for the building B1 in both directions (0.20 in the X direction and 0.35 in the Y direction). The lower $C_{h,max}$ values for the building B1 are due to its significantly greater seismic weight, which is 13,703 kN compared to 1,327 kN for building B2, i.e. +933%. This difference is primarily attributed to the substantially larger total floor area (+776.15%), the more significant dead load from the slabs (+82.9%), thicker blocks, and more grouted regions.

At first glance, the higher $C_{h,max}$ values observed for the building B2 layout indicate that this building yields greater seismic resistance, as it can resist slightly higher base shear force in proportion to its weight (V/W). Furthermore, the higher ultimate top drifts for building B2, particularly compared to CAN-B-0.55(4)-12.3-Y, suggest that this building may exhibit better seismic performance compared to the building B1 analyzed here, due to its higher ductility.

Since seismic forces are proportional to the seismic weight, a heavier structure will inherently face greater seismic demands. A detailed evaluation of seismic performance for both structural layouts is provided in the following section.

6.5 SEISMIC PERFORMANCE EVALUATION

This section evaluates the seismic performance of the Canadian (B1) and Brazilian (B2) buildings through a comprehensive set of analyses. First, seismic force reduction factors (R_d , R_o , and R), derived from energy-equivalent idealized responses based on pushover curves, quantify the ductility and overstrength of the two buildings. Next, the Capacity Spectrum Method (CSM) offers an additional perspective for comparing seismic capacity and demand between the two buildings in different regions. Together, these analyses provide a comprehensive understanding of how structural layouts influence the building's seismic performance across varying seismic contexts.

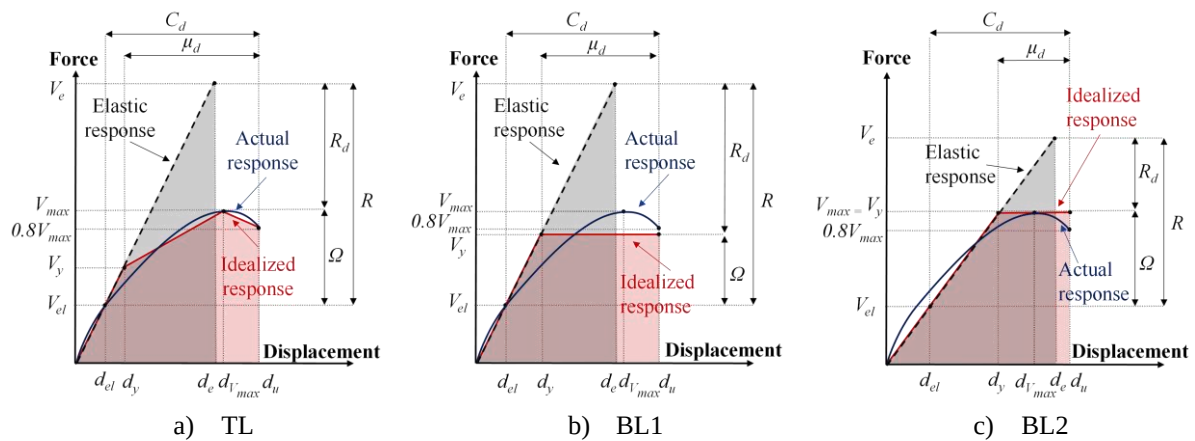
6.5.1 Seismic force reduction factors

The seismic performance assessment is generally made based on idealized force–displacement backbone responses that are equivalent to actual ones. However, there is no universally accepted method for defining these idealized responses, as several approaches are found in the literature (Banting; El-Dakhakhni, 2014; Bolhassani et al., 2016a; Calderón; Sandoval; Arnau, 2017; El-Dakhakhni; Banting; Miller, 2013; Morandi et al., 2022; Ramírez; Sandoval; Almazán, 2016; Seif ElDin; Ashour; Galal, 2019). Three approaches — trilinear and bilinear methods, as described below — were considered to evaluate their impact on determining seismic performance factors for concrete masonry buildings.

In general, the three approaches are based on equating the energy dissipation, U , under the actual and idealized response curves, ensuring that the idealized response accurately represents the energy dissipation of the actual response. The equal energy principle was adopted since all the walls analyzed here have $T < 0.5$ s (Seif ElDin; Ashour; Galal, 2019). The seismic parameters derived from each approach are represented in Figure 140, which schematically compares the methods used to approximate the actual nonlinear response of the concrete masonry buildings evaluated here. Further details and equations for these methods can be found in Medeiros et al. (2022a) and Palhares et al. (2025).

The trilinear approach (TL), depicted in Figure 140a, idealizes the actual response with a trilinear elastic-hardening-softening backbone curve that reaches the same peak load as the actual response and has an ultimate displacement, d_u , corresponding to $0.8 \cdot V_{max}$ in the post-peak phase. The initial stiffness was calculated over a range of 5% to 33% of V_{max} , following the guidelines provided in TMS 402/602 (2022). The yield displacement, d_y , was derived from equalizing energy.

Figure 140: Idealization approaches for equivalent elastoplastic response.



Source: Author.

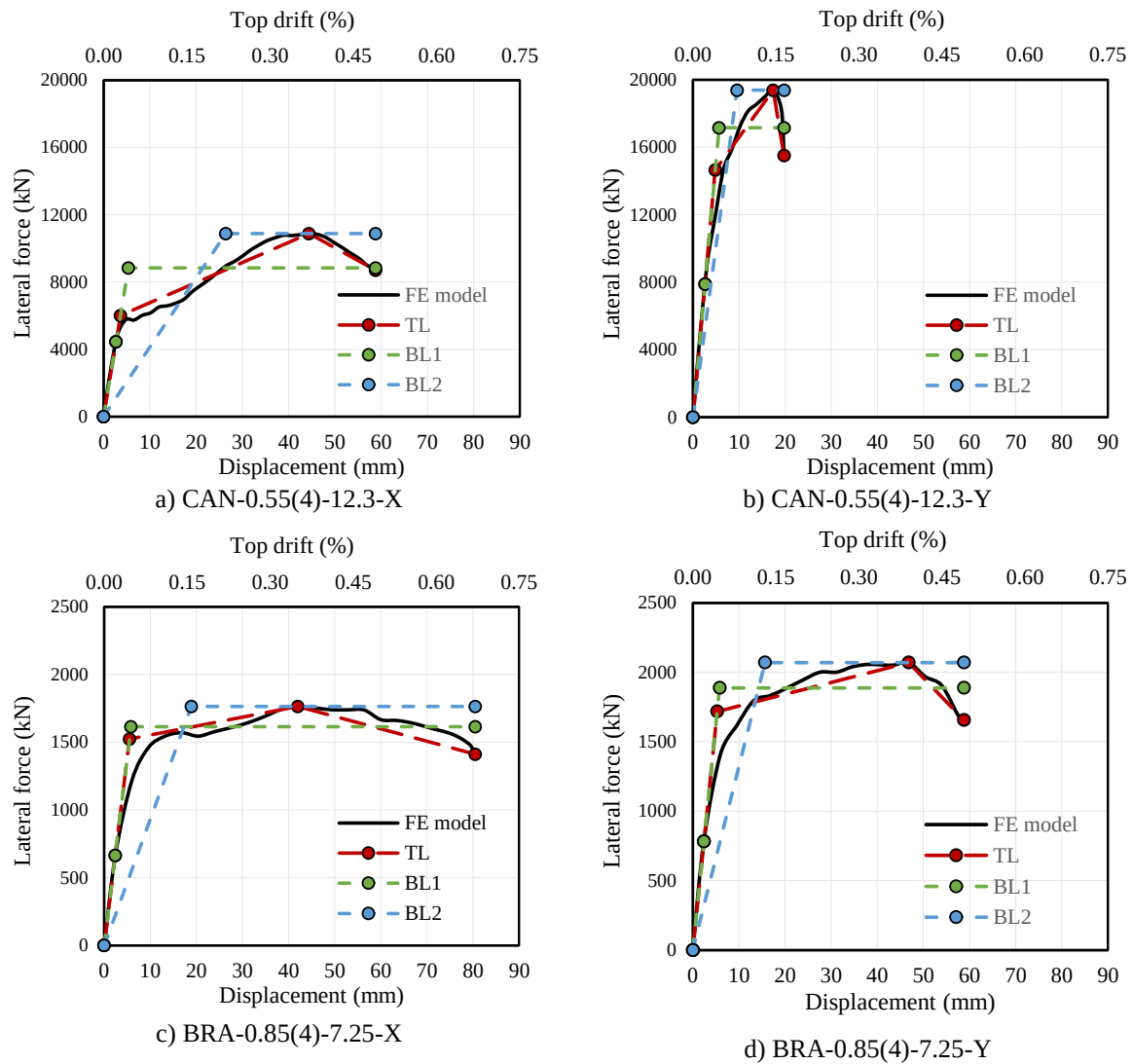
The first bilinear approach (BL1) is depicted in Figure 140b. This idealized is based on a bilinear elastic-perfectly plastic model with d_u corresponding to $0.8 \cdot V_{max}$ in the post-peak stage. As in the TL approach, in this method the initial elastic stiffness results from intersecting the response curve with the secant stiffness line [at the point (d_{el}, V_{el})]. The yielding displacement, d_y , is obtained to ensure equivalent energy dissipation under the actual and idealized curves.

In the second bilinear approach (BL2), as illustrated in Figure 140c, the response is idealized as a bilinear elastic-perfectly plastic curve, with a yield force plateau ($V_y = V_{max}$) and an ultimate displacement, d_u , corresponding to a load of $0.8 \cdot V_{max}$ in the post-peak stage. The yield displacement is determined based on the energy-equivalent response.

The idealized curves for the different approaches, along with the actual (numerical) response, are shown in Figure 141 for each building model.

As pointed out by Medeiros et al. (2022a) and Palhares et al. (2025), significant differences among the idealized responses are highlighted in Figure 141. The TL approach closely matches the actual response, maintaining three key characteristics: the initial elastic stiffness, the lateral force capacity, and the post-peak decay. The BL1 approach provides the same initial elastic stiffness as the actual response but underestimates the lateral force capacity of the wall by about 20%. In contrast, the BL2 approach, despite matching the lateral force capacity, significantly underestimates the initial stiffness and provides a less ductile response than the other approaches, making it the most conservative approach in seismic parameter calculation.

Figure 141: Actual and idealized responses for the building models.



Source: Author.

For an analysis more on the safe side, disregarding the post-peak phase is an option, as this would disregard the additional ductility in the post-peak behavior. In this scenario, V_u and d_u are set equal to V_{max} and d_{Vmax} , respectively, with total energy under the actual and idealized curves limited to the peak load point.

The values of the overstrength factor, Ω ; ductility-related force modification factor, R_d ; and response force modification factor, R , defined as the product of R_d and Ω (denoted as R_0 by the NBCC (2020)), (see Figure 140) are presented in Figure 142 for the three approaches. Additionally, the values of Ω , R_d , and R without considering the post-peak stage are presented in Figure 143. The estimations without considering the post-peak are indicated by the symbol * in superscript, e.g., TL^* , $BL1^*$, and $BL2^*$.

Figure 142: Estimated values of seismic parameters R , R_d , and Ω .

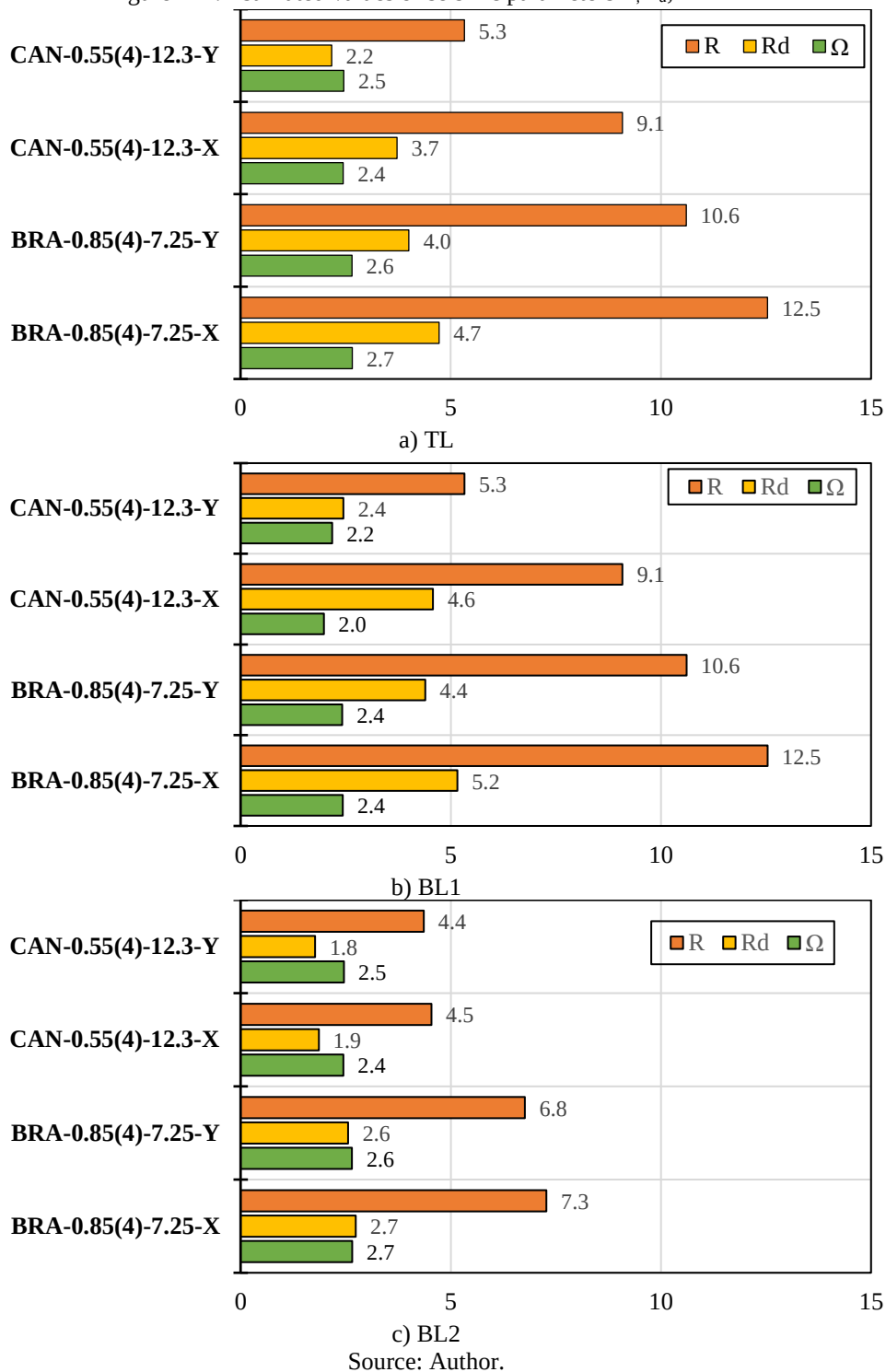
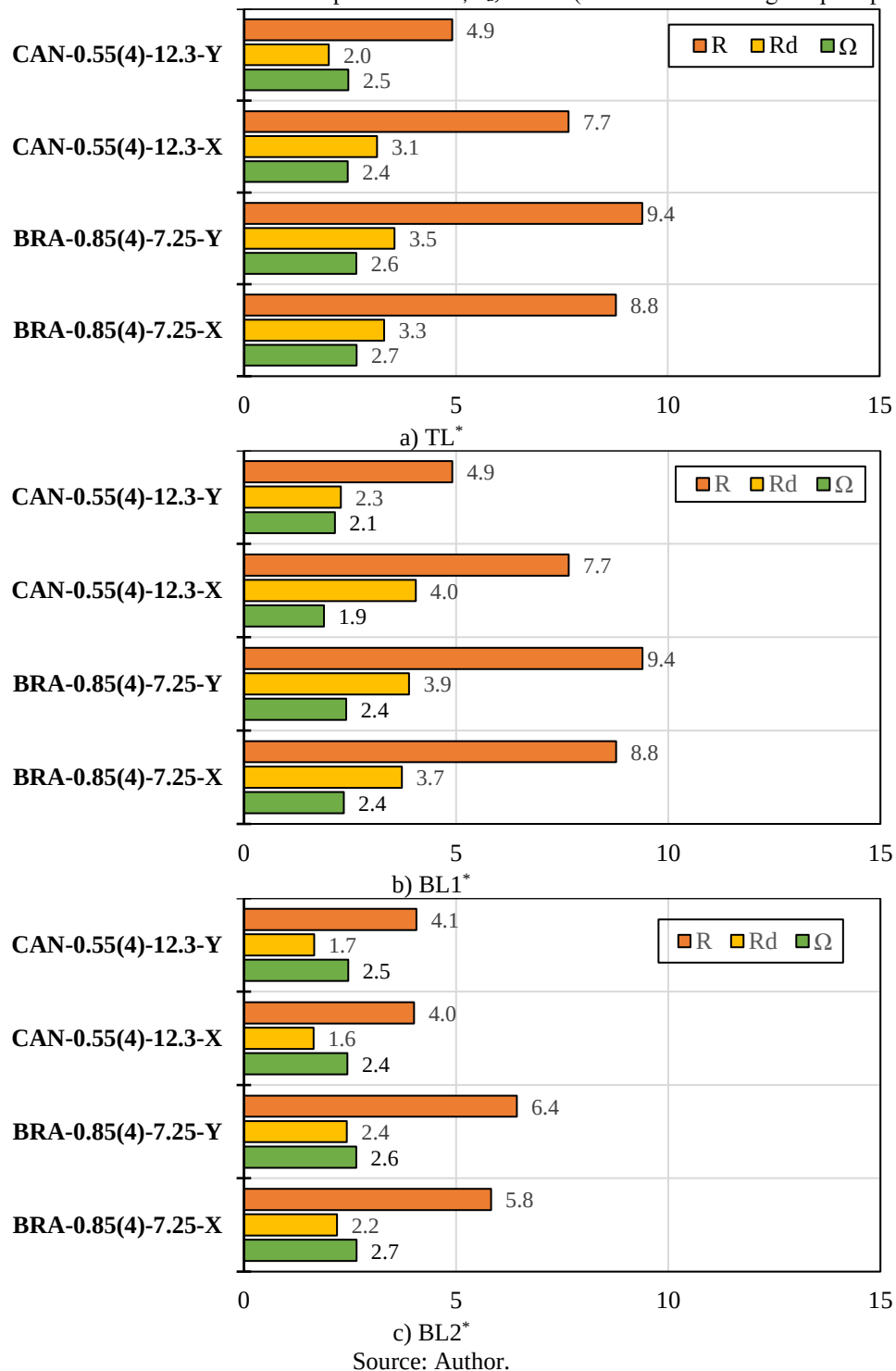


Figure 143: Estimated values of seismic parameters R , R_d , and Ω (without considering the post-peak).



Considerable high values of the response modification factor, R , were obtained for all building models presented here. These findings demonstrate that the seismic performance of geometrically regular concrete masonry buildings with in-plane rigid floors can exceed the expectations presented in current design standards, particularly for the TL and BL1 approaches. The NBCC (2020) adopts values between 1.5 and 3.0 for the ductility-related modification

factor, R_d , and an overstrength factor, R_o , equal to 1.5 for systems with masonry shear walls. The ASCE/SEI 7 (2022) indicates values of R ranging from 1.5 to 5.5. Differences in masonry systems considered in the Canadian and American standards are associated with reinforcement details. The NBCC (2020) categorizes them into conventional ($R_d = 1.5$, $R_o = 1.5$, $R = 2.25$), moderately ductile ($R_d = 2.0$, $R_o = 1.5$, $R = 3.0$), and ductile ($R_d = 3.0$, $R_o = 1.5$, $R = 4.5$), while ASCE/SEI 7 (2022) classifies shear walls into ordinary plain ($R = 1.5$), ordinary reinforced ($R = 2.0$), intermediate reinforced ($R = 3.5$), and special reinforced ($R = 5.0$) categories. The standards specify limits for the maximum uniform reinforcement spacing and the minimum rebar diameter for each category classification. The Brazilian standard ABNT NBR 15421 (2023) specifies a seismic force reduction factor $R = 1.5$ for masonry structures with usual detailing, referring to the detailing prescribed by the masonry code ABNT NBR 16868 (2020). However, the ABNT NBR 16868 (2020) does not define seismic-specific detailing categories or minimum requirements for seismic performance, resulting in ambiguity regarding what qualifies as “usual detailing” in the context of seismic design.

The significantly higher R values observed in this study, compared to those provided in seismic standards, align with the values found by Medeiros et al. (2022a), for single multi-storey, in-plane loaded PGRM walls, and Palhares et al. (2025), for three-dimensional PGRM structural systems. This discrepancy may be in part because parameters in standards are often derived from results obtained for isolated walls, therefore disregarding the extra resistance related to 3D effects in the overall behavior of the entire building. Considering the global response allows a significant reserve of capacity to be exploited both at and after the post-peak. In fact, adopting idealization approaches like BL2 may lead to more conservative results when applied to single wall design, but considering the complete structural system would yield a more realistic representation.

Building B2 consistently exhibits higher R values than building B1 in both X and Y directions. The maximum R value of building B1, achieved with TL and BL1 approaches, is 9.1, whereas the maximum R of building B2 is 12.5, both in its respective X direction. The BL2 approach yields the minimum R value: 4.4 for the CAN-0.55(4)-12.3-Y and 6.8 for the BRA-0.85(4)-7.25-Y.

Disregarding the post-peak phase reduces R values across all approaches for both buildings, though building B2 still maintains higher R values than building B1. In this case, the maximum and minimum R values (corresponding to TL* and BL2* approaches) are respectively 7.7 and 4.0 for building B1 and 9.4 and 5.8 for building B2. This indicates that the post-peak

significantly impacts R values, particularly for building B2.

Although both TL and BL1 approaches produce identical R values, the TL approach yields lower R_d values but higher Ω values, whereas the BL2 approach is the most conservative, providing the lowest R values.

Overall, the estimated seismic parameters denote that building B2's structural layout allows greater force reduction and enhanced ductility and overstrength, so better seismic performance. This higher ductility can be attributed to several key factors related to the structural layout, which significantly influence the overall seismic performance. Morandi et al. (2022) highlight a wide range of variability in the seismic performance of URM buildings, largely depending on architectural configurations.

A relevant factor is the presence of flanged walls in building B2's structural layout, which has been shown to improve the ductility and resistance of in-plane PGRM walls (Elmeligy; AbdelRahman; Galal, 2025; Palhares et al., 2025; Shedid; El-Dakhakhni; Drysdale, 2011). The flanges enhance the ability of walls to sustain larger displacements without significant strength loss, even under shear mode failure. Additionally, this layout characteristic plays a crucial role in structural redundancy. The redundancy ensures that multiple walls share lateral forces, enhancing the building's resilience to seismic loads.

Additionally, the lower level of axial load in the walls of building B2 contributes significantly to their ductile response. Previous studies (Davidson, 1996; Medeiros et al., 2022a; Palhares et al., 2023, 2024) have demonstrated that higher axial stress can adversely affect ductility and limit energy dissipation, underscoring the importance of controlling axial loads for seismic performance. High axial load typically increases the ultimate strength of walls but simultaneously reduces the walls' ability to deform, resulting in a less ductile response.

To further enhance ductility for structural layouts like building B1, it is reasonable to consider alternative structural layouts (such as with better distribution of loadbearing walls, especially connecting orthogonal walls in a flanged wall configuration); adjustments in hollow-core slab (HCS) orientations to better control axial load levels on key shear walls. These alternatives would enhance energy dissipation and inelastic deformation capacity. Additionally, structural modifications could also be considered, such as implementing boundary elements or confined ends in the design of critical walls, as shown by Aly and Galal (2020).

6.5.2 Maximum attainable design spectral acceleration, $S(T)_{max}$, from pushover analyses

Most seismic standards define seismic loading using elastic acceleration response spectra based on single-degree-of-freedom (SDOF) systems. Since the seismic demand from an elastic spectrum represents the acceleration values for a structure assumed to behave elastically during an earthquake, the maximum spectral acceleration a structure can withstand, $S(T)_{max}$, can be determined from the pushover analyses. This is achieved by the principle of equal elastic energy (illustrated as the area within the gray trapezoid in Figure 140) and inelastic energy (represented by the red region in the idealization approaches for equivalent elastoplastic response). The TL, BL1 and BL2 approaches, previously employed to derive seismic parameters using idealized force–displacement curves, are used to estimate $S(T)_{max}$, as defined by Equation 37. This procedure is aligned with the equivalent static lateral force method prescribed by seismic design standards (ABNT, 2023; NBCC, 2020; EN 1998-1, 2004), in which the total base shear is distributed among the floors of the building for each principal direction. While seismic loading is defined using elastic acceleration response spectra for a single-degree-of-freedom (SDOF) system, this approach may loss accuracy for taller buildings with pronounced multi-degree-of-freedom (MDOF) behavior and, therefore, the results should be carefully interpreted.

$$S(T)_{max} = \frac{V_e}{M_v \cdot W} = \frac{V_{max} \cdot R_d}{M_v \cdot W} \quad \text{Eq. 37}$$

where: V_e is the maximum base shear force if the structure remains elastic; V_{max} is the maximum base shear from the idealized force–displacement curves; R_d is the ductility-related force reduction factor; W is the seismic weight of the building; and M_v is a factor specified in NBCC (2020) to account for higher modes effects. For buildings with $T < 1.0$ second, $M_v = 1.0$. The buildings being analyzed have T around 0.2 s. Therefore, M_v does not impact the results. For tall buildings ($T > 1.0$ second), the effects of the higher modes of vibration should be considered (T.F. Paret, K.K. Sasaki, D.H. Eilbeck, 1996).

The maximum design spectral acceleration, $S(T)_{max}$, can be compared with the design spectral acceleration, $S(T)$, for different regions. Soil conditions are critical in shaping the design spectrum and, consequently, the $S(T)$ value. Rocky or stiff soils typically result in lower ground motion amplification, whereas soft soils tend to amplify ground motion, leading to higher spectral accelerations. Seismic standards apply site-specific amplification factors to the spectral accelerations to consider for these effects. In general, $S(T)$ is calculated using Equation

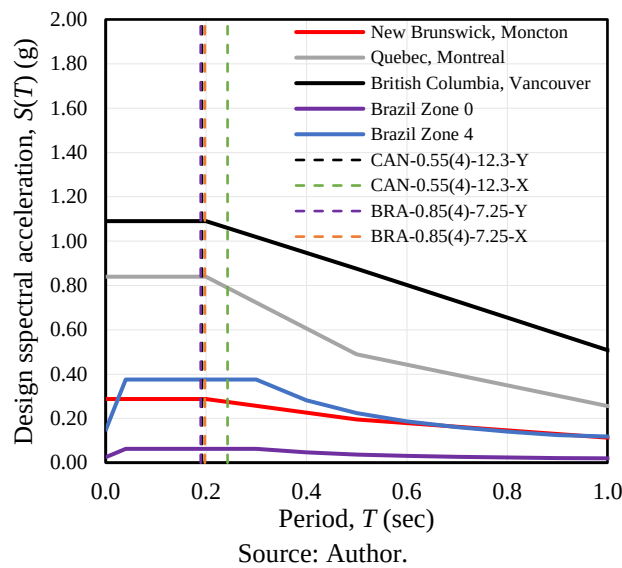
38.

$$S(T) = F(T) \cdot S_a(T) \quad \text{Eq. 38}$$

where: $S_a(T)$ is the 5%-damped spectral response acceleration provided by the seismic design standards, such as NBCC (2020) and NBR 15421 (2023), for the reference site (corresponding to very dense soil or soft rock); $F(T)$ represents the site coefficient, with $F(T) = 1.0$ for the reference case. Here, the reference site is considered; therefore, $S(T) = S_a(T)$.

Design spectral acceleration, $S(T)$, values for different regions are obtained and compared against the $S(T)_{max}$. Figure 144 presents the design spectral acceleration profiles for three Canadian regions (Moncton, Montreal and Vancouver) and Brazil's lowest and highest seismic zones (Zone 0 and Zone 4), alongside with the fundamental period, T , values for both buildings in each principal direction.

Figure 144: Design spectral acceleration profiles, $S(T)$, across Canadian and Brazilian seismic regions.

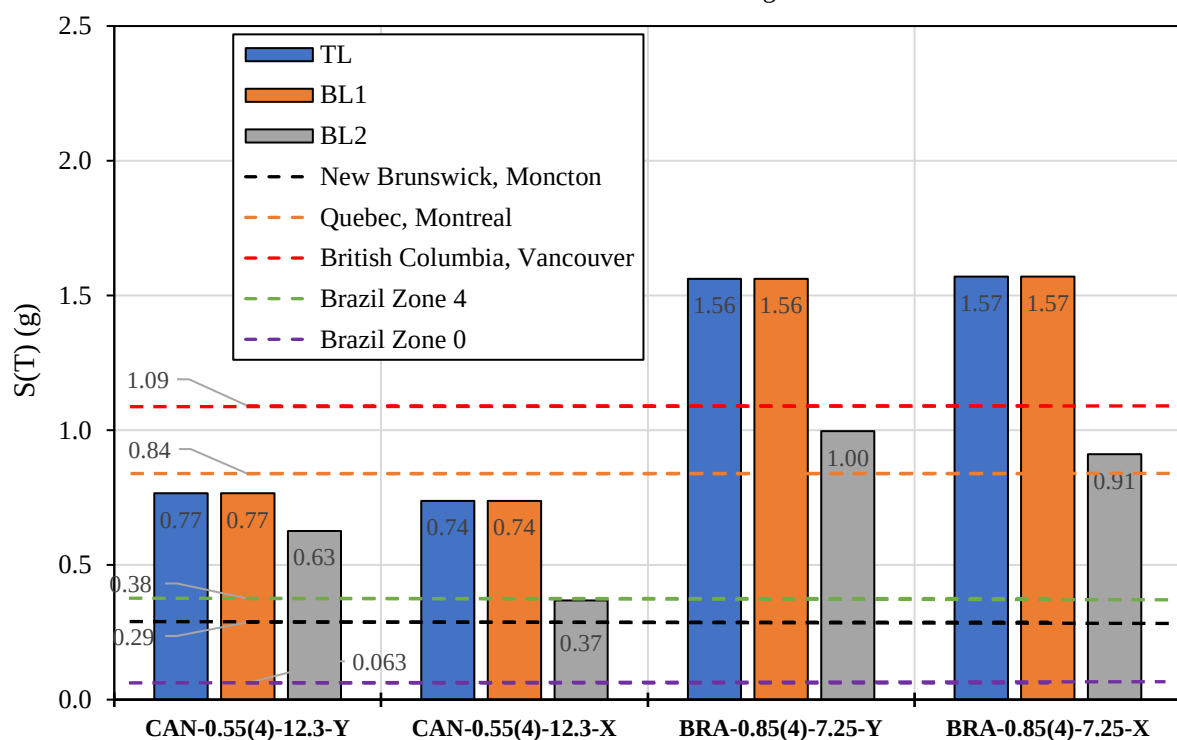


From Figure 144, it is evident that the fundamental period, T , of the analyzed buildings is within the range where design spectral acceleration demands, $S(T)$, are generally at their highest in the design spectrum. This indicates a critical period range for mid-rise buildings in seismic regions as they experience peak demand levels. For instance, the four-storey buildings analyzed here have global translation periods of around 0.2 s in both directions. These values correspond to the initial, high-demand portion of the spectral curves, particularly for regions like Vancouver, which exhibits the highest design spectral acceleration (1.09 g), followed by Montreal, Brazil's Zone 4, Moncton, and Brazil's Zone 0. If these buildings had longer fundamental periods, such as those found in taller structures, they would experience lower

seismic demands as $S(T)$ decreases with increasing periods. Thus, the mid-rise buildings presented here are exposed to relatively higher design spectral acceleration demands due to their fundamental periods being within the peak of the design spectra.

A comparison of the $S(T)_{max}$ values for the buildings, using the TL, BL1, and BL2 approaches, alongside with the seismic demand, $S(T)$, for the Canadian regions (Moncton, Montreal and Vancouver) and Brazilian zones is shown in Figure 145 (considering post-peak behavior) and Figure 146 (disregarding post-peak behavior).

Figure 145: Maximum spectral acceleration, $S(T)_{max}$ (considering post-peak behavior) compared to seismic demand for Canadian and Brazilian regions.



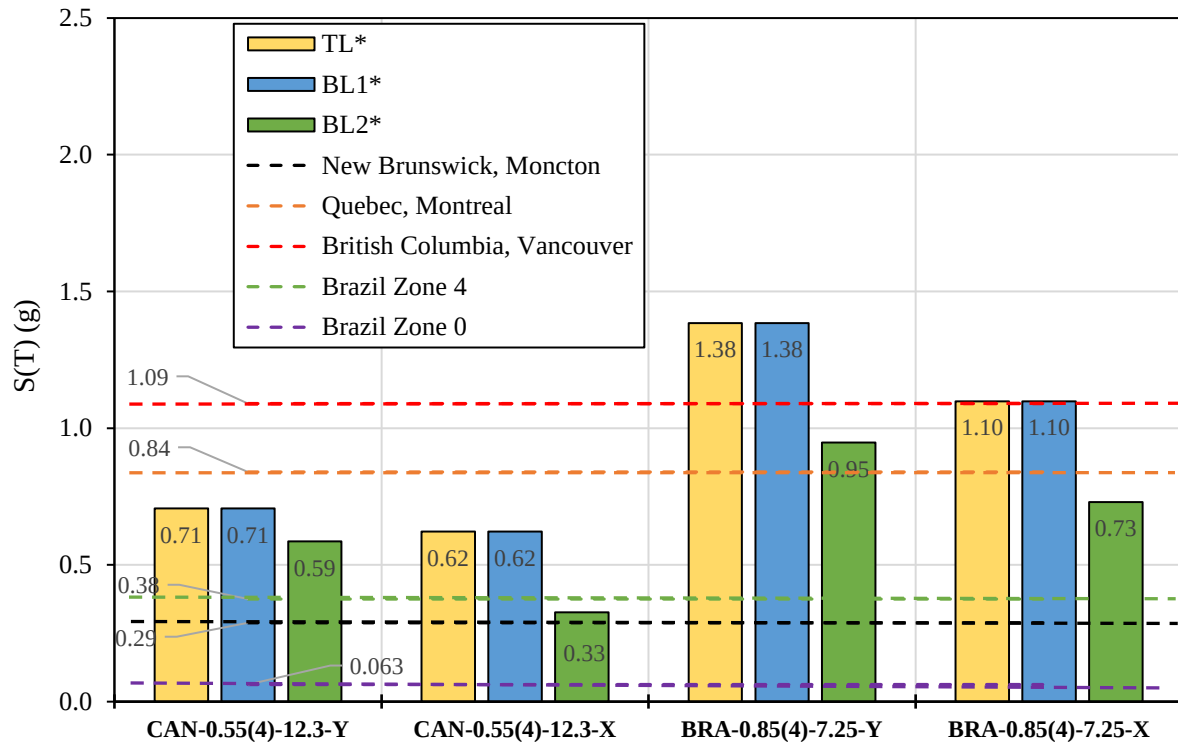
Source: Author.

Building B2 clearly outperforms building B1, reaching an $S(T)_{max}$ of 1.57g in the X direction, while building B1 achieves a maximum acceleration of only 0.77g in the Y direction. The $S(T)_{max}$ values of building B2 are considerably high since, under the TL and BL1 approaches, they surpass Vancouver's seismic demands — a region considered to have high seismicity — regardless of considering post-peak behavior. The TL and BL1 approaches yield higher $S(T)_{max}$ values than the more conservative BL2 approach.

Building B1, designed for the Moncton region, meets the local seismic demands under all approaches considered, including the most conservative one, BL2*. However, it does not adequately satisfy the higher seismic requirements of Montreal and Vancouver, even under the

TL and BL1 approaches with post-peak behavior. This suggests that further design adjustments would be necessary for building B1 to be suitable for higher seismicity regions.

Figure 146: Maximum spectral acceleration, $S(T)_{max}$ (disregarding post-peak behavior) compared to seismic demand for Canadian and Brazilian regions.



Source: Author.

This marked difference in $S(T)_{max}$ values between the two buildings underscores the substantial impact of layout configuration (such as flanged walls and reduced axial loads due to better distributed walls) on seismic performance. These findings confirm the inherent ductility and good seismic performance of building B2, as previously highlighted from the values of the seismic force reduction factors. This performance is primarily attributed to the effective layout of the building, which allows enhancing its capacity to withstand higher seismic demands, despite the absence of specific seismic reinforcement detailing. This insight could have practical implications for seismic design, potentially allowing for more adaptable reinforcement detailing requirements while maintaining structural performance.

6.5.3 Capacity Spectrum Method (CSM)

The Capacity Spectrum Method (CSM) is a widely used approach for assessing the seismic performance of structures, originally developed by Freeman (1998). This method compares the structure's capacity with the building demand due to the earthquake ground motion.

In the CSM, as described by Freeman (1998), the pushover curve obtained from nonlinear static analysis is converted into a format for direct comparison with the seismic demand spectrum. This conversion translates the base shear, V , and roof displacement, d , of the multi-degree-of-freedom (MDOF) system into spectral acceleration, S_a , and spectral displacement, S_d , for an equivalent single-degree-of-freedom (SDOF) system. The resulting plot, the ADRS (Acceleration-Displacement Response Spectrum), allows a visual comparison between the structure's capacity and seismic demand.

The conversion of the capacity curve to the S_d - S_a capacity spectrum involves the dynamic characteristics of the structure: period, T , mode shape, ϕ_i , and lumped floor mass, m_i . The conversion from base shear, V , to spectral acceleration, S_a , is given by Equation 40.

$$S_a = \frac{V}{\alpha_m \cdot W} \quad \text{Eq. 40}$$

where: W is the total seismic weight of the building; and α_m is the effective mass ratio, a factor that adjusts the proportion of the building's mass that participates in the fundamental mode.

The spectral displacement, S_d , is calculated by converting the lateral displacement, d , of the MDOF system to the equivalent displacement of the SDOF system using the modal participation factor, Γ , as shown in Equation 41.

$$S_d = \frac{d}{\Gamma} \quad \text{Eq. 41}$$

The Γ factor describes how much of the structure's total mass effectively contributes to the displacement in the fundamental vibration mode and is calculated by Equation 42.

$$\Gamma = \frac{\sum m_x \cdot \phi_x}{\sum m_x \cdot \phi_x^2} \quad \text{Eq. 42}$$

where: m_x is the mass at level x , and ϕ_x is the mode shape displacement at level x .

Response spectra represent the seismic demand, typically based on 5% damping for elastic response. However, spectra corresponding to higher damping ratios can be used to better approximate inelastic behavior, since spectral acceleration is reduced due to increased damping

to reflect energy dissipated through inelastic deformations. The effective damping, ξ_{eff} , is calculated, according to Dwairi et al. (2007) and Priestley and Grant (2005), as shown in Equation 43.

$$\xi_{eff} = \xi_0 + c \cdot \left(\frac{\mu-1}{\pi \cdot \mu} \right) \quad \text{Eq. 43}$$

where: ξ_0 is the initial elastic damping (usually 5%); c is a constant that depends on the type of failure. Morandi et al. (2022) suggest values for c based on failure type (e.g. 0.44 for diagonal tension failure); and μ represents the ductility ratio, calculated as $\mu = \Delta_{max} / \Delta_y$, where Δ_{max} is the maximum displacement and Δ_y is the yield displacement (defined by the idealization approach for the pushover curve). Higher ductility ratios indicate more significant inelastic deformation, corresponding to higher effective damping.

To reduce the spectral acceleration for 5% damping, $S_{a,5\%}$, to the effective damping, Equation 44 applies a scaling factor, providing a reduced spectral acceleration, $S_{a,reduced}$, suitable for the inelastic response.

$$S_{a,reduced} = S_{a,5\%} \cdot \sqrt{\frac{5\%}{\xi_{eff}}} \quad \text{Eq. 44}$$

To create a demand spectrum in terms of spectral displacement, S_d , the typical Equation 45 is used (Chopra, 2012).

$$S_d = \frac{T^2}{4\pi} \cdot S_a \quad \text{Eq. 45}$$

The pushover curves of the three idealizations approaches (TL, BL1, and BL2) were used to calculate the ductility, μ , and consequently the effective damping, ξ_{eff} , for each building. Table 23 summarizes the μ and ξ_{eff} values for each approach, along with the associated dynamic characteristics of the building: modal participation factor, Γ ; and effective mass ratio, α_m .

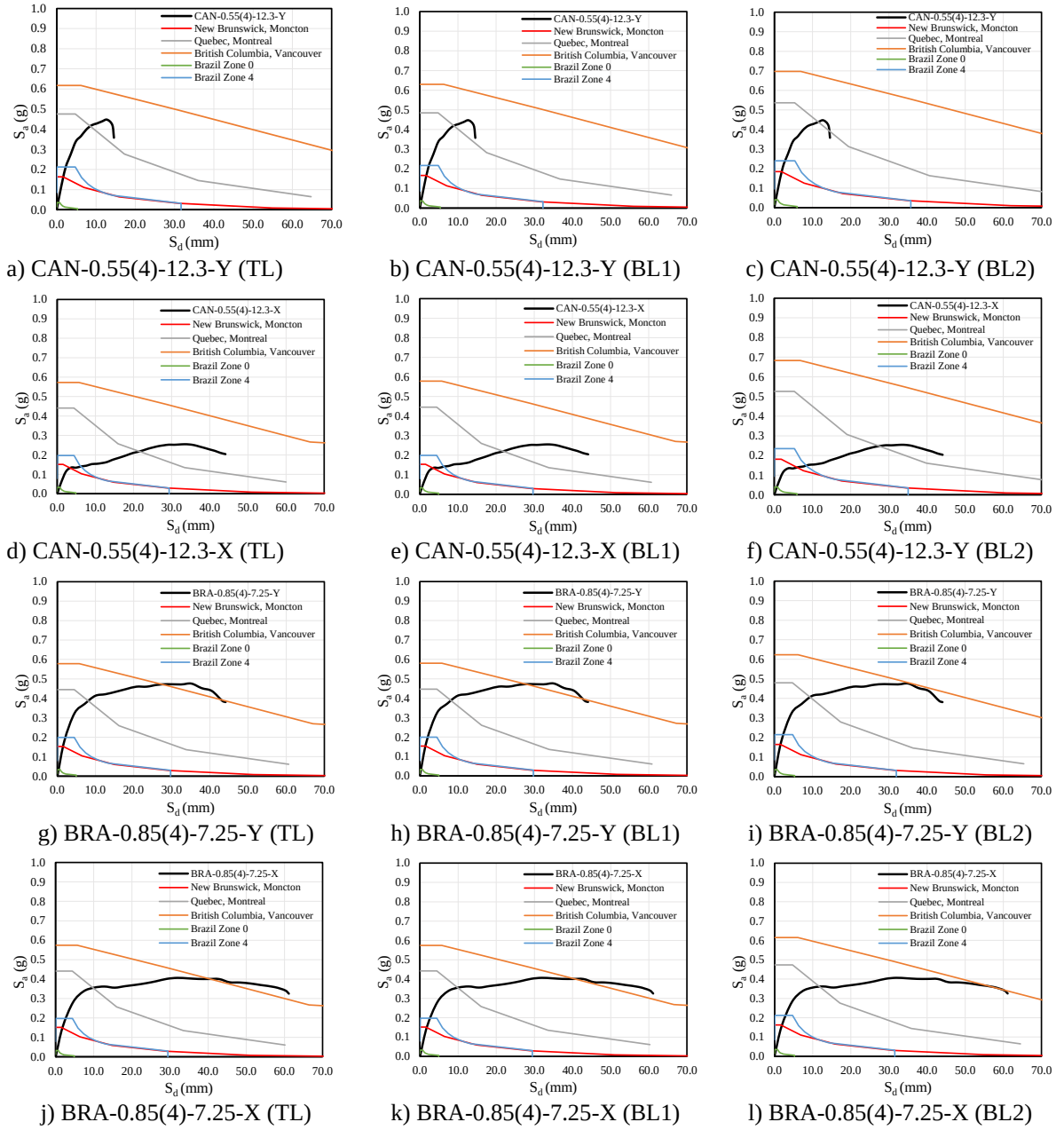
The CSM curves for each building (B1 and B2) and approach (TL, BL1 and BL2) are shown in Figure 147, comparing capacity spectra to demand spectra for Moncton, Montreal, Vancouver, Brazil Zone 0, and Brazil Zone 4.

Table 23: Ductility and effective damping for each idealization approach.

Building	M			ζ^{eff}			Γ	α
	TL	BL1	BL2	TL	BL1	BL2		
CAN-B-0.55(4)-12.3-X	4.1	3.5	2.1	0.156	0.150	0.122	1.36	0.791
CAN-B-0.55(4)-12.3-Y	16.2	11.0	2.2	0.181	0.177	0.127	1.33	0.779
BRA-0.85(4)-7.25-X	11.1	10.1	3.8	0.177	0.176	0.153	1.32	0.820
BRA-0.85(4)-7.25-Y	14.6	13.8	4.3	0.180	0.180	0.157	1.32	0.819

Source: Author.

Figure 147: Capacity spectrum against demand spectrum for the buildings.



Source: Author.

In all approaches (TL, BL1 and BL2), the capacity spectra of the CAN-B-0.55(4)-12.3-X/Y building are not intercepted by the demand spectrum for Vancouver (Figure 147(a-f)). Based on the provided capacity models, building B1 lacks sufficient capacity to withstand the seismic demands expected in Vancouver, denoting potential vulnerability and the need for design improvements of the building when located in such city. The demand spectrum for Montreal intersects the capacity spectrum near the maximum capacity for the TL and BL1 approaches and shifts slightly inward for the BL2 approach, indicating a more conservative estimate. This implies that building B1, particularly when using the more conservative BL2 approach, meets Montreal's seismic requirements only at the threshold of their capacity. The closer the intersection is to the maximum capacity, the smaller the safety margin, indicating a near-capacity response for these structures under Montreal's seismic demands.

For building B2, the capacity spectrum intersects the Vancouver demand spectrum at a displacement $S_d \approx 30$ mm and acceleration $S_a \approx 0.5g$ for BR-B-0.85(4)-7.25-Y, while $S_d \approx 40$ mm and $S_a \approx 0.4g$ for BR-B-0.85(4)-7.25-X, when TL and BL1 approaches are adopted. This intersection occurs near the maximum base shear capacity, suggesting that the building meets Vancouver's seismic demand with limited safety margins. However, for BL2 approach, the capacity spectrum is close to but does not intersect Vancouver's demand curve, indicating a potential deficit in capacity for high seismicity demands. Montreal's demand intersects the capacity curve around $S_d \approx 8$ mm and $S_a \approx 0.4$ g for BR-B-0.85(4)-7.25-Y and $S_d \approx 10$ mm and $S_a \approx 0.4$ g for BR-B-0.85(4)-7.25-X. This suggests that the building could withstand Montreal's demands with a moderate safety margin, indicating that it provides adequate seismic performance for moderate seismic demands. This is notable for a building without specific reinforcement for seismic design.

The demand spectra for Moncton and Brazilian Zones 0 and 4 intersect the capacity spectra of both buildings at the early elastic branch of the TL, BL1 and BL2 approaches. This early intersection indicates that the structural capacity is sufficient to meet the seismic demands of these regions with a considerable safety margin, meaning that the buildings are well within safe limits and will be operational when subjected to seismic activity typical of Moncton and Brazil.

Higher effective damping values in the TL and BL1 approaches generally improve the building's capacity to withstand seismic loads. The BL2 approach offers the most conservative estimate, reflecting a more rigorous assessment of the building's capacity to resist seismic

demands. While this conservatism may overstate building limitations in moderate seismicity regions, it ensures a crucial safety margin for higher seismic demands.

6.6 CONCLUSIONS

Two typical concrete masonry buildings from Canada (building B1) and Brazil (building B2) are comprehensively compared, highlighting the influence of the structural conception and layout on seismic performance. Notably, the buildings analyzed have fundamental periods within the peak plateau of the design spectra where seismic demands are highest, making these cases particularly critical for seismic performance evaluation.

Eigenvalue analyses revealed that seismic standard predictions for the fundamental period, T , are not consistently conservative. While ASCE/SEI 7 (2022) and Eurocode 8 (2004) generally estimate lower T values, on the safe side, for layouts like B1, NBCC (2020) tends to provide higher values for low- to mid-rise buildings, potentially leading to lower seismic forces and unconservative design demand estimation. For layouts like B2, standards such as ASCE/SEI 7 (2022) and Eurocode 8 (2004) often predict higher T values, particularly in taller buildings, which could result in unconservative designs. These results underscore the critical role of building height and structural layout in determining T values, highlighting the limitations of simplified height-based predictions like those in NBCC (2020) and NBR 15421 (2023).

Pushover analysis showed that building B1, despite its higher lateral force capacity and reinforcement level, is less seismically efficient than building B2, due to its significantly higher seismic weight per floor area. Furthermore, building B2's higher ductility and ultimate top drifts suggest better seismic performance overall, emphasizing its capacity to resist seismic forces more efficiently due to its lower weight per floor area. These findings highlight the importance of the relative seismic weight and layout efficiency in evaluating building seismic performance.

The estimates of the response modification factor, R , exceeded standard-prescribed values for both buildings, with B2 achieving consistently higher R values due to its higher ductility and overstrength. These findings suggest that seismic standards underestimate the performance of geometrically regular concrete masonry buildings. The assumption that walls meeting the same reinforcement detailing have the same R value needs to be reconsidered, as the ductility behavior is influenced by many other aspects. The choice of force-displacement curve idealization approach should be made to balance a realistic representation of capacity and necessary conservatism in design. While BL2 provides a conservative safety margin, it may underestimate the actual ductile capacity of the buildings, as evidenced by the TL approach.

Although building B2 was designed without explicit seismic provisions, the pushover analysis demonstrates a higher maximum spectral acceleration, $S(T)_{max}$, than building B1. In contrast, the B1 building, designed for earthquake resistance, has limitations under higher seismic demands, such as those in Vancouver. These findings underscore that an optimized structural layout plays a significant role in seismic performance, potentially more critical than reinforcement detailing and wall thickness. Notably, the Canadian layout offers the advantage of maintaining architectural flexibility while effectively meeting the seismic demands of its intended region.

Results from the Capacity Spectrum Method corroborate these findings. B1 lacks sufficient capacity to withstand Vancouver's seismic demands under all capacity curve idealization approaches (TL, BL1, and BL2) and has limited safety margins for Montreal. Conversely, B2 demonstrates higher capacity and meets Vancouver's demands under TL and BL1 approaches while performing adequately in Montreal with moderate safety margins. In low-seismicity regions, both buildings remain elastic, ensuring high safety margins.

While both buildings are suitable for low to moderate seismicity regions, B2 performed better in high seismicity scenarios, offering valuable insights for future design strategies. Its favorable structural configuration — including well-distributed walls, flanged connections, and lower axial loads — proves essential for enhanced seismic resilience and serves as a benchmark for improving building design.

It is important to emphasize that the findings of this study are specific to the buildings evaluated and should not be generalized to all Canadian or Brazilian masonry buildings. Future research should expand the scope of analysis to include taller and irregular buildings and incorporate torsional effects for a more comprehensive understanding of seismic behavior. Additionally, implementing advanced analysis methods such as Non-Linear Time History (NLTH) analysis or Incremental Dynamic Analysis (IDA), as outlined in FEMA P695 (2009) (FEMA, 2009), could provide deeper insights into seismic performance. However, these approaches require significant computational resources.

7. CONCLUSIONS AND FUTURE RESEARCH

This thesis aimed to expand the knowledge about the seismic behavior of concrete masonry structures, focusing on partially grouted reinforced masonry (PGRM) systems. By benchmarking different numerical modeling approaches in simulating experimental datasets from the author's research group and those found in the literature, this study provides insights into various features of masonry structures and their seismic behavior. The research followed a scale-up approach, analyzing individual walls and progressively advancing to wall systems and entire buildings to investigate wall configurations, structural layouts, and regional seismic design provisions. The specific conclusions from the analyses carried out in the different chapters of the thesis, as well as suggestions for future research, are summarized below:

a) Chapter 3: A macro-modeling approach for non-linear analysis of multi-storey perforated masonry walls with grout and reinforcement concentrated at their pier ends

In this chapter, a simplified macro-modeling approach was proposed and validated to simulate the in-plane behavior of PGRM walls with vertical reinforcement concentrated at their ends, primarily governed by a diagonal shear mechanism. The key findings are:

- The modeling approach was validated concerning experimental walls representing realistic configurations of multi-storey masonry buildings with openings. It provided consistent predictions for force-displacement responses, cracking patterns, and displacement profiles, with the advantage of low computational cost.
- The lateral resistance of the experimental walls was accurately predicted with the macro-models, with a maximum deviation of -5.0% from average experimental values.
- The macro-models satisfactorily estimated horizontal displacement profiles up to the peak load, aligning well with experimental results. In the post-peak, trends were captured, but some experimental variations were not fully matched, indicating areas for further refinement.
- The influence of axial stress levels on lateral force capacity was effectively estimated, with results aligning well with FEM simulations and analytical predictions for axial stress levels higher than those from experimental tests.
- The macro-model performed well for walls with vertical reinforcement concentrated at the pier ends and continuous across floors. However, further validation is needed to address

additional detailing scenarios, such as discontinuous reinforcement and varying distributions of the grouted region.

b) Chapter 4: A unified modeling approach for concrete masonry shear walls: conception, validation and insights

In this chapter, a FEM approach was validated concerning a wide range of masonry shear wall configurations, which includes both predominant failure modes: shear and flexure. The unified modeling strategy effectively combines macro-modeling for grouted masonry and simplified micro-modeling for ungrouted masonry, providing a robust yet efficient framework for analyzing masonry structures. The key conclusions are:

- The numerical model accurately simulated the lateral force-displacement response and damage pattern, including masonry cracking, vertical reinforcement strains, and ultimate failure modes. This was achieved for a wide range of wall configurations, including unreinforced masonry and partially and fully grouted reinforced masonry, demonstrating reliability across masonry typologies.
- The model accurately predicted key response parameters compared to experimental average counterparts across all wall types. The percentage differences were 4.1% for maximum lateral load, $V_{\max}$; -2.2% for displacement at maximum lateral load, $d_{V\max}$, -16.3% for ultimate displacement, d_{Vu} , -1.0% for secant stiffness, K_{sec} , and -19.2% for dissipated energy, U . These results are in line with the variability reported in the literature, indicating the suitability of the model to analyze concrete masonry structures comprehensively.
- A sensitivity analysis conducted on a reference PGRM wall model identified Young's modulus, compressive strength, and interface friction angle as the most influential parameters affecting force-displacement response.
- The proposed modeling approach incorporates distinctive features, including half-block mesh size, simplified interface elements, and requirement for few input parameters. Additionally, its validation, in many cases, provides a reliable basis for real-world applications.
- Recommendations of assumptions and criteria for input parameters were presented, supporting the usability of the approach for future research and applications.

c) Chapter 5: Seismic assessment of partially grouted and reinforced multi-storey masonry walls in 3D systems

The unified modeling approach established in Chapter 4 was extended in this chapter to three-dimensional building systems, focusing on multi-storey configurations. The following conclusions are drawn:

- Force-controlled pushover analyses revealed convergence issues, highlighting the complexity of capturing the building's complete nonlinear response. Displacement-controlled pushover analyses have made it possible to overcome these problems effectively by using displacement distributions derived from eigen mode shapes.
- Orthogonal walls (flanges) significantly increase stiffness and lateral capacity, particularly in multi-storey systems where flexural behavior plays a crucial role. Flanges increased V_{max} by approximately 35% and d_u by 25% in the three-storey configuration compared to the one-storey configuration. These findings underscore the importance of adequate interlocking between walls to improve performance under lateral loads.
- A higher axial load level ($20\% \cdot f_m$) increased V_{max} by 40–56% compared to walls with a lower axial load ($5\% \cdot f_m$). However, this enhancement in capacity impaired ductility. Furthermore, increasing the number of stories from one to four reduced K_{sec} by up to 68.9% for low axial load cases and 75.1% for high axial load cases, while d_u increased significantly by up to 330% among both load scenarios.
- A comparison of vertical reinforcement arrangements — concentrated at wall ends versus uniformly distributed — revealed similar stiffness and load capacity for both cases. Efficiency differences, measured by $V_{max} \cdot R$, range from -17.3% to 10.8%, denoting comparable performance. These results suggest that concentrated reinforcement can be as effective as distributed reinforcement and support its potential consideration in design standards such as TMS 402/602-22 and CSA S304-24.
- Significant differences in seismic performance parameters were observed when considering different capacity curve idealization methods. The trilinear (TL) approach provided the closest approximation to the actual wall behavior, representing the initial stiffness, lateral capacity, and post-peak response well. In contrast, although simpler, bilinear (BL) approaches were more conservative and underestimated ductility.
- High R values, ranging from 6.8 to 13.9 using the TL approach, were obtained depending on the structural system configuration. These results highlight the robust seismic performance of multi-storey PGRM wall systems. However further case studies involving real-

world buildings are needed to develop general recommendations for R values applicable in design standards.

d) Chapter 6: Seismic performance evaluation of concrete masonry buildings in different regional contexts: Canada vs. Brazil

This chapter provided a comparative analysis of two concrete masonry buildings, one from Canada (B1) and another from Brazil (B2), focusing on structural differences and seismic performance. The key conclusions are as follows:

- Eigenvalue analyses revealed that seismic standard estimations for the fundamental period, T , as provided by ASCE/SEI 7 (2022), Eurocode 8 (2004), NBCC (2020), and NBR 15421 (2023), are not consistently conservative. These findings underscore the significant influence of building height and structural layout on T values and highlight the need to refine height-based equations in design standards to improve accuracy and ensure safety.
- Pushover analysis revealed that building B2 outperformed B1 regarding seismic efficiency. However, the Canadian layout (B1) stands out for its ability to combine architectural flexibility with effective seismic performance, making it suitable for the intended seismic demands.
- Significant variation in R values was observed, exceeding standard-prescribed limits. For example, B1 had a maximum R of 9.1, while B2 reached 12.5 using the TL approach. This indicates that seismic standards may underestimate the performance of geometrically regular masonry buildings and should consider factors beyond reinforcement detailing, such as structural layout, to result in a more accurate and sustainable design.
- In the context of linear seismic design, the elastic model must accurately represent the initial lateral stiffness of the building. Furthermore, the “ultimate” limit state—assessed using elastic analysis—should correspond to a lateral load level that is at least equal to or greater than the elastic limit, V_{el} , which represents the seismic design load.
- Analysis results using the Capacity Spectrum Method show that B2 meets Vancouver’s seismic demands corresponding to the TL and BL1 approaches, while B1 has insufficient capacity in regions with high seismic demand. These findings confirm B2’s robust seismic performance, even if explicit seismic provisions were not considered.
- Although both B1 and B2 buildings are suitable for low to moderate seismicity regions, B2’s optimized structural configuration—including well-distributed walls, flanged connections, and lower axial load levels—has better seismic performance. These findings

provide valuable insights into improving seismic design, particularly for multi-storey masonry buildings in diverse seismic contexts.

e) Future research

The findings of this research are limited to the cases analyzed and do not cover other scenarios that may require further investigation or refinement. Therefore, the following suggestions for future research are proposed:

- Conduct large-scale experimental testing on multi-storey PGRM building systems to evaluate the influence of building height, reinforcement detailing, orthogonal wall interactions, and structural layout configurations on seismic performance.
- Propose additional modeling simplifications to assess building-scale seismic performance while efficiently capturing the complex interactions in PGRM structural systems under diverse conditions and ensuring computational efficiency.
- Evaluate the accuracy of simplified equations for estimating T in a wider range of masonry buildings. Suggest refinements to design standards and investigate the impact of T predictions on seismic force estimates for equivalent static force methods in real-world scenarios.
- Explore more diverse structural layout configurations, including tall and irregular buildings, to propose regional adjustments to seismic standards and refine R values for use in linear seismic design. Additionally, advanced methods such as Non-Linear Time History Analysis and Incremental Dynamic Analysis can provide deeper insights into seismic performance.

The findings of this thesis contribute significantly to the concrete masonry structural engineering field by improving the understanding of seismic behavior and informing best design practices. The proposed modeling approaches, validated against experimental datasets, offer reliable tools for assessing masonry systems under seismic loads. Furthermore, comparative seismic performance analysis in different regional contexts provides valuable insights for adapting seismic design standards to meet specific regional demands. Continued research in these areas will ensure safer, more sustainable masonry structures that effectively respond to the evolving challenges posed by seismic events.

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APPENDIX A – Actual and idealized numerical responses for all wall structural systems of Chapter 5

Figure A.1: Numerical actual and idealized responses for one-storey structural systems.

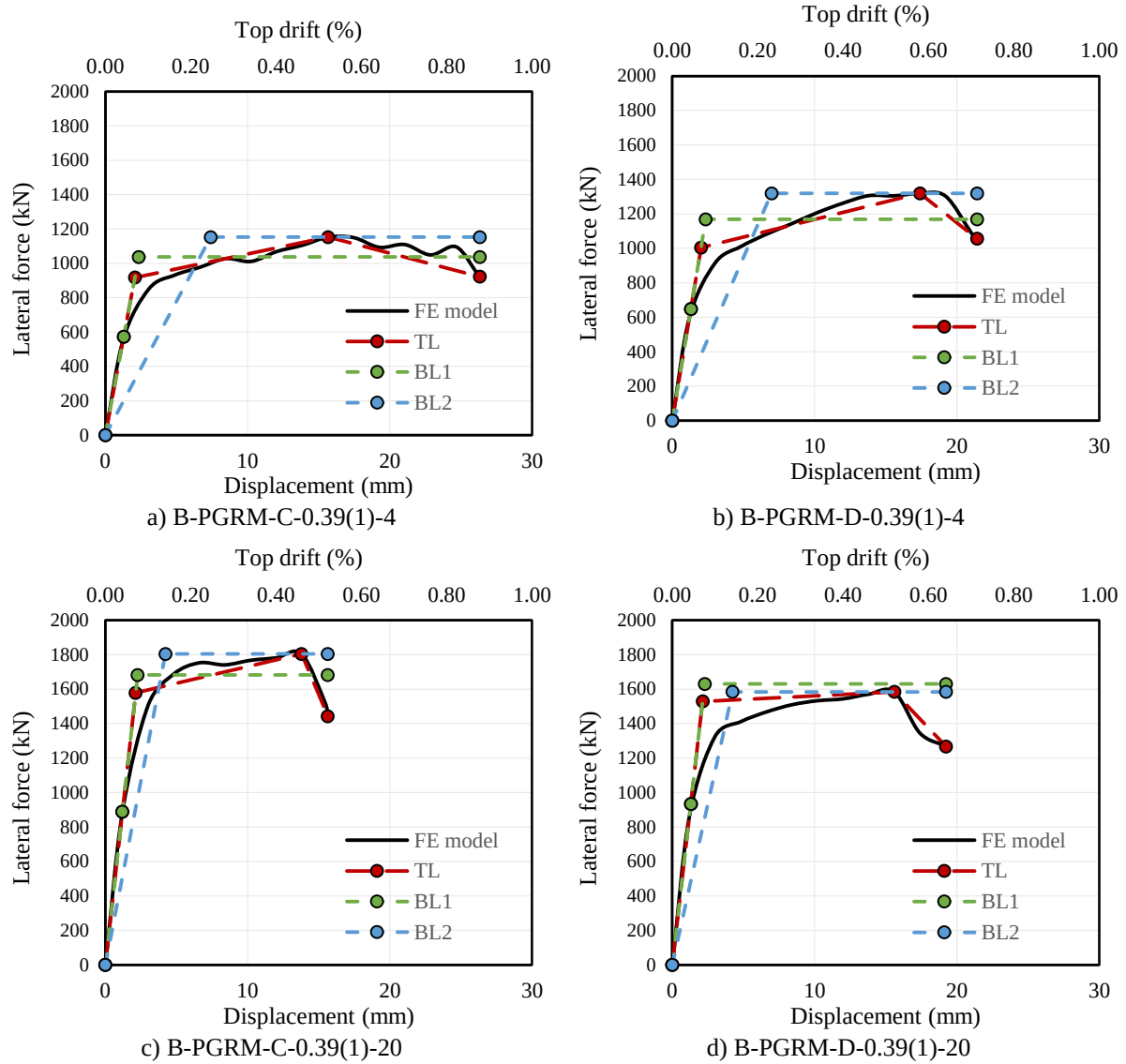


Figure A.2: Numerical actual and idealized responses for three-storey structural systems.

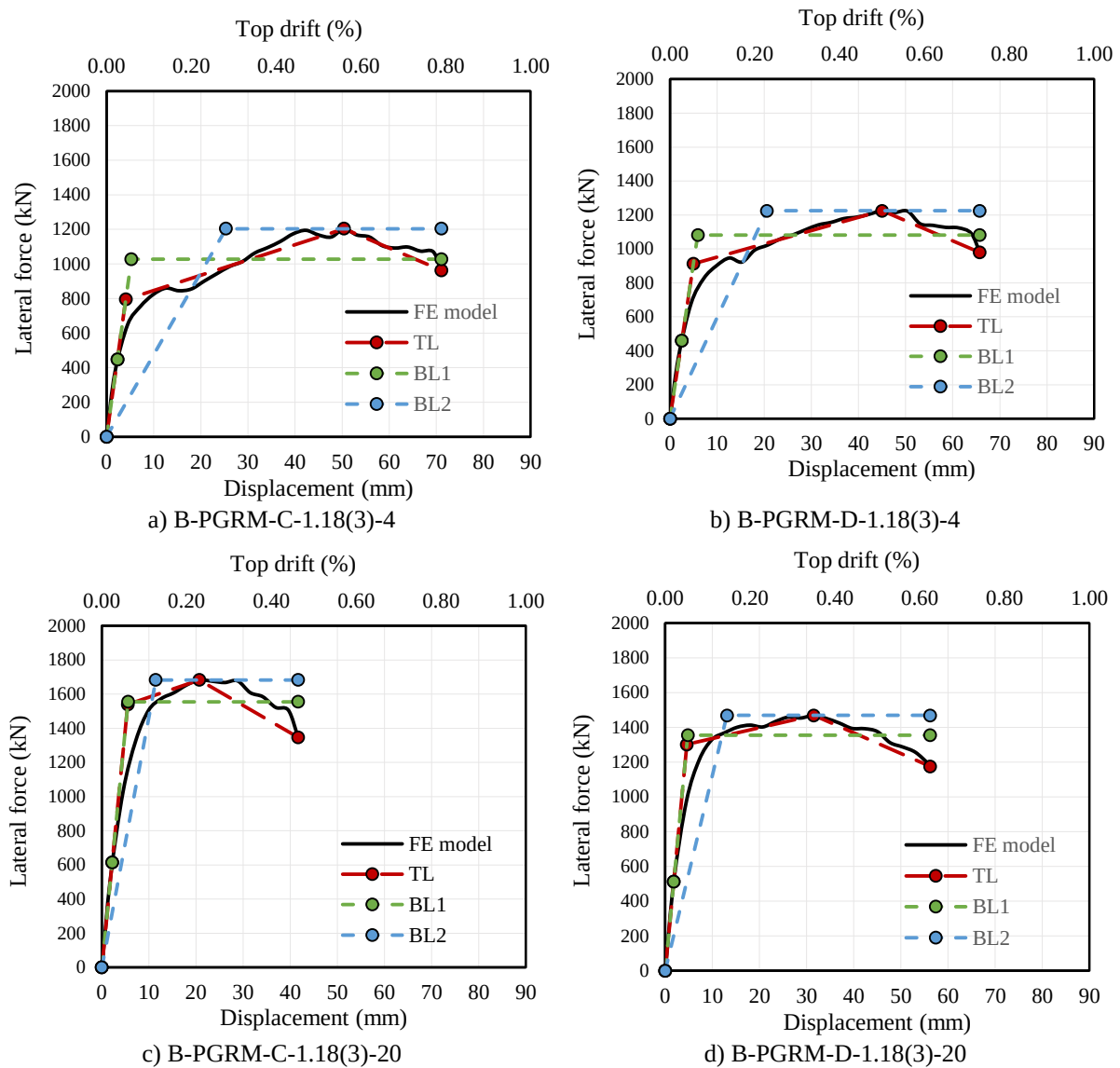


Figure A.3: Numerical actual and idealized responses for four-storey structural systems.

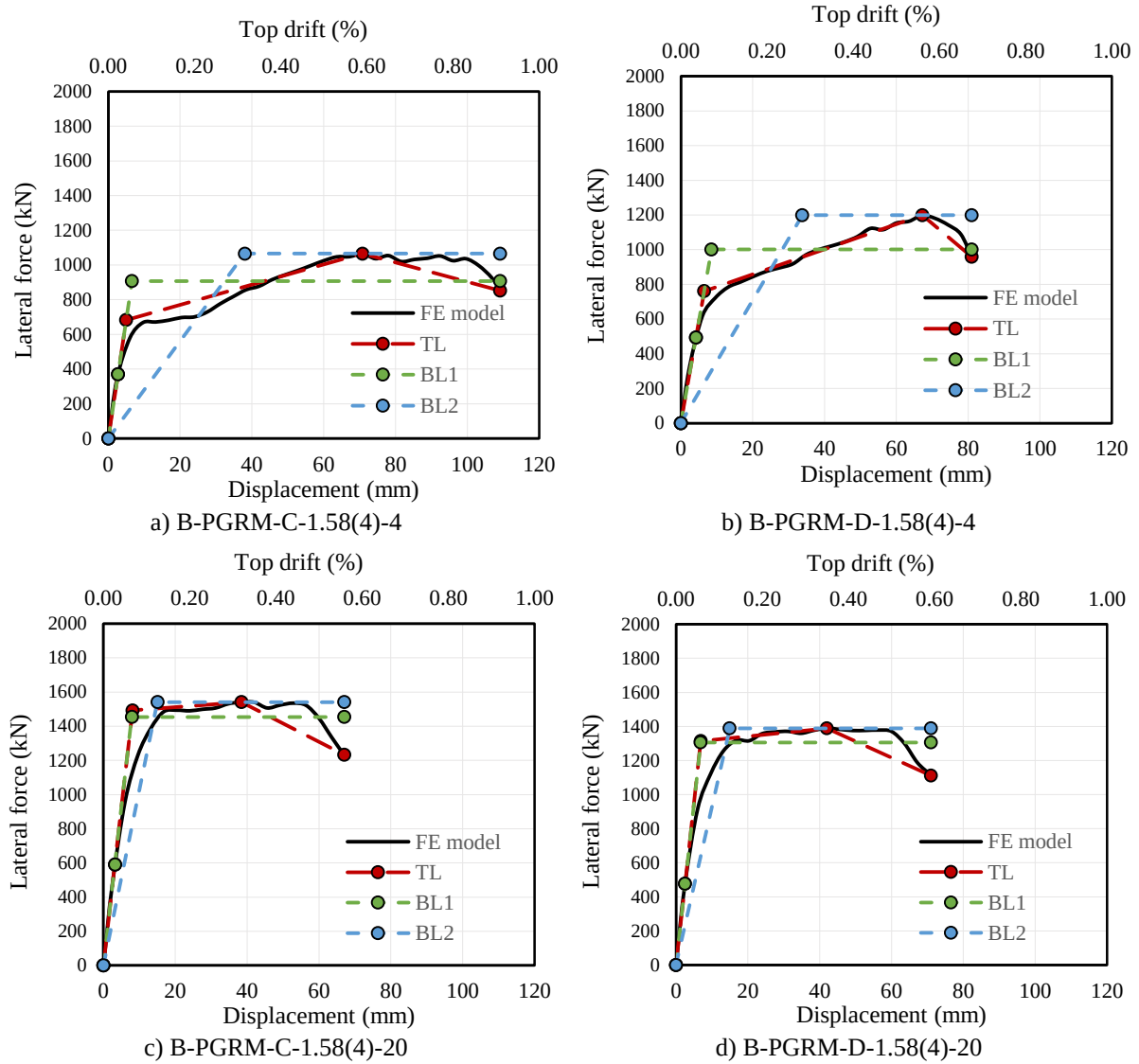
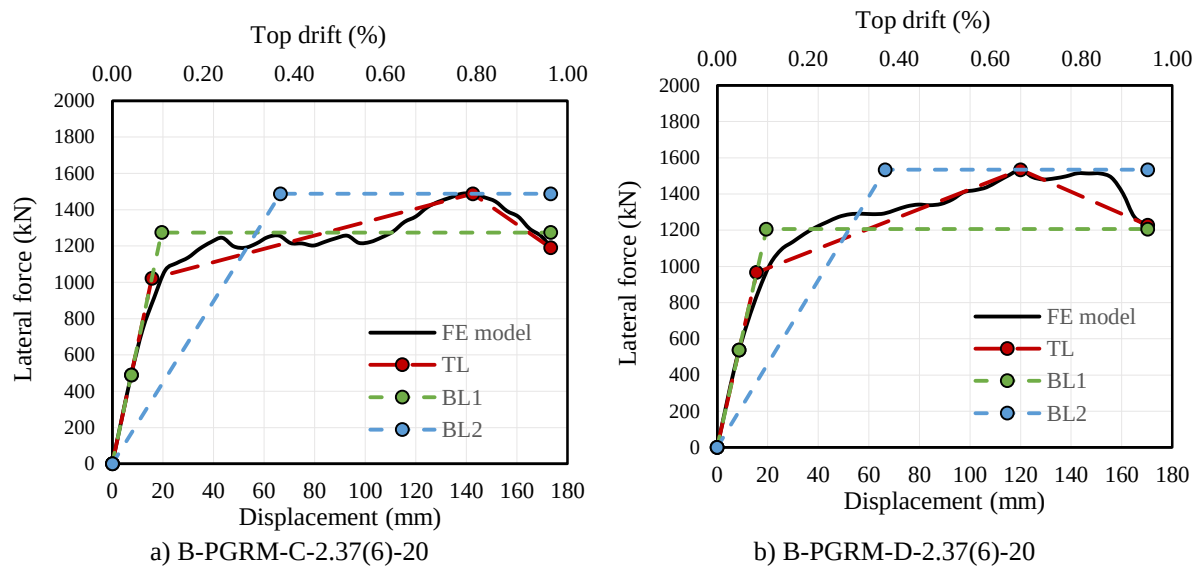


Figure A.4: Numerical actual and idealized responses for six-storey structural systems.



APPENDIX B – Seismic parameters obtained by other idealized response approaches of Chapter 5

Table B.1: Summary of structural responses (BL1 approach).

Model	V_{max} (kN)	$\sigma_{s@Vmax}$ (MPa)	d_{Vmax} (mm)	d_u (mm)	U (kN·mm)	K_{sec} (kN/mm)	T_n (s)	μ_d	R_d	Ω	R	C_d
B-PGRM-C-0.39(1)-4	1152.9	207.5	15.7	26.3	26086.8	441.0	0.05	11.2	4.6	1.8	8.4	20.3
B-PGRM-D-0.39(1)-4	1318.6	189.0	17.4	21.4	23653.5	497.4	0.05	9.1	4.2	1.8	7.5	16.5
B-PGRM-C-0.39(1)-20	1803.9	93.7	13.8	15.6	24407.2	740.8	0.05	6.9	3.6	1.9	6.8	13.0
B-PGRM-D-0.39(1)-20	1584.1	135.9	15.6	19.2	26398.0	717.8	0.05	8.5	4.0	1.7	7.0	14.8
B-PGRM-C-1.18(3)-4	1203.5	416.6	50.4	71.1	70308.8	194.7	0.12	13.5	5.1	2.3	11.7	30.9
B-PGRM-D-1.18(3)-4	1225.1	376.8	45.0	65.7	67909.4	184.0	0.12	11.2	4.6	2.4	10.9	26.3
B-PGRM-C-1.18(3)-20	1683.1	134.6	20.7	41.7	60461.3	279.1	0.12	7.5	3.7	2.5	9.5	18.9
B-PGRM-D-1.18(3)-20	1469.0	81.5	31.5	56.2	72866.9	284.3	0.12	11.8	4.8	2.6	12.6	31.2
B-PGRM-C-1.58(4)-4	1065.1	483.3	70.8	109.2	96006.2	137.3	0.17	16.5	5.7	2.4	13.9	40.4
B-PGRM-D-1.58(4)-4	1198.8	419.7	67.2	81.0	76803.4	117.4	0.17	9.5	4.2	2.0	8.6	19.3
B-PGRM-C-1.58(4)-20	1541.0	201.4	38.4	67.0	91635.6	184.6	0.17	8.5	4.0	2.5	9.8	20.9
B-PGRM-D-1.58(4)-20	1389.0	154.5	42.0	71.0	88281.0	191.0	0.17	10.4	4.4	2.7	12.2	28.4
B-PGRM-C-2.37(6)-20	1487.4	164.2	142.5	173.3	208397.9	65.3	0.26	8.9	4.1	2.6	10.7	23.1
B-PGRM-D-2.37(6)-20	1533.4	220.4	120.0	170.3	215156.9	61.8	0.27	8.7	4.1	2.2	9.1	19.6

Table B.2: Summary of structural responses (BL2 approach).

Model	V_{max} (kN)	$\sigma_{s@Vmax}$ (MPa)	d_{Vmax} (mm)	d_u (mm)	U (kN·mm)	K_{sec} (kN/mm)	T_n (s)	μ_d	R_d	Ω	R	C_d
B-PGRM-C-0.39(1)-4	1152.9	207.5	15.7	26.3	26086.8	441.0	0.05	3.6	2.5	2.0	5.0	7.1
B-PGRM-D-0.39(1)-4	1318.6	189.0	17.4	21.4	23653.5	497.4	0.05	3.1	2.3	2.0	4.6	6.3
B-PGRM-C-0.39(1)-20	1803.9	93.7	13.8	15.6	24407.2	740.8	0.05	3.7	2.5	2.0	5.1	7.5
B-PGRM-D-0.39(1)-20	1584.1	135.9	15.6	19.2	26398.0	717.8	0.05	4.6	2.8	1.7	4.8	7.7
B-PGRM-C-1.18(3)-4	1203.5	416.6	50.4	71.1	70308.8	194.7	0.12	2.8	2.1	2.7	5.8	7.5
B-PGRM-D-1.18(3)-4	1225.1	376.8	45.0	65.7	67909.4	184.0	0.12	3.2	2.3	2.7	6.2	8.5
B-PGRM-C-1.18(3)-20	1683.1	134.6	20.7	41.7	60461.3	279.1	0.12	3.6	2.5	2.7	6.9	10.0
B-PGRM-D-1.18(3)-20	1469.0	81.5	31.5	56.2	72866.9	284.3	0.12	4.3	2.8	2.9	7.9	12.3
B-PGRM-C-1.58(4)-4	1065.1	483.3	70.8	109.2	96006.2	137.3	0.17	2.9	2.2	2.9	6.3	8.2
B-PGRM-D-1.58(4)-4	1198.8	419.7	67.2	81.0	76803.4	117.4	0.17	2.4	1.9	2.4	4.7	5.8
B-PGRM-C-1.58(4)-20	1541.0	201.4	38.4	67.0	91635.6	184.6	0.17	4.5	2.8	2.6	7.3	11.6
B-PGRM-D-1.58(4)-20	1389.0	154.5	42.0	71.0	88281.0	191.0	0.17	4.8	2.9	2.9	8.5	13.8
B-PGRM-C-2.37(6)-20	1487.4	164.2	142.5	173.3	208397.9	65.3	0.26	2.6	2.1	3.0	6.2	7.9
B-PGRM-D-2.37(6)-20	1533.4	220.4	120.0	170.3	215156.9	61.8	0.27	2.6	2.0	2.9	5.8	7.3

Table B.3: Summary of structural responses (TL* approach).

Model	V_{max} (kN)	$\sigma_{s@Vmax}$ (MPa)	d_{Vmax} (mm)	d_u (mm)	U (kN·mm)	K_{sec} (kN/mm)	T_n (s)	μ_d	R_d	Ω	R	C_d
B-PGRM-C-0.39(1)-4	1152.9	207.5	15.7	26.3	26086.8	441.0	0.05	8.3	3.1	2.0	6.2	12.1
B-PGRM-D-0.39(1)-4	1318.6	189.0	17.4	21.4	23653.5	497.4	0.05	8.8	3.3	2.0	6.7	13.4
B-PGRM-C-0.39(1)-20	1803.9	93.7	13.8	15.6	24407.2	740.8	0.05	6.5	3.1	2.0	6.3	11.5
B-PGRM-D-0.39(1)-20	1584.1	135.9	15.6	19.2	26398.0	717.8	0.05	7.4	3.6	1.7	6.1	12.0
B-PGRM-C-1.18(3)-4	1203.5	416.6	50.4	71.1	70308.8	194.7	0.12	12.8	3.6	2.7	9.6	21.9
B-PGRM-D-1.18(3)-4	1225.1	376.8	45.0	65.7	67909.4	184.0	0.12	9.6	3.3	2.7	8.8	18.0
B-PGRM-C-1.18(3)-20	1683.1	134.6	20.7	41.7	60461.3	279.1	0.12	4.5	2.3	2.7	6.3	9.4
B-PGRM-D-1.18(3)-20	1469.0	81.5	31.5	56.2	72866.9	284.3	0.12	7.2	3.2	2.9	9.3	17.5
B-PGRM-C-1.58(4)-4	1065.1	483.3	70.8	109.2	96006.2	137.3	0.17	16.0	3.7	2.9	10.7	26.2
B-PGRM-D-1.58(4)-4	1198.8	419.7	67.2	81.0	76803.4	117.4	0.17	10.7	3.2	2.4	7.7	16.0
B-PGRM-C-1.58(4)-20	1541.0	201.4	38.4	67.0	91635.6	184.6	0.17	5.4	2.8	2.6	7.2	12.0
B-PGRM-D-1.58(4)-20	1389.0	154.5	42.0	71.0	88281.0	191.0	0.17	6.8	3.1	2.9	9.1	16.8
B-PGRM-C-2.37(6)-20	1487.4	164.2	142.5	173.3	208397.9	65.3	0.26	9.2	3.1	3.0	9.5	19.0
B-PGRM-D-2.37(6)-20	1533.4	220.4	120.0	170.3	215156.9	61.8	0.27	7.8	2.7	2.9	7.7	13.8

Table B.4: Summary of structural responses (BL1* approach).

Model	V_{max} (kN)	$\sigma_{s@Vmax}$ (MPa)	d_{Vmax} (mm)	d_u (mm)	U (kN·mm)	K_{sec} (kN/mm)	T_n (s)	μ_d	R_d	Ω	R	C_d
B-PGRM-C-0.39(1)-4	1152.9	207.5	15.7	26.3	26086.8	441.0	0.05	7.0	3.6	1.7	6.2	12.1
B-PGRM-D-0.39(1)-4	1318.6	189.0	17.4	21.4	23653.5	497.4	0.05	7.5	3.7	1.8	6.7	13.4
B-PGRM-C-0.39(1)-20	1803.9	93.7	13.8	15.6	24407.2	740.8	0.05	6.1	3.3	1.9	6.3	11.5
B-PGRM-D-0.39(1)-20	1584.1	135.9	15.6	19.2	26398.0	717.8	0.05	6.8	3.6	1.8	6.2	12.0
B-PGRM-C-1.18(3)-4	1203.5	416.6	50.4	71.1	70308.8	194.7	0.12	9.9	4.3	2.2	9.6	21.9
B-PGRM-D-1.18(3)-4	1225.1	376.8	45.0	65.7	67909.4	184.0	0.12	7.9	3.9	2.3	8.8	18.0
B-PGRM-C-1.18(3)-20	1683.1	134.6	20.7	41.7	60461.3	279.1	0.12	3.9	2.6	2.4	6.3	9.4
B-PGRM-D-1.18(3)-20	1469.0	81.5	31.5	56.2	72866.9	284.3	0.12	6.6	3.5	2.6	9.3	17.5
B-PGRM-C-1.58(4)-4	1065.1	483.3	70.8	109.2	96006.2	137.3	0.17	11.6	4.7	2.3	10.7	26.2
B-PGRM-D-1.58(4)-4	1198.8	419.7	67.2	81.0	76803.4	117.4	0.17	8.1	3.9	2.0	7.7	16.0
B-PGRM-C-1.58(4)-20	1541.0	201.4	38.4	67.0	91635.6	184.6	0.17	4.9	3.0	2.4	7.2	12.0
B-PGRM-D-1.58(4)-20	1389.0	154.5	42.0	71.0	88281.0	191.0	0.17	6.2	3.4	2.7	9.1	16.8
B-PGRM-C-2.37(6)-20	1487.4	164.2	142.5	173.3	208397.9	65.3	0.26	7.4	3.7	2.6	9.5	19.0
B-PGRM-D-2.37(6)-20	1533.4	220.4	120.0	170.3	215156.9	61.8	0.27	6.3	3.4	2.2	7.5	13.8

Table B.5: Summary of structural responses (BL2* approach).

Model	V_{max} (kN)	$\sigma_{s@Vmax}$ (MPa)	d_{Vmax} (mm)	d_u (mm)	U (kN·mm)	K_{sec} (kN/mm)	T_n (s)	μ_d	R_d	Ω	R	C_d
B-PGRM-C-0.39(1)-4	1152.9	207.5	15.7	26.3	26086.8	441.0	0.05	2.5	2.0	2.0	4.0	5.0
B-PGRM-D-0.39(1)-4	1318.6	189.0	17.4	21.4	23653.5	497.4	0.05	2.7	2.1	2.0	4.3	5.5
B-PGRM-C-0.39(1)-20	1803.9	93.7	13.8	15.6	24407.2	740.8	0.05	3.5	2.5	2.0	5.0	7.2
B-PGRM-D-0.39(1)-20	1584.1	135.9	15.6	19.2	26398.0	717.8	0.05	4.0	2.6	1.7	4.5	6.8
B-PGRM-C-1.18(3)-4	1203.5	416.6	50.4	71.1	70308.8	194.7	0.12	2.3	1.9	2.7	5.1	6.1
B-PGRM-D-1.18(3)-4	1225.1	376.8	45.0	65.7	67909.4	184.0	0.12	2.5	2.0	2.7	5.3	6.6
B-PGRM-C-1.18(3)-20	1683.1	134.6	20.7	41.7	60461.3	279.1	0.12	2.2	1.8	2.7	5.1	6.0
B-PGRM-D-1.18(3)-20	1469.0	81.5	31.5	56.2	72866.9	284.3	0.12	3.4	2.4	2.9	6.9	9.7
B-PGRM-C-1.58(4)-4	1065.1	483.3	70.8	109.2	96006.2	137.3	0.17	2.0	1.8	2.9	5.0	5.9
B-PGRM-D-1.58(4)-4	1198.8	419.7	67.2	81.0	76803.4	117.4	0.17	2.1	1.8	2.4	4.3	5.1
B-PGRM-C-1.58(4)-20	1541.0	201.4	38.4	67.0	91635.6	184.6	0.17	3.0	2.3	2.6	5.9	7.9
B-PGRM-D-1.58(4)-20	1389.0	154.5	42.0	71.0	88281.0	191.0	0.17	3.4	2.4	2.9	7.0	9.9
B-PGRM-C-2.37(6)-20	1487.4	164.2	142.5	173.3	208397.9	65.3	0.26	2.3	1.9	3.0	5.8	7.0
B-PGRM-D-2.37(6)-20	1533.4	220.4	120.0	170.3	215156.9	61.8	0.27	2.0	1.7	2.9	4.9	5.6

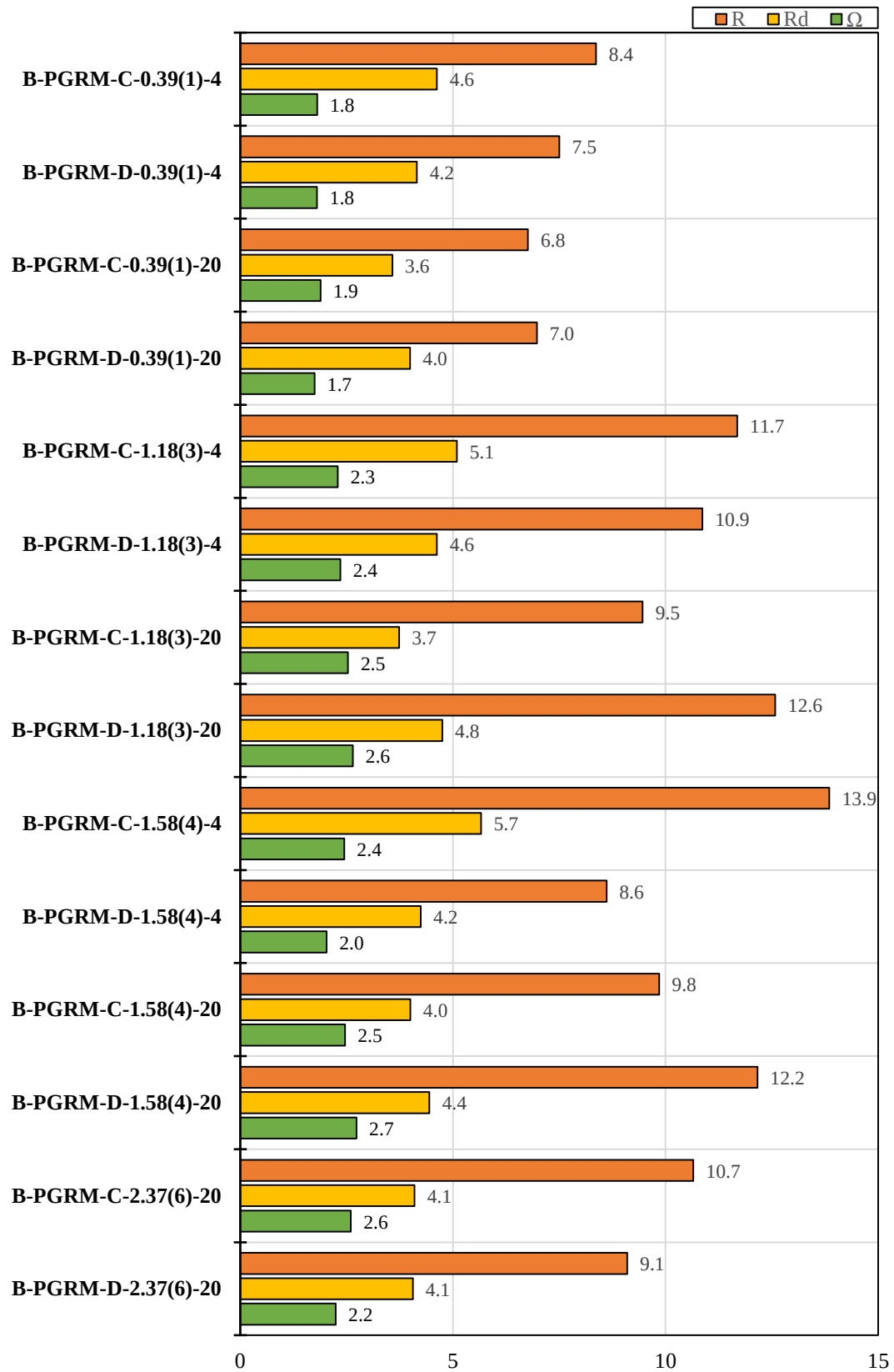
Figure B.1: Results of seismic parameters R , R_d , and Ω (BL1 approach).

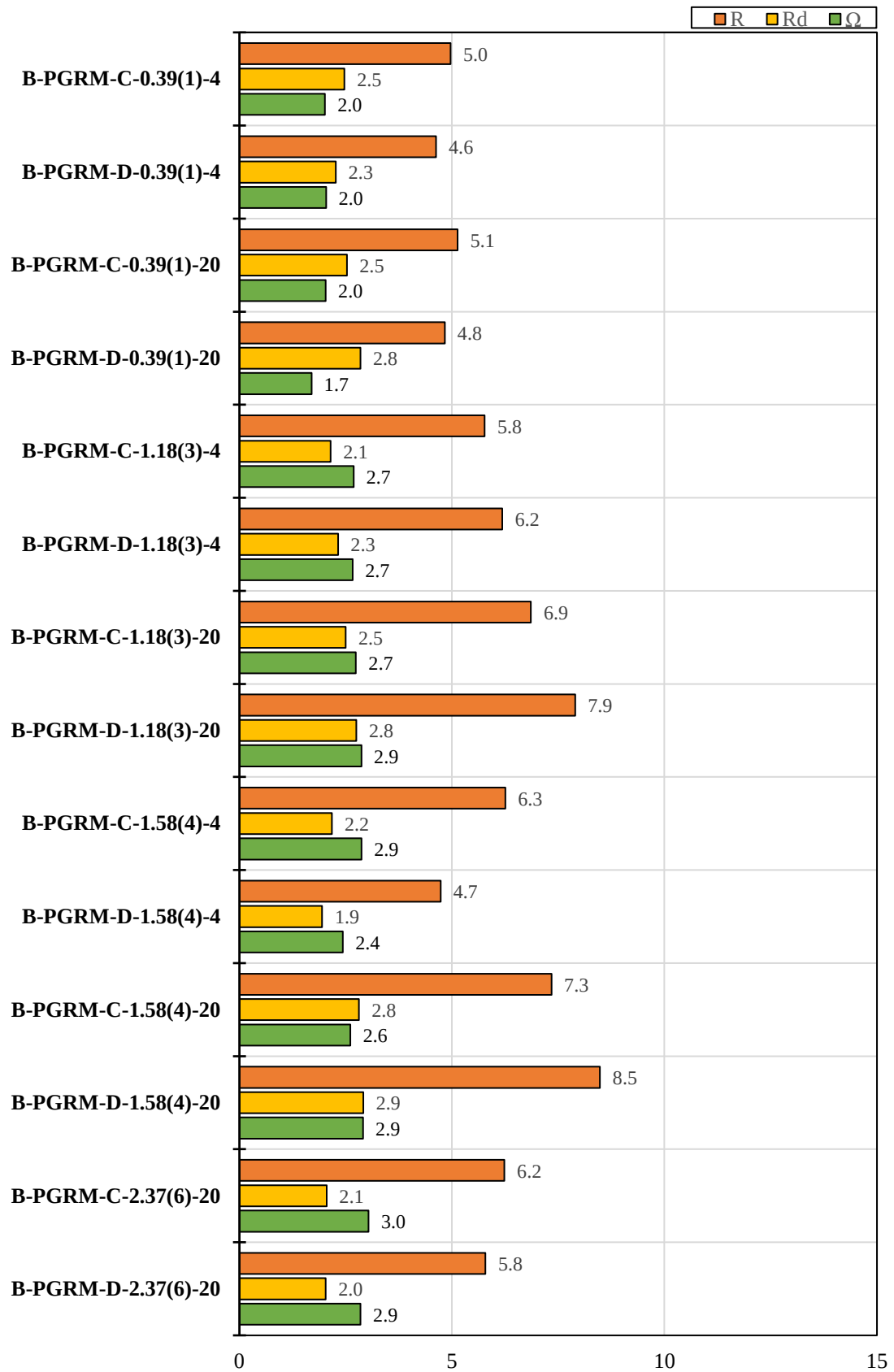
Figure B.2: Results of seismic parameters R , R_d , and Ω (BL2 approach).

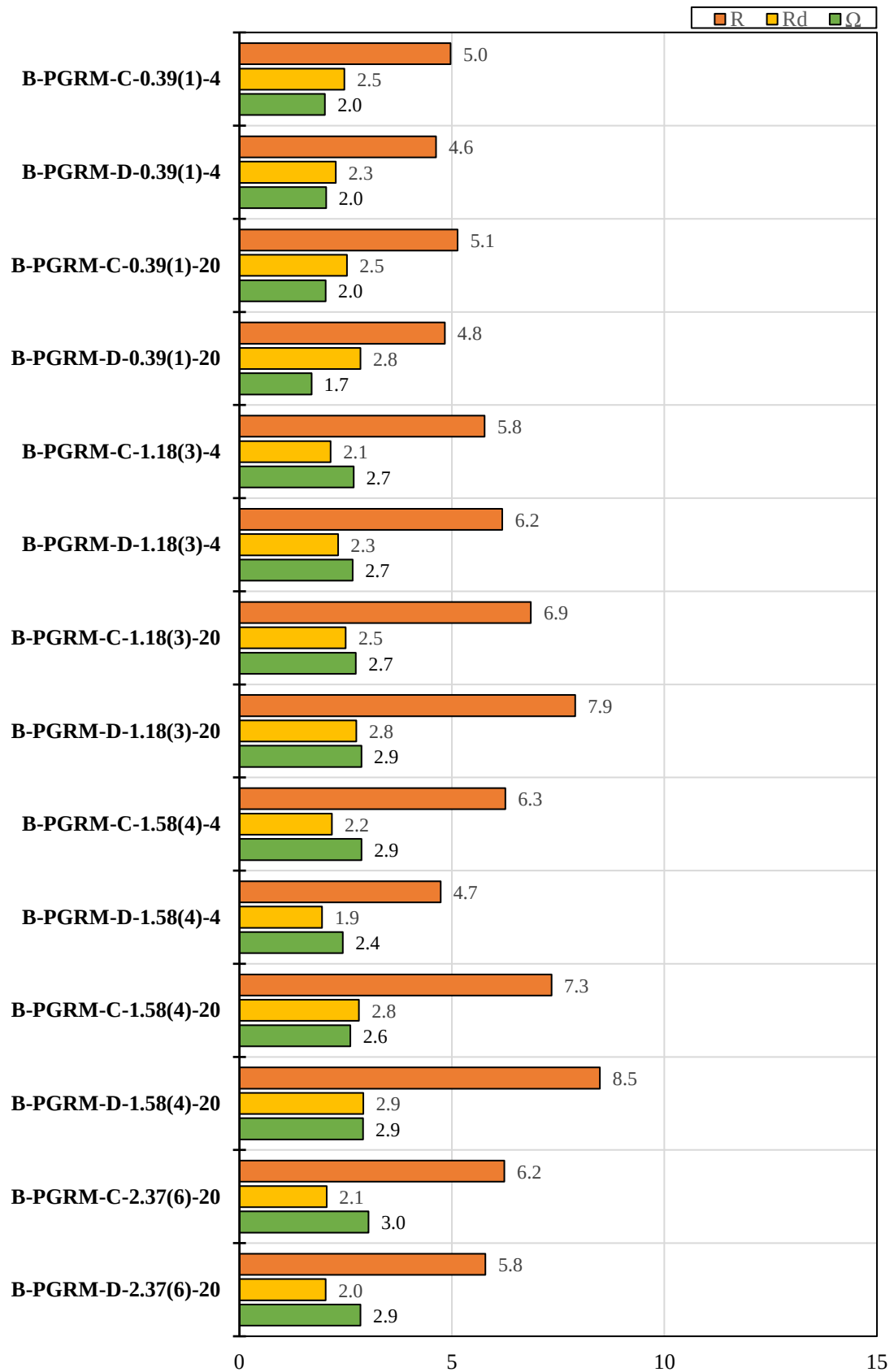
Figure B.3: Results of seismic parameters R , R_d , and Ω (BL1* approach).

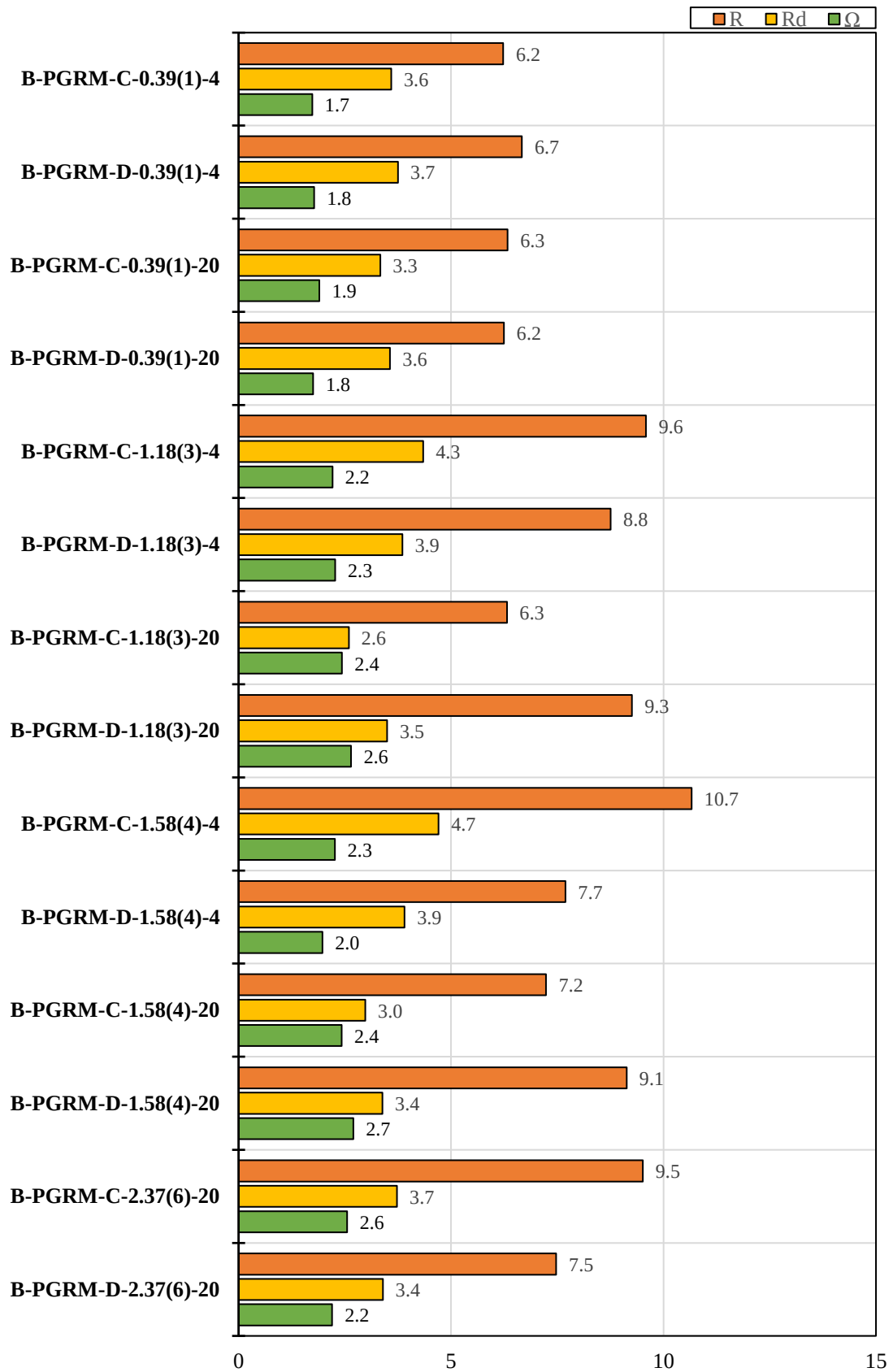
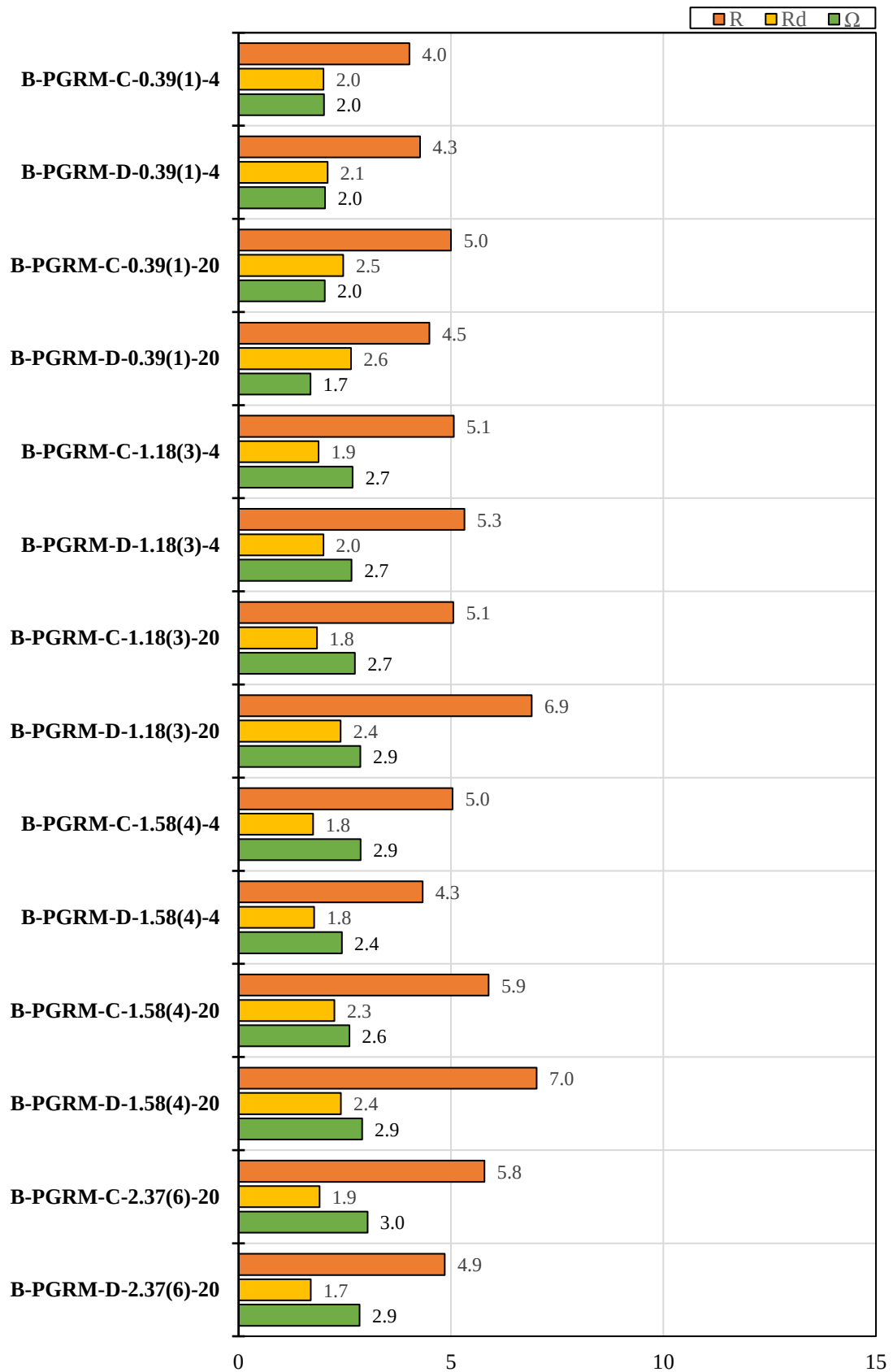
Figure B.4: Results of seismic parameters R , R_d , and Ω (TL* approach).

Figure B.5: Results of seismic parameters R , R_d , and Ω (BL2* approach).

APPENDIX C – Structural and reinforcement detailing of the walls in buildings B1 and B2 of Chapter 6

Figure C.1: Reinforcement detailing of the wall W1 in the Building B1 (single story, dimensions in mm).

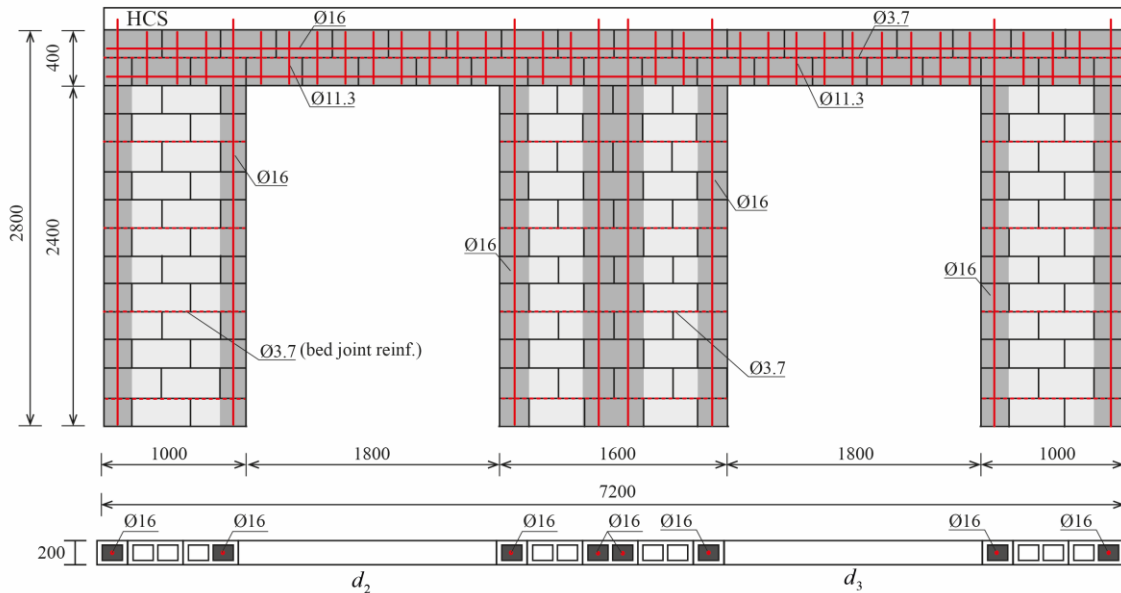


Figure C.2: Reinforcement detailing of the wall W2 in the Building B1 (single story, dimensions in mm).

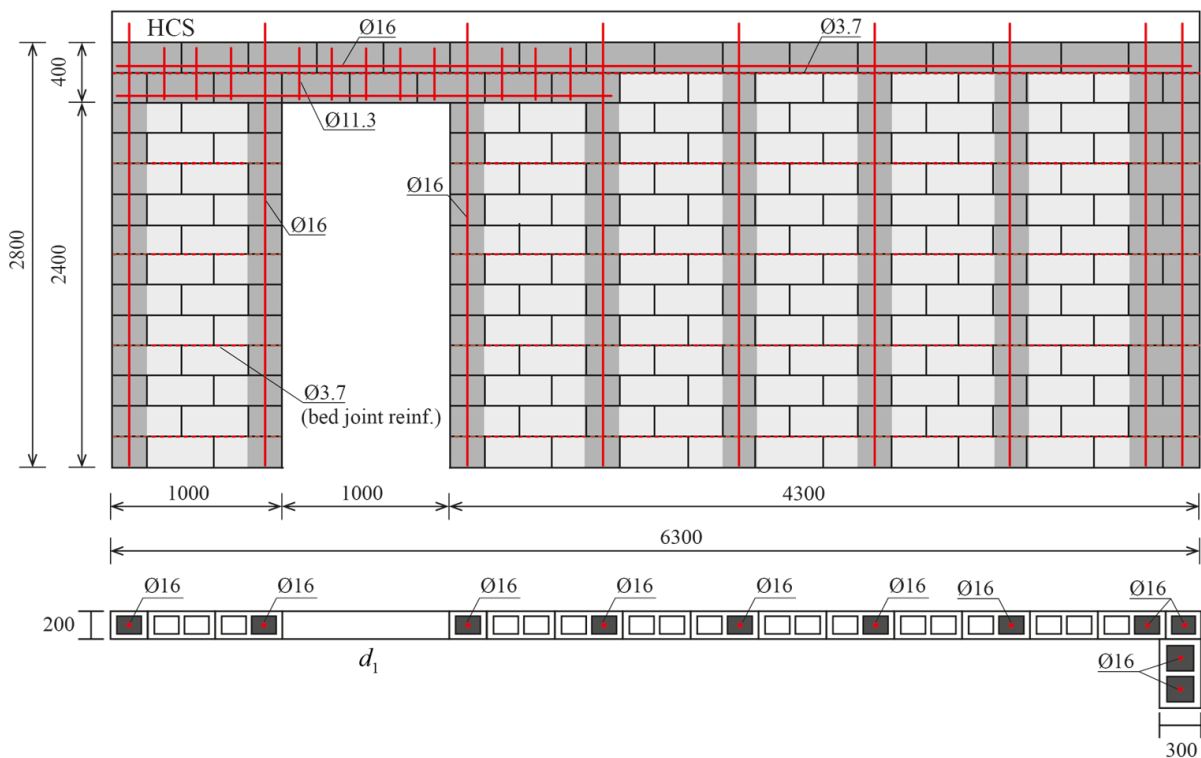


Figure C.3: Reinforcement detailing of the wall W3 in the Building B1 (single storey, dimensions in mm).

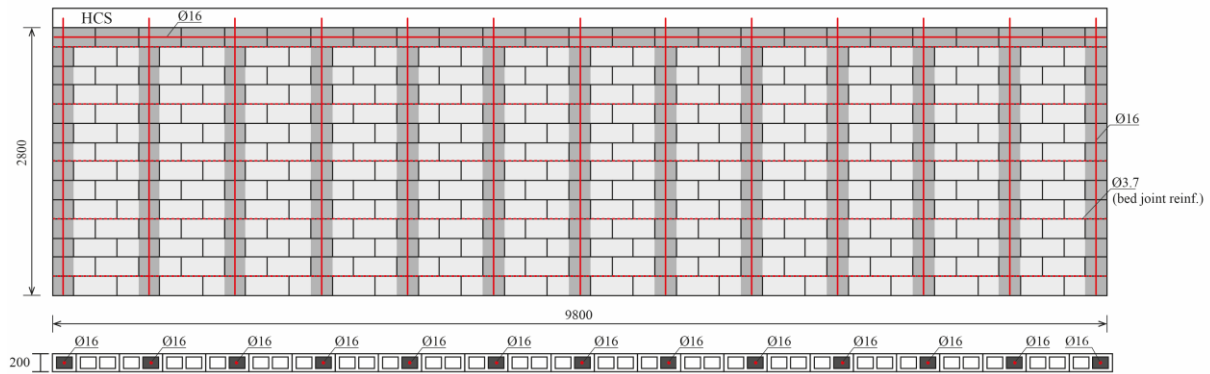


Figure C.4: Reinforcement detailing of the wall W4 in the Building B1 (single story, dimensions in mm).

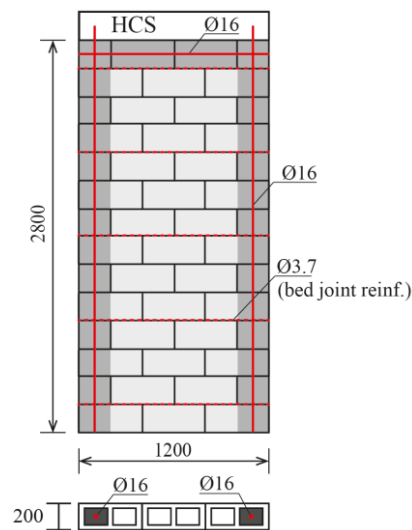


Figure C.5: Reinforcement detailing of the wall W5 in the Building B1 (single storey, dimensions in mm).

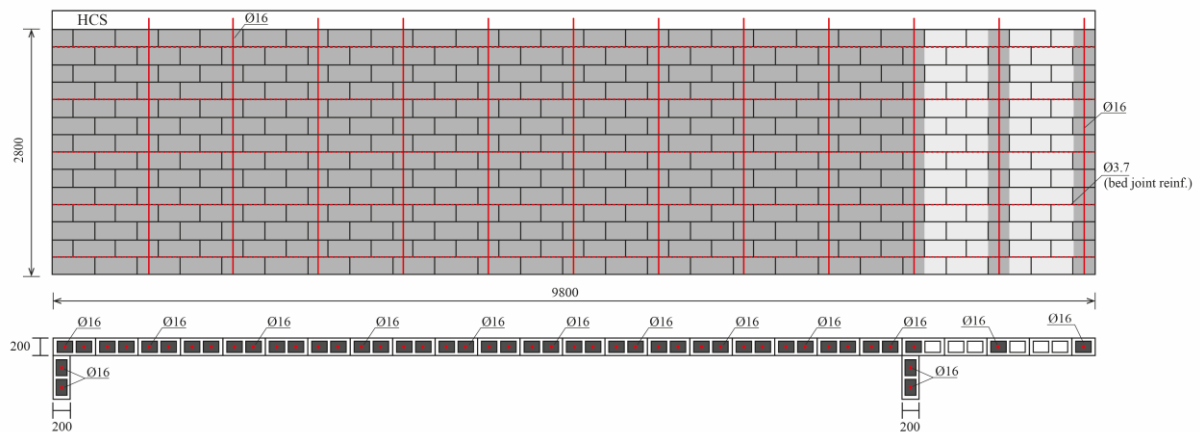


Figure C.6: Reinforcement detailing of the wall W6 in the Building B1 (varied detailing along each storey, dimension in mm).



Figure C.7: Reinforcement detailing of the wall W7 in the Building B1 (varied detailing along each storey, dimension in mm).

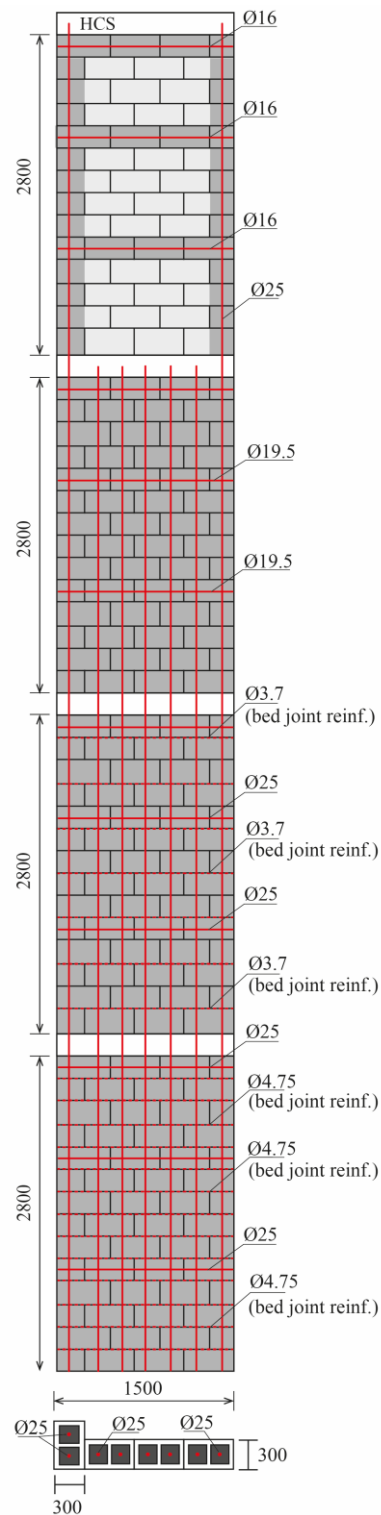


Figure C.8: Reinforcement detailing of the wall W1 in the Building B2 (single storey, dimensions in mm).

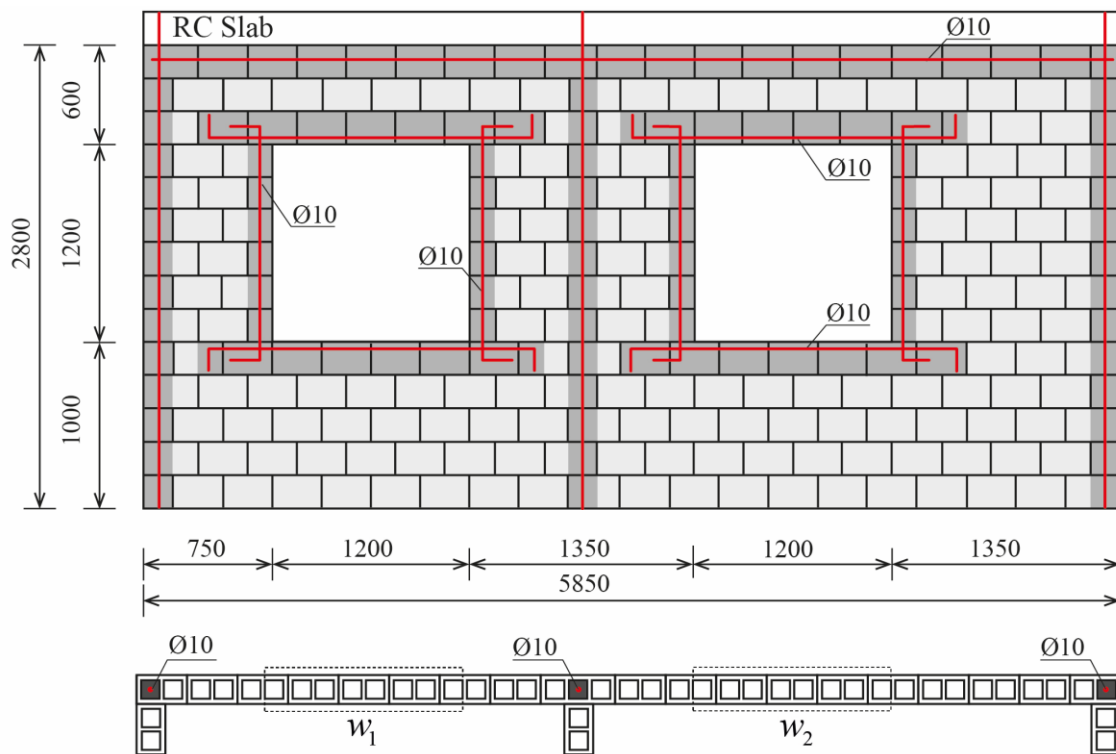


Figure C.9: Reinforcement detailing of the wall W2 in the Building B2 (single storey, dimensions in mm).

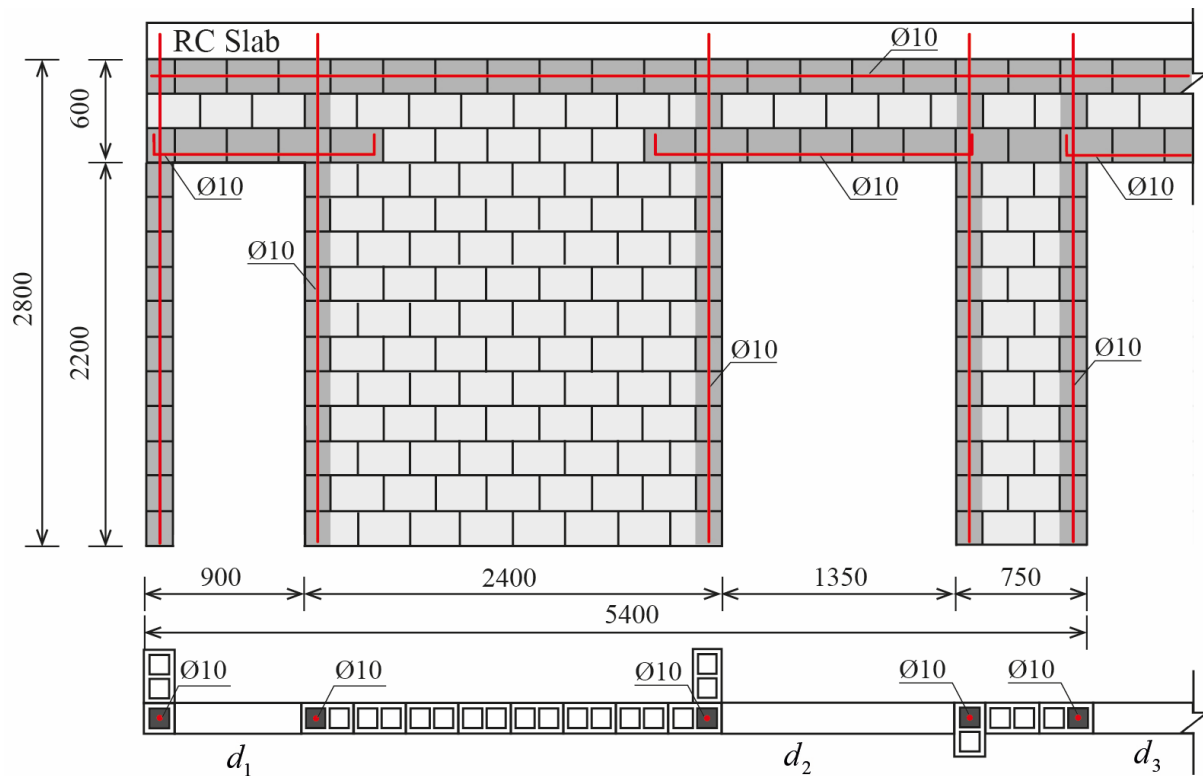


Figure C.10: Reinforcement detailing of the wall W3 in the Building B2 (single storey, dimensions in mm).

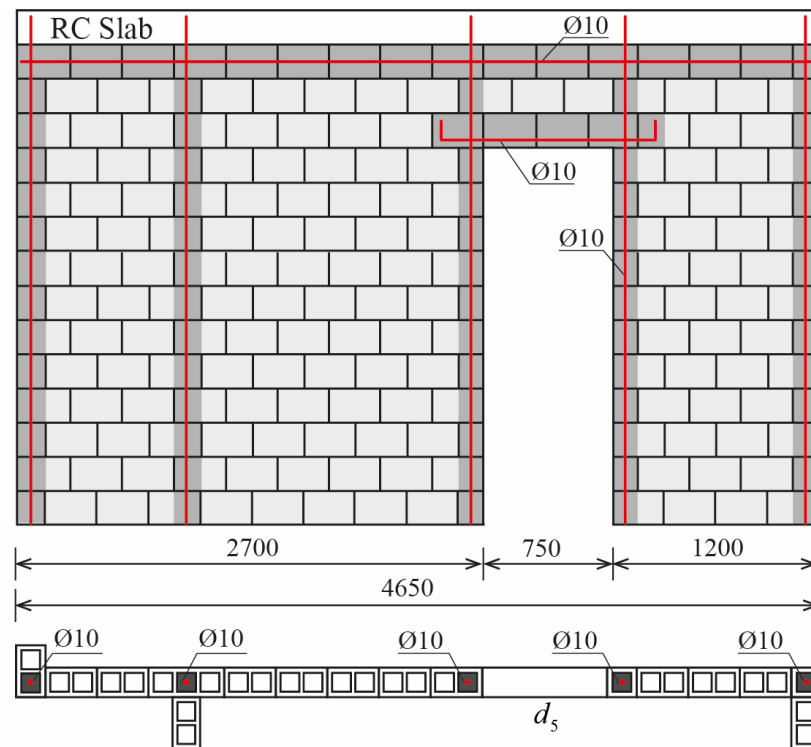


Figure C.11: Reinforcement detailing of the wall W4 in the Building B2 (single storey, dimensions in mm).

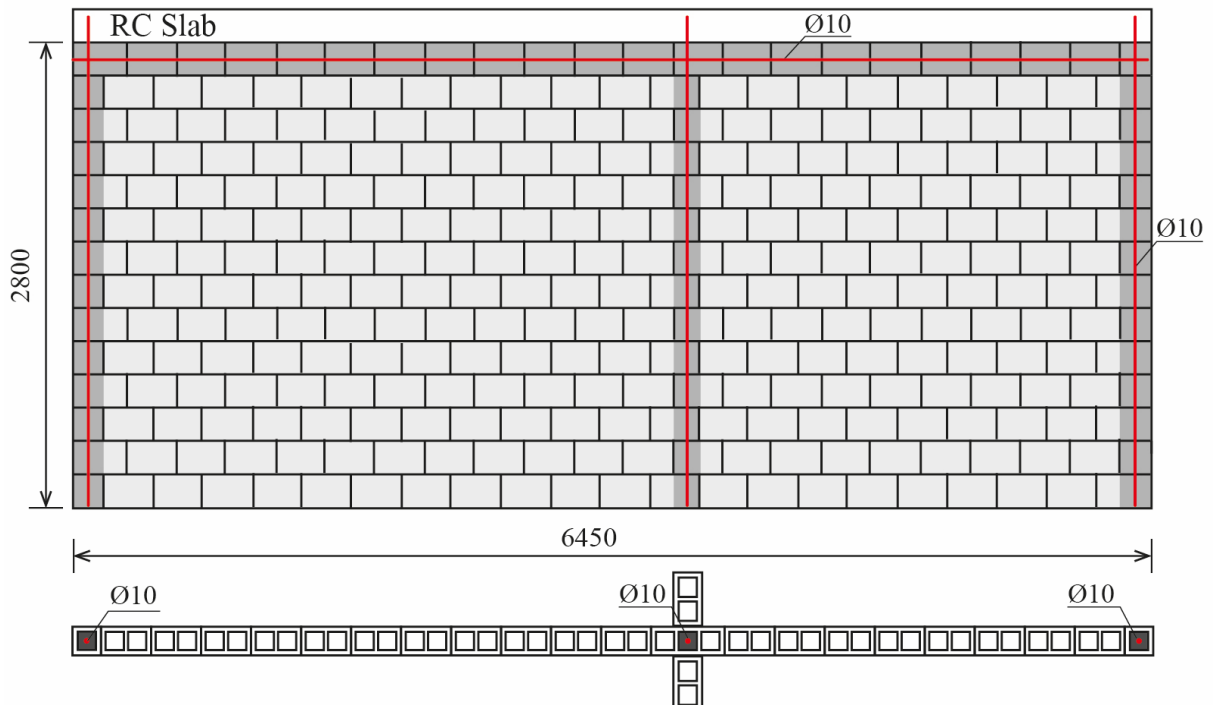


Figure C.12: Reinforcement detailing of the wall W5 in the Building B2 (single storey, dimensions in mm).

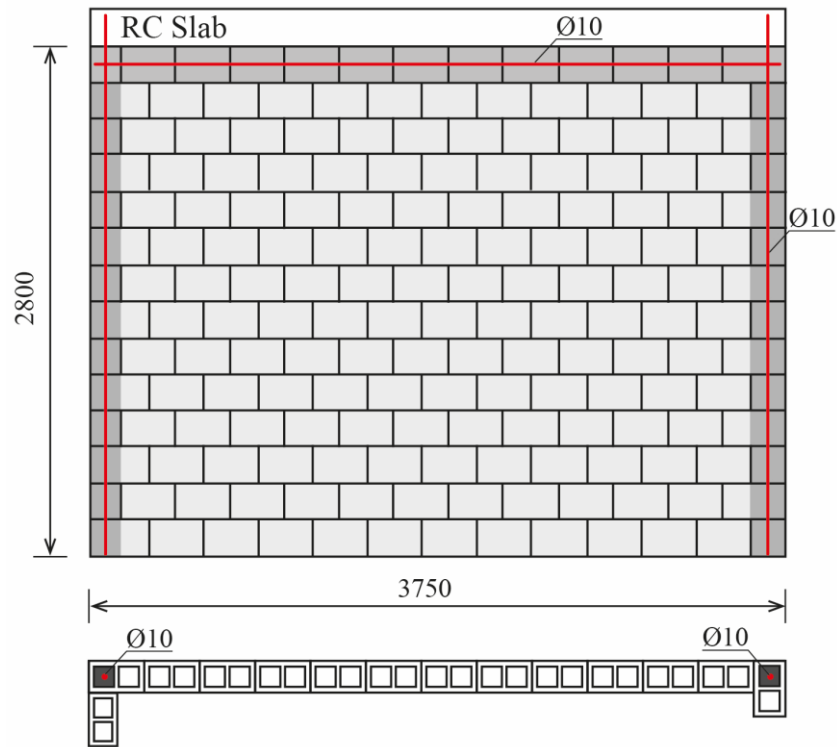


Figure C.13: Reinforcement detailing of the wall W6 in the Building B2 (single storey, dimensions in mm).

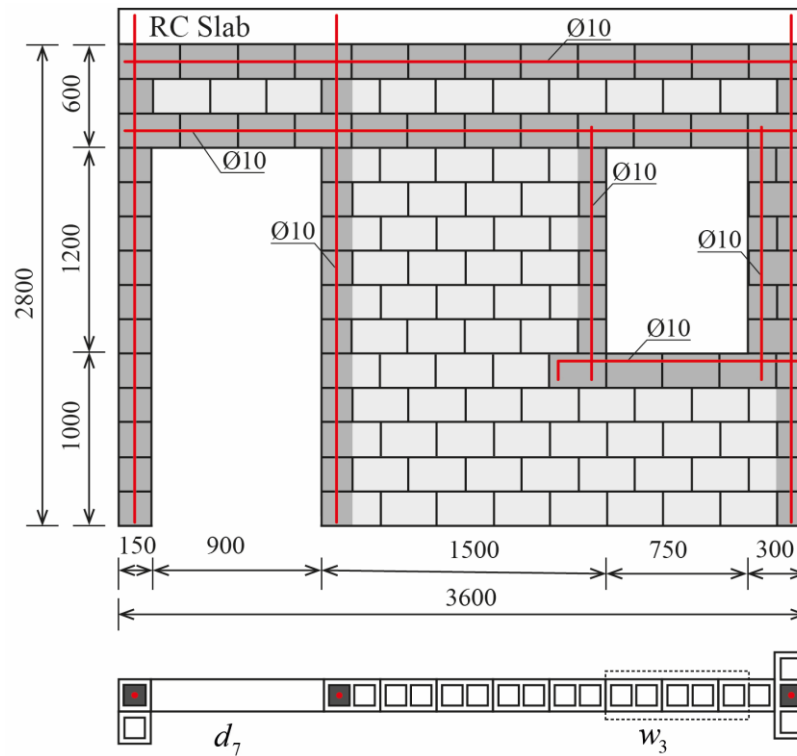


Figure C.14: Reinforcement detailing of the wall W7 in the Building B2 (single storey, dimensions in mm).

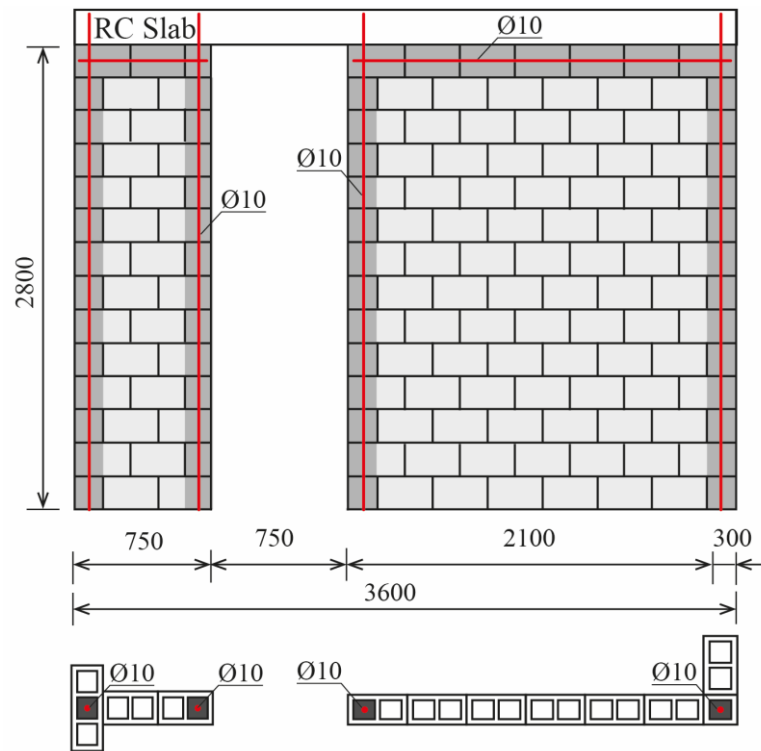


Figure C.15: Reinforcement detailing of the wall W8 in the Building B2 (single storey, dimensions in mm).

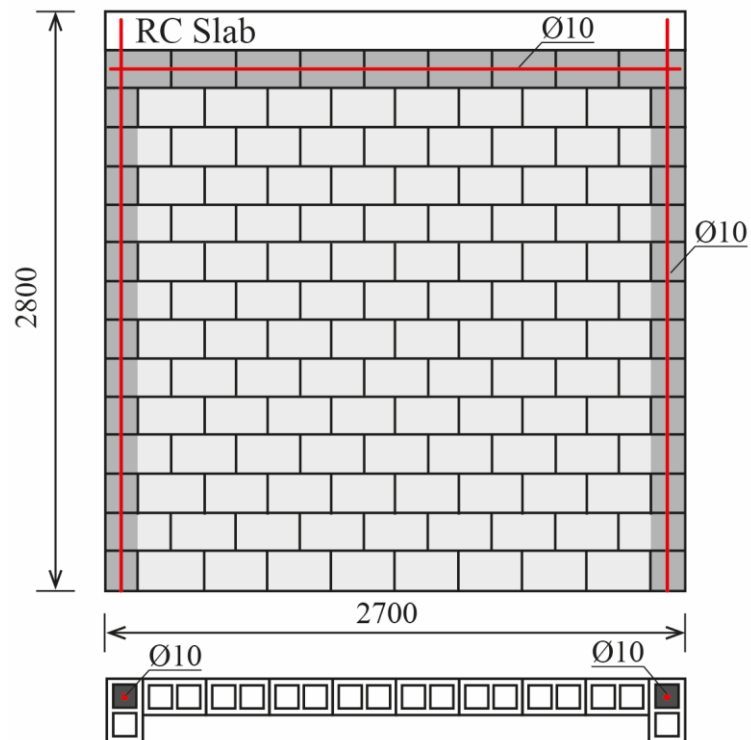


Figure C.16: Reinforcement detailing of the wall W9 in the Building B2 (single storey, dimensions in mm).

